REVISION RECORD FOR THE STATE OF CALIFORNIA

SUPPLEMENT

October 12, 2006

2001 Title 24, Part 2, California Building Code

**PLEASE NOTE: The date of this Supplement is for identification purposes only.
See the History Note Appendix for the adoption and effective dates of the provisions.**

It is suggested that the section number as well as the page number be checked when inserting this material and removing the superseded material. In case of doubt, rely on the section numbers rather than the page numbers because the section numbers must run consecutively.

It is further suggested that the superseded material be retained with this revision record sheet so that the prior wording of any section can be easily ascertained. Please keep the removed pages with this revision page for future reference.

NOTE

Due to the fact that the application date for a building permit establishes the California Building Standards code provisions that are effective at the local level, which apply to the plans, specifications, and construction for that permit, it is strongly recommended that the removed pages be retained for historical reference.

VOLUME 2

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CHAPTER 21A—MASONRY

ENFORCING AUTHORITY	LOCAL BUILDING OFFICIAL						LOCAL FIRE OFFICIAL	LOCAL HEALTH OFFICIAL	STATE AGENCY										
ADOPTING AGENCY	CEC	CA	HCD 1/AC	HCD 1	HCD 2	DSA AC	SFM	DHS	DWR	AGR	BOC	BSC	DSA SS	OSHPD 1	OSHPD 2	OSHPD 3	OSHPD 4	DOSH*	SL
Adopt entire California chapter													X	X			X		

*DOSH has not adopted the 1997 edition of the *Uniform Building Code*. The 1995 edition of the *California Building Code* remains effective.
The ◆ designation indicates that the State Fire Marshal's adoption of this chapter or individual sections is applicable to structures subject to HCD 1 and/or HCD 2.

CHAPTER 22—STEEL

ENFORCING AUTHORITY	LOCAL BUILDING OFFICIAL						LOCAL FIRE OFFICIAL	LOCAL HEALTH OFFICIAL	STATE AGENCY										
ADOPTING AGENCY	CEC	CA	HCD 1/AC	HCD 1	HCD 2	DSA AC	SFM	DHS	DWR	AGR	BOC	BSC	DSA SS	OSHPD 1	OSHPD 2	OSHPD 3	OSHPD 4	DOSH*	SL
Adopt entire UBC chapter without amendments				X	X							X		X	X		X		
Adopt entire UBC chapter as amended (amended sections listed below)																			
Adopt only those sections that are listed below																			

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CHAPTER 22A—STEEL

ENFORCING AUTHORITY	LOCAL BUILDING OFFICIAL						LOCAL FIRE OFFICIAL	LOCAL HEALTH OFFICIAL	STATE AGENCY										
ADOPTING AGENCY	CEC	CA	HCD 1/AC	HCD 1	HCD 2	DSA AC	SFM	DHS	DWR	AGR	BOC	BSC	DSA SS	OSHPD 1	OSHPD 2	OSHPD 3	OSHPD 4	DOSH*	SL
Adopt entire California chapter													X	X			X		

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CHAPTER 22B—STEEL

ENFORCING AUTHORITY	LOCAL BUILDING OFFICIAL						LOCAL FIRE OFFICIAL	LOCAL HEALTH OFFICIAL	STATE AGENCY										
ADOPTING AGENCY	CEC	CA	HCD 1/AC	HCD 1	HCD 2	DSA AC	SFM	DHS	DWR	AGR	BOC	BSC	DSA SS	OSHPD 1	OSHPD 2	OSHPD 3	OSHPD 4	DOSH*	SL
Adopt entire CA chapter without amendments				X	X							X							
Adopt entire CA chapter as amended (amended sections listed below)																			
Adopt only those sections that are listed below												X							
Div. I - IV CA												X							
Div. V §2215B CA												X							
Div. VI - IX CA												X							

CHAPTER 23—WOOD

ENFORCING AUTHORITY	LOCAL BUILDING OFFICIAL						LOCAL FIRE OFFICIAL	LOCAL HEALTH OFFICIAL	STATE AGENCY										
			HCD											OSHPD					
ADOPTING AGENCY	CEC	CA	1/AC	1	2	DSA AC	SFM	DHS	DWR	AGR	BOC	BSC	DSA SS	1	2	3	4	DOSH*	SL
Adopt entire UBC chapter without amendments												X							
Adopt entire UBC chapter as amended (amended sections listed below)				X	X										X				
Adopt only those sections that are listed below																			
2304.2 CA				X															
2304.5 CA				X															
2306.3 CA																			
2306.5 CA																			
2306.7 CA																			
2306.14 CA																			
2321.1.1 CA															X				

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CHAPTER 23A—WOOD

ENFORCING AUTHORITY	LOCAL BUILDING OFFICIAL						LOCAL FIRE OFFICIAL	LOCAL HEALTH OFFICIAL	STATE AGENCY										
			HCD											OSHPD					
ADOPTING AGENCY	CEC	CA	1/AC	1	2	DSA AC	SFM	DHS	DWR	AGR	BOC	BSC	DSA SS	1	2	3	4	DOSH*	SL
Adopt entire California chapter													X	X			X		

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APPENDIX CHAPTER 16—STRUCTURAL FORCES

ENFORCING AUTHORITY	LOCAL BUILDING OFFICIAL						LOCAL FIRE OFFICIAL	LOCAL HEALTH OFFICIAL	STATE AGENCY										
			HCD											OSHPD					
ADOPTING AGENCY	CEC	CA	1/AC	1	2	DSA AC	SFM	DHS	DWR	AGR	BOC	BSC	DSA SS	1	2	3	4	DOSH*	SL
Adopt entire UBC chapter without amendments													X	X	X				
Adopt entire UBC chapter as amended (amended sections listed below)																			
Adopt only those sections that are listed below																			

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Volume 2

Chapters 1 through 15 are printed in Volume 1 of the *Uniform Building Code*.

Chapter 16
STRUCTURAL DESIGN REQUIREMENTS
Division I—GENERAL DESIGN REQUIREMENTS

SECTION 1601 — SCOPE

This chapter prescribes general design requirements applicable to all structures regulated by this code.

SECTION 1602 — DEFINITIONS

The following terms are defined for use in this code:

ALLOWABLE STRESS DESIGN is a method of proportioning structural elements such that computed stresses produced in the elements by the allowable stress load combinations do not exceed specified allowable stress (also called working stress design).

BALCONY, EXTERIOR, is an exterior floor system projecting from a structure and supported by that structure, with no additional independent supports.

DEAD LOADS consist of the weight of all materials and fixed equipment incorporated into the building or other structure.

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

FACTORED LOAD is the product of a load specified in Sections 1606 through 1611 and a load factor. See Section 1612.2 for combinations of factored loads.

LIMIT STATE is a condition in which a structure or component is judged either to be no longer useful for its intended function (serviceability limit state) or to be unsafe (strength limit state).

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

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

STRENGTH DESIGN is a method of proportioning structural elements such that the computed forces produced in the elements by the factored load combinations do not exceed the factored element strength. The term "strength design" is used in the design of concrete and masonry structures.

SECTION 1603 — NOTATIONS

D = dead load.

E = earthquake load set forth in Section 1630.1.

E_m = estimated maximum earthquake force that can be developed in the structure as set forth in Section 1630.1.1.

F = load due to fluids.

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

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

L_r = roof live load, including any permitted live load reduction.

P = ponding load.

S = snow load.

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

W = load due to wind pressure.

SECTION 1604 — STANDARDS

The standards listed below are recognized standards (see Section 3504).

1. Wind Design.
   - 1.1 ASCE 7, Chapter 6, Minimum Design Loads for Buildings and Other Structures
   - 1.2 ANSI EIA/TIA 222-E, Structural Standards for Steel Antenna Towers and Antenna Supporting Structures
   - 1.3 ANSI/NAAMM FP1001, Guide Specifications for the Design Loads of Metal Flagpoles

SECTION 1605 — DESIGN

1605.1 General. Buildings and other structures and all portions thereof shall be designed and constructed to sustain, within the limitations specified in this code, all loads set forth in Chapter 16 and elsewhere in this code, combined in accordance with Section 1612. Design shall be in accordance with Strength Design, Load and Resistance Factor Design or Allowable Stress Design methods, as permitted by the applicable materials chapters.

> **EXCEPTION:** Unless otherwise required by the building official, buildings or portions thereof that are constructed in accordance with the conventional light-framing requirements specified in Chapter 23 of this code shall be deemed to meet the requirements of this section.

1605.2 Rationality. Any system or method of construction to be used shall be based on a rational analysis in accordance with well-established principles of mechanics. Such analysis shall result in a system that provides a complete load path capable of transferring all loads and forces from their point of origin to the load-resisting elements. The analysis shall include, but not be limited to, the provisions of Sections 1605.2.1 through 1605.2.3.

1605.2.1 Distribution of horizontal shear. The total lateral force shall be distributed to the various vertical elements of the lateral-force-resisting system in proportion to their rigidities considering the rigidity of the horizontal bracing system or diaphragm. Rigid elements that are assumed not to be part of the lateral-force-resisting system may be incorporated into buildings, provided that their effect on the action of the system is considered and provided for in the design.

Provision shall be made for the increased forces induced on resisting elements of the structural system resulting from torsion due to eccentricity between the center of application of the lateral forces and the center of rigidity of the lateral-force-resisting system. For accidental torsion requirements for seismic design, see Section 1630.6.

1605.2.2 Stability against overturning.

|| CA *1605.2.2.1* Every structure shall be designed to resist the overturning effects caused by the lateral forces specified in this chapter. See Section 1611.6 for retaining walls, Section 1615 for wind and Section 1626 for seismic.

|| CA *1605.2.2.2 [For OSHPD 2]* Every structure shall be designed to resist the overturning effects caused by the lateral forces specified in this chapter. See Section 1611.6 for retaining walls, Section *1621* for wind and Section *1630.8* for seismic.

1605.2.3 Anchorage. Anchorage of the roof to walls and columns, and of walls and columns to foundations, shall be provided to resist the uplift and sliding forces that result from the application of the prescribed forces.

Concrete and masonry walls shall be anchored to all floors, roofs and other structural elements that provide lateral support for the wall. Such anchorage shall provide a positive direct connection capable of resisting the horizontal forces specified in this chapter but not less than the minimum forces in Section 1611.4. In addition, in Seismic Zones 3 and 4, diaphragm to wall anchorage using embedded straps shall have the straps attached to or hooked around the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet (1219 mm). Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall. See Sections 1632, 1633.2.8 and 1633.2.9 for earthquake design requirements.

1605.3 Erection of Structural Framing. Walls and structural framing shall be erected true and plumb in accordance with the design.

SECTION 1606 — DEAD LOADS

1606.1 General. Dead loads shall be as defined in Section 1602 and this section.

1606.2 Partition Loads. Floors in office buildings and other buildings where partition locations are subject to change shall be designed to support, in addition to all other loads, a uniformly distributed dead load equal to 20 pounds per square foot (psf) (0.96 kN/m^2) of floor area.

> **EXCEPTION:** Access floor systems shall be designed to support, in addition to all other loads, a uniformly distributed dead load not less than 10 psf (0.48 kN/m^2) of floor area.

SECTION 1607 — LIVE LOADS

1607.1 General. Live loads shall be the maximum loads expected by the intended use or occupancy but in no case shall be less than the loads required by this section.

1607.2 Critical Distribution of Live Loads. Where structural members are arranged to create continuity, members shall be designed using the loading conditions, which would cause maximum shear and bending moments. This requirement may be satisfied in accordance with the provisions of Section 1607.3.2 or 1607.4.2, where applicable.

1607.3 Floor Live Loads.

1607.3.1 General. Floors shall be designed for the unit live loads as set forth in Table 16-A. These loads shall be taken as the minimum live loads in pounds per square foot of horizontal projection to be used in the design of buildings for the occupancies listed, and loads at least equal shall be assumed for uses not listed in this section but that create or accommodate similar loadings.

Where it can be determined in designing floors that the actual live load will be greater than the value shown in Table 16-A, the actual live load shall be used in the design of such buildings or portions thereof. Special provisions shall be made for machine and apparatus loads.

1607.3.2 Distribution of uniform floor loads. Where uniform floor loads are involved, consideration may be limited to full dead load on all spans in combination with full live load on adjacent spans and alternate spans.

1607.3.3 Concentrated loads. Provision shall be made in designing floors for a concentrated load, *L,* as set forth in Table 16-A placed upon any space $2^1/_2$ feet (762 mm) square, wherever this load upon an otherwise unloaded floor would produce stresses greater than those caused by the uniform load required therefor.

Provision shall be made in areas where vehicles are used or stored for concentrated loads, *L,* consisting of two or more loads spaced 5 feet (1524 mm) nominally on center without uniform live loads. Each load shall be 40 percent of the gross weight of the maximum-size vehicle to be accommodated. Parking garages for the storage of private or pleasure-type motor vehicles with no repair or refueling shall have a floor system designed for a concentrated load of not less than 2,000 pounds (8.9 kN) acting on an area of 20 square inches (12 903 mm^2) without uniform live loads. The condition of concentrated or uniform live load, combined in accordance with Section 1612.2 or 1612.3 as appropriate, producing the greatest stresses shall govern.

1607.3.4 Special loads. Provision shall be made for the special vertical and lateral loads as set forth in Table 16-B.

1607.3.5 Live loads posted. The live loads for which each floor or portion thereof of a commercial or industrial building is or has been designed shall have such design live loads conspicuously posted by the owner in that part of each story in which they apply, using durable metal signs, and it shall be unlawful to remove or deface such notices. The occupant of the building shall be responsible for keeping the actual load below the allowable limits.

1607.4 Roof Live Loads.

1607.4.1 General. Roofs shall be designed for the unit live loads, L_r, set forth in Table 16-C. The live loads shall be assumed to act vertically upon the area projected on a horizontal plane.

1607.4.2 Distribution of loads. Where uniform roof loads are involved in the design of structural members arranged to create continuity, consideration may be limited to full dead loads on all spans in combination with full roof live loads on adjacent spans and on alternate spans.

> **EXCEPTION:** Alternate span loading need not be considered where the uniform roof live load is 20 psf (0.96 kN/m^2) or more or where load combinations, including snow load, result in larger members or connections.

For those conditions where light-gage metal preformed structural sheets serve as the support and finish of roofs, roof structural members arranged to create continuity shall be considered adequate if designed for full dead loads on all spans in combination with the most critical one of the following superimposed loads:

1. Snow load in accordance with Section 1614.

2. The uniform roof live load, L_r, set forth in Table 16-C on all spans.

3. A concentrated gravity load, L_r, of 2,000 pounds (8.9 kN) placed on any span supporting a tributary area greater than 200 square feet (18.58 m^2) to create maximum stresses in the member, whenever this loading creates greater stresses than those caused by the uniform live load. The concentrated load shall be placed on the member over a length of $2^1/_2$ feet (762 mm) along the span. The concentrated load need not be applied to more than one span simultaneously.

4. Water accumulation as prescribed in Section 1611.7.

1607.4.3 Unbalanced loading. Unbalanced loads shall be used where such loading will result in larger members or connections. Trusses and arches shall be designed to resist the stresses caused by unit live loads on one half of the span if such loading results in reverse stresses, or stresses greater in any portion than the stresses produced by the required unit live load on the entire span. For roofs whose structures are composed of a stressed shell, framed or solid, wherein stresses caused by any point loading are distributed throughout the area of the shell, the requirements for unbalanced unit live load design may be reduced 50 percent.

1607.4.4 Special roof loads. Roofs to be used for special purposes shall be designed for appropriate loads as approved by the building official.

Greenhouse roof bars, purlins and rafters shall be designed to carry a 100-pound-minimum (444.8 N) concentrated load, L_r, in addition to the uniform live load.

1607.5 Reduction of Live Loads. The design live load determined using the unit live loads as set forth in Table 16-A for floors and Table 16-C, Method 2, for roofs may be reduced on any member supporting more than 150 square feet (13.94 m^2), including flat slabs, except for floors in places of public assembly and for live loads greater than 100 psf (4.79 kN/m^2), in accordance with the following formula:

$$R = r\,(A - 150) \qquad (7\text{-}1)$$

For **SI:**

$$R = r\,(A - 13.94)$$

The reduction shall not exceed 40 percent for members receiving load from one level only, 60 percent for other members or R, as determined by the following formula:

$$R = 23.1\,(1 + D/L) \qquad (7\text{-}2)$$

WHERE:

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

D = dead load per square foot (m^2) of area supported by the member.

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

R = reduction in percentage.

r = rate of reduction equal to 0.08 percent for floors. See Table 16-C for roofs.

For storage loads exceeding 100 psf (4.79 kN/m^2), no reduction shall be made, except that design live loads on columns may be reduced 20 percent.

The live load reduction shall not exceed 40 percent in garages for the storage of private pleasure cars having a capacity of not more than nine passengers per vehicle.

1607.6 Alternate Floor Live Load Reduction. As an alternate to Formula (7-1), the unit live loads set forth in Table 16-A may be reduced in accordance with Formula (7-3) on any member, including flat slabs, having an influence area of 400 square feet (37.2 m^2) or more.

$$L = L_o\left(0.25 + \frac{15}{\sqrt{A_I}}\right) \qquad (7\text{-}3)$$

For **SI:**

$$L = L_o\left[0.25 + 4.57\left(\frac{1}{\sqrt{A_I}}\right)\right]$$

WHERE:

A_I = influence area, in square feet (m^2). The influence area A_I is four times the tributary area for a column, two times the tributary area for a beam, equal to the panel area for a two-way slab, and equal to the product of the span and the full flange width for a precast T-beam.

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

L_o = unreduced design live load per square foot (m^2) of area supported by the member (Table 16-A).

The reduced live load shall not be less than 50 percent of the unit live load L_o for members receiving load from one level only, nor less than 40 percent of the unit live load L_o for other members.

SECTION 1608 — SNOW LOADS

Snow loads shall be determined in accordance with Chapter 16, Division II.

SECTION 1609 — WIND LOADS

Wind loads shall be determined in accordance with Chapter 16, Division III.

SECTION 1610 — EARTHQUAKE LOADS

Earthquake loads shall be determined in accordance with Chapter 16, Division IV.

SECTION 1611 — OTHER MINIMUM LOADS

1611.1 General. In addition to the other design loads specified in this chapter, structures shall be designed to resist the loads specified in this section and the special loads set forth in Table 16-B.

1611.2 Other Loads. Buildings and other structures and portions thereof shall be designed to resist all loads due to applicable fluid pressures, F, lateral soil pressures, H, ponding loads, P, and self-straining forces, T. See Section 1611.7 for ponding loads for roofs.

1611.3 Impact Loads. Impact loads shall be included in the design of any structure where impact loads occur.

1611.4 Anchorage of Concrete and Masonry Walls. Concrete and masonry walls shall be anchored as required by Section 1605.2.3. Such anchorage shall be capable of resisting the load combinations of Section 1612.2 or 1612.3 using the greater of the wind or earthquake loads required by this chapter or a minimum horizontal force of 280 pounds per linear foot (4.09 kN/m) of wall, substituted for E.

1611.5 Interior Wall Loads. Interior walls, permanent partitions and temporary partitions that exceed 6 feet (1829 mm) in height shall be designed to resist all loads to which they are subjected but not less than a load, L, of 5 psf (0.24 kN/m^2) applied perpendicular to the walls. The 5 psf (0.24 kN/m^2) load need not be applied simultaneously with wind or seismic loads. The deflection of such walls under a load of 5 psf (0.24 kN/m^2) shall not exceed $^1/_{240}$ of the span for walls with brittle finishes and $^1/_{120}$ of the span for walls with flexible finishes. See Table 16-O for earthquake design requirements where such requirements are more restrictive.

> **EXCEPTION:** Flexible, folding or portable partitions are not required to meet the load and deflection criteria but must be anchored to the supporting structure to meet the provisions of this code. *[For OSHPD 2] See Table 16-O and Section 601.5.3 as adopted by the State Fire Marshal.*

1611.6 Retaining Walls. Retaining walls shall be designed to resist loads due to the lateral pressure of retained material in accordance with accepted engineering practice. Walls retaining drained soil, where the surface of the retained soil is level, shall be designed for a load, H, equivalent to that exerted by a fluid weighing not less than 30 psf per foot of depth (4.71 kN/m^2/m) and having a depth equal to that of the retained soil. Any surcharge shall be in addition to the equivalent fluid pressure.

Retaining walls shall be designed to resist sliding by at least 1.5 times the lateral force and overturning by at least 1.5 times the overturning moment, using allowable stress design loads.

1611.7 Water Accumulation.

1611.7.1 All roofs shall be designed with sufficient slope or camber to ensure adequate drainage after the long-term deflection from dead load or shall be designed to resist ponding load, P, combined in accordance with Section 1612.2 or 1612.3. Ponding load shall include water accumulation from any source, including snow, due to deflection. See Section 1506 and Table 16-C, Footnote 3, for drainage slope. See Section 1615 for deflection criteria.

1611.7.2 [For BSC] All roofs shall be designed with sufficient slope or camber to ensure adequate drainage after the long-term deflection from dead load or shall be designed to resist ponding load, P, combined in accordance with Section 1612.2 or 1612.3. Ponding load shall include water accumulation from any source, including snow, due to deflection. See Section 1506 and Table 16-C, Footnote 3, for drainage slope. See Section *1613* for deflection criteria.

1611.8 Hydrostatic Uplift. All foundations, slabs and other footings subjected to water pressure shall be designed to resist a uniformly distributed uplift load, F, equal to the full hydrostatic pressure.

1611.9 Flood-resistant Construction. For flood-resistant construction requirements, where specifically adopted, see Appendix Chapter 31, Division I.

1611.10 Heliport and Helistop Landing Areas. In addition to other design requirements of this chapter, heliport and helistop landing or touchdown areas shall be designed for the following loads, combined in accordance with Section 1612.2 or 1612.3:

1. Dead load plus actual weight of the helicopter.

2. Dead load plus a single concentrated impact load, L, covering 1 square foot (0.093 m^2) of 0.75 times the fully loaded weight of the helicopter if it is equipped with hydraulic-type shock absorbers, or 1.5 times the fully loaded weight of the helicopter if it is equipped with a rigid or skid-type landing gear.

3. The dead load plus a uniform live load, L, of 100 psf (4.8 kN/m^2). The required live load may be reduced in accordance with Section 1607.5 or 1607.6.

1611.11 Prefabricated Construction.

1611.11.1 Connections. Every device used to connect prefabricated assemblies shall be designed as required by this code and shall be capable of developing the strength of the members connected, except in the case of members forming part of a structural frame designed as specified in this chapter. Connections shall be capable of withstanding uplift forces as specified in this chapter.

1611.11.2 Pipes and conduit. In structural design, due allowance shall be made for any material to be removed for the installation of pipes, conduits or other equipment.

1611.11.3 Tests and inspections. See Section 1704 for requirements for tests and inspections of prefabricated construction.

SECTION 1612 — COMBINATIONS OF LOADS

1612.1 General. Buildings and other structures and all portions thereof shall be designed to resist the load combinations specified in Section 1612.2 or 1612.3 and, where required by Chapter 16, Division IV, or Chapters 18 through 23, the special seismic load combinations of Section 1612.4.

The most critical effect can occur when one or more of the contributing loads are not acting. All applicable loads shall be considered, including both earthquake and wind, in accordance with the specified load combinations.

1612.2 Load Combinations Using Strength Design or Load and Resistance Factor Design.

1612.2.1 Basic load combinations. Where Load and Resistance Factor Design (Strength Design) is used, structures and all portions thereof shall resist the most critical effects from the following combinations of factored loads:

$$1.4D \qquad (12\text{-}1)$$

$$1.2D + 1.6L + 0.5\,(L_r \text{ or } S) \qquad (12\text{-}2)$$

$$1.2D + 1.6\,(L_r \text{ or } S) + (f_1 L \text{ or } 0.8W) \qquad (12\text{-}3)$$

$$1.2D + 1.3W + f_1 L + 0.5\,(L_r \text{ or } S) \qquad (12\text{-}4)$$

$$1.2D \pm 1.0E + (f_1 L + f_2 S) \qquad (12\text{-}5)$$

$$0.9D \pm (1.0\rho E_h \text{ or } 1.3W) \qquad (12\text{-}6)$$

WHERE:

E = *load effects of earthquake, or related internal moments and forces.*

E_h = *the earthquake load due to the base shear, V, as set forth in Section 1630.2 or the design lateral force, F_p, as set forth in Section 1632.*

f_1 = 1.0 for floors in places of public assembly, for live loads in excess of 100 psf (4.9 kN/m^2), and for garage live load.

= 0.5 for other live loads.

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

= 0.2 for other roof configurations.

> **EXCEPTIONS:** 1. Factored load combinations for concrete per Section 1909.2 where load combinations do not include seismic forces.
> 2. Where other factored load combinations are specifically required by the provisions of this code.

1612.2.2 Other loads. Where F, H, P or T are to be considered in design, each applicable load shall be added to the above combinations factored as follows: 1.3F, 1.6H, 1.2P and 1.2T.

1612.3 Load Combinations Using Allowable Stress Design.

1612.3.1 Basic load combinations. Where allowable stress design (working stress design) is used, structures and all portions thereof shall resist the most critical effects resulting from the following combinations of loads:

$$D \qquad (12\text{-}7)$$

$$D + L + (L_r \text{ or } S) \qquad (12\text{-}8)$$

$$D + \left(W \text{ or } \frac{E}{1.4}\right) \qquad (12\text{-}9)$$

$$0.9D \pm \frac{E}{1.4} \qquad (12\text{-}10)$$

$$D + 0.75\left[L + (L_r \text{ or } S) + \left(W \text{ or } \frac{E}{1.4}\right)\right] \qquad (12\text{-}11)$$

No increase in allowable stresses shall be used with these load combinations except as specifically permitted elsewhere in this code.

1612.3.2 Alternate basic load combinations.

1612.3.2.1 In lieu of the basic load combinations specified in Section 1612.3.1, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following load combinations. When using these alternate basic load combinations a one-third increase shall be permitted in allowable stresses for all combinations, including *W* or *E*.

$$D + L + (L_r \text{ or } S) \qquad (12\text{-}12)$$

$$D + L + \left(W \text{ or } \frac{E}{1.4}\right) \qquad (12\text{-}13)$$

$$D + L + W + \frac{S}{2} \qquad (12\text{-}14)$$

$$D + L + S + \frac{W}{2} \qquad (12\text{-}15)$$

$$D + L + S + \frac{E}{1.4} \qquad (12\text{-}16)$$

$$0.9D \pm \frac{E}{1.4} \qquad (12\text{-}16\text{-}1)$$

EXCEPTIONS: 1. Crane hook loads need not be combined with roof live load or with more than three fourths of the snow load or one half of the wind load.

2. Design snow loads of 30 psf (1.44 kN/m^2) or less need not be combined with seismic loads. Where design snow loads exceed 30 psf (1.44 kN/m^2), the design snow load shall be included with seismic loads, but may be reduced up to 75 percent where consideration of siting, configuration and load duration warrant when approved by the building official.

1612.3.2.2 [For BSC, HCD 1 & HCD 2] In lieu of the basic load combinations specified in Section 1612.3.1, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following load combinations. When using these alternate basic load combinations, a one-third increase shall be permitted in allowable stresses for all combinations including W or E *but not concurrent with the duration of load increase permitted in Division III of Chapter 23.*

$$\mathrm{D + L + (L_r \text{ or } S)} \qquad (12\text{-}12)$$

$$\mathrm{D + L + (W \text{ or } E/1.4)} \qquad (12\text{-}13)$$

$$\mathrm{D + L + W + S/2} \qquad (12\text{-}14)$$

$$\mathrm{D + L + S + W/2} \qquad (12\text{-}15)$$

$$\mathrm{D + L + S + E/1.4} \qquad (12\text{-}16)$$

$$\mathrm{0.9D \pm E/1.4} \qquad (12\text{-}16\text{-}1)$$

EXCEPTIONS: 1. Crane hook loads need not be combined with roof live load or with more than three fourths of the snow load or one half of the wind load.

2 Design snow loads of 30 psf (1.44 kN/m ^{2}) or less need not be combined with seismic loads. Where design snow loads exceed 30 psf (1.44 kN/m ^{2}), the design snow load shall be included with seismic loads, but may be reduced up to 75 percent where consideration of siting, configuration and load duration warrant when approved by the building official.

1612.3.3 Other loads. Where *F, H, P* or *T* are to be considered in design, each applicable load shall be added to the combinations specified in Sections 1612.3.1 and 1612.3.2. When using the alternate load combinations specified in Section 1612.3.2, a one-third increase shall be permitted in allowable stresses for all combinations including *W* or *E*.

1612.4 Special Seismic Load Combinations. For both Allowable Stress Design and Strength Design, the following special load combinations for seismic design shall be used as specifically required by Chapter 16, Division IV, or by Chapters 18 through 23:

$$1.2D + f_1L + 1.0E_m \qquad (12\text{-}17)$$

$$0.9D \pm 1.0E_m \qquad (12\text{-}18)$$

WHERE:

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

= 0.5 for other live loads.

SECTION 1613 — DEFLECTION

The deflection of any structural member shall not exceed the values set forth in Table 16-D, based on the factors set forth in Table 16-E. The deflection criteria representing the most restrictive condition shall apply. Deflection criteria for materials not specified shall be developed in a manner consistent with the provisions of this section. See Section 1611.7 for camber requirements. Span tables for light wood-frame construction as specified in Chapter 23, Division VII, shall conform to the design criteria contained therein. For concrete, see Section 1909.5.2.6; for aluminum, see Section 2003; for glazing framing, see Section 2404.2.

Division II—SNOW LOADS

SECTION 1614 — SNOW LOADS

Buildings and other structures and all portions thereof that are subject to snow loading shall be designed to resist the snow loads, as determined by the building official, in accordance with the load combinations set forth in Section 1612.2 or 1612.3.

Potential unbalanced accumulation of snow at valleys, parapets, roof structures and offsets in roofs of uneven configuration shall be considered.

Snow loads in excess of 20 psf (0.96 kN/m^2) may be reduced for each degree of pitch over 20 degrees by R_s as determined by the formula:

$$R_s = \frac{S}{40} - \frac{1}{2} \qquad (14\text{-}1)$$

For **SI:**

$$R_s = \frac{S}{40} - 0.024$$

WHERE:

R_s = snow load reduction in pounds per square foot (kN/m^2) per degree of pitch over 20 degrees.

S = total snow load in pounds per square foot (kN/m^2).

For alternate design procedure, where specifically adopted, see Appendix Chapter 16, Division I.

PI = plasticity index of soil determined in accordance with approved national standards.

R = numerical coefficient representative of the inherent overstrength and global ductility capacity of lateral-force-resisting systems, as set forth in Table 16-N *[for BSC, HCD 1 & HCD 2] Table 16-N.1* or 16-P.

r = a ratio used in determining ρ. See Section 1630.1.

S_A, S_B, S_C, S_D, S_E, S_F = soil profile types as set forth in Table 16-J.

T = elastic fundamental period of vibration, in seconds, of the structure in the direction under consideration.

V = the total design lateral force or shear at the base given by Formula (30-5), (30-6), (30-7) or (30-11).

V_x = the design story shear in Story x.

W = the total seismic dead load defined in Section 1630.1.1.

w_i, w_x = that portion of W located at or assigned to Level i or x, respectively.

W_p = the weight of an element or component.

w_{px} = the weight of the diaphragm and the element tributary thereto at Level x, including applicable portions of other loads defined in Section 1630.1.1.

Z = seismic zone factor as given in Table 16-I.

Δ_M = Maximum Inelastic Response Displacement, which is the total drift or total story drift that occurs when the structure is subjected to the Design Basis Ground Motion, including estimated elastic and inelastic contributions to the total deformation defined in Section 1630.9.

Δ_S = Design Level Response Displacement, which is the total drift or total story drift that occurs when the structure is subjected to the design seismic forces.

δ_i = horizontal displacement at Level i relative to the base due to applied lateral forces, f, for use in Formula (30-10).

ρ = Redundancy/Reliability Factor given by Formula (30-3).

Ω_o = Seismic Force Amplification Factor, which is required to account for structural overstrength and set forth in Table 16-N *[for BSC, HCD 1 & HCD 2] Table 16-N.1*.

SECTION 1629 — CRITERIA SELECTION

1629.1 Basis for Design. The procedures and the limitations for the design of structures shall be determined considering seismic zoning, site characteristics, occupancy, configuration, structural system and height in accordance with this section. Structures shall be designed with adequate strength to withstand the lateral displacements induced by the Design Basis Ground Motion, considering the inelastic response of the structure and the inherent redundancy, overstrength and ductility of the lateral-force-resisting system. The minimum design strength shall be based on the Design Seismic Forces determined in accordance with the static lateral force procedure of Section 1630, except as modified by Section 1631.5.4. Where strength design is used, the load combinations of Section 1612.2 shall apply. Where Allowable Stress Design is used, the load combinations of Section 1612.3 shall apply. Allowable Stress Design may be used to evaluate sliding or overturning at the soil-structure interface regardless of the design approach used in the design of the structure, provided load combinations of Section 1612.3 are utilized. One- and two-family dwellings in Seismic Zone 1 need not conform to the provisions of this section.

1629.2 Occupancy Categories. For purposes of earthquake-resistant design, each structure shall be placed in one of the occupancy categories listed in Table 16-K. Table 16-K assigns importance factors, I and I_p, and structural observation requirements for each category.

1629.3 Site Geology and Soil Characteristics. Each site shall be assigned a soil profile type based on properly substantiated geotechnical data using the site categorization procedure set forth in Division V, Section 1636 and Table 16-J.

> **EXCEPTION:** When the soil properties are not known in sufficient detail to determine the soil profile type, Type S_D shall be used. Soil Profile Type S_E or S_F need not be assumed unless the building official determines that Type S_E or S_F may be present at the site or in the event that Type S_E or S_F is established by geotechnical data.

1629.3.1 Soil profile type. Soil Profile Types S_A, S_B, S_C, S_D and S_E are defined in Table 16-J and Soil Profile Type S_F is defined as soils requiring site-specific evaluation as follows:

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

2. Peats and/or highly organic clays, where the thickness of peat or highly organic clay exceeds 10 feet (3048 mm).

3. Very high plasticity clays with a plasticity index, $PI > 75$, where the depth of clay exceeds 25 feet (7620 mm).

4. Very thick soft/medium stiff clays, where the depth of clay exceeds 120 feet (36 576 mm).

1629.4 Site Seismic Hazard Characteristics. Seismic hazard characteristics for the site shall be established based on the seismic zone and proximity of the site to active seismic sources, site soil profile characteristics and the structure's importance factor.

1629.4.1 Seismic zone. Each site shall be assigned a seismic zone in accordance with Figure 16-2. Each structure shall be assigned a seismic zone factor Z, in accordance with Table 16-I.

1629.4.2 Seismic Zone 4 near-source factor.

1629.4.2.1 In Seismic Zone 4, each site shall be assigned a near-source factor in accordance with Table 16-S and the Seismic Source Type set forth in Table 16-U. The value of N_a used to determine C_a need not exceed 1.1 for structures complying with all the following conditions:

1. The soil profile type is S_A, S_B, S_C or S_D.

2. $\rho = 1.0$.

3. Except in single-story structures, Group R, Division 3 and Group U, Division 1 Occupancies, moment frame systems designated as part of the lateral-force-resisting system shall be special moment-resisting frames.

4. The exceptions to Section 2213.7.5 shall not apply, except for columns in one-story buildings or columns at the top story of multistory buildings.

5. None of the following structural irregularities is present: Type 1, 4 or 5 of Table 16-L, and Type 1 or 4 of Table 16-M.

1629.4.2.2 [For BSC, HCD 1 & HCD 2] In Seismic Zone 4, each site shall be assigned a near-source factor in accordance with Table 16-S and the Seismic Source Type set forth in Table 16-U. The value of N_a used in determining C_a need not exceed 1.1 for structures complying with all the following conditions:

1. The soil profile type is S_A, S_B, S_C or S_D.
2. $\rho = 1.0$.
3. Except in single-story structures, Group R, Division 3 and Group U, Division 1 Occupancies, moment frame systems designated as part of the lateral-force-resisting system shall be special moment-resisting frames.
4. *The provisions in Sections 9.6a and 9.6b of AISC - Seismic Part 1 shall not apply, except for columns in one-story buildings or columns at the top story of multistory buildings.*
5. None of the following structural irregularities is present: Type 1, 4 or 5 of Table 16-L, and Type 1 or 4 of Table 16-M.

1629.4.3 Seismic response coefficients. Each structure shall be assigned a seismic coefficient, C_a, in accordance with Table 16-Q and a seismic coefficient, C_v, in accordance with Table 16-R.

1629.5 Configuration Requirements.

1629.5.1 General. Each structure shall be designated as being structurally regular or irregular in accordance with Sections 1629.5.2 and 1629.5.3.

1629.5.2 Regular structures. Regular structures have no significant physical discontinuities in plan or vertical configuration or in their lateral-force-resisting systems such as the irregular features described in Section 1629.5.3.

1629.5.3 Irregular structures.

1. Irregular structures have significant physical discontinuities in configuration or in their lateral-force-resisting systems. Irregular features include, but are not limited to, those described in Tables 16-L and 16-M. All structures in Seismic Zone 1 and Occupancy Categories 4 and 5 in Seismic Zone 2 need to be evaluated only for vertical irregularities of Type 5 (Table 16-L) and horizontal irregularities of Type 1 (Table 16-M).

2. Structures having any of the features listed in Table 16-L shall be designated as if having a vertical irregularity.

> **EXCEPTION:** Where no story drift ratio under design lateral forces is greater than 1.3 times the story drift ratio of the story above, the structure may be deemed to not have the structural irregularities of Type 1 or 2 in Table 16-L. The story drift ratio for the top two stories need not be considered. The story drifts for this determination may be calculated neglecting torsional effects.

3. Structures having any of the features listed in Table 16-M shall be designated as having a plan irregularity.

1629.6 Structural Systems.

1629.6.1 General. Structural systems shall be classified as one of the types listed in Table 16-N *[for BSC, HCD 1 & HCD 2] Table 16-N.1* and defined in this section.

1629.6.2 Bearing wall system. A structural system without a complete vertical load-carrying space frame. Bearing walls or bracing systems provide support for all or most gravity loads. Resistance to lateral load is provided by shear walls or braced frames.

1629.6.3 Building frame system. A structural system with an essentially complete space frame providing support for gravity loads. Resistance to lateral load is provided by shear walls or braced frames.

1629.6.4 Moment-resisting frame system. A structural system with an essentially complete space frame providing support for gravity loads. Moment-resisting frames provide resistance to lateral load primarily by flexural action of members.

1629.6.5 Dual system. A structural system with the following features:

1. An essentially complete space frame that provides support for gravity loads.

2. Resistance to lateral load is provided by shear walls or braced frames and moment-resisting frames (SMRF, IMRF, MMRWF or steel OMRF). The moment-resisting frames shall be designed to independently resist at least 25 percent of the design base shear.

3. The two systems shall be designed to resist the total design base shear in proportion to their relative rigidities considering the interaction of the dual system at all levels.

1629.6.6 Cantilevered column system. A structural system relying on cantilevered column elements for lateral resistance.

1629.6.7 Undefined structural system. A structural system not listed in Table 16-N *[for BSC, HCD 1 & HCD 2] Table 16-N.1*.

1629.6.8 Nonbuilding structural system. A structural system conforming to Section 1634.

1629.7 Height Limits. Height limits for the various structural systems in Seismic Zones 3 and 4 are given in Table 16-N *[for BSC, HCD 1 & HCD 2] Table 16-N.1*.

> **EXCEPTION:** Regular structures may exceed these limits by not more than 50 percent for unoccupied structures, which are not accessible to the general public.

1629.8 Selection of Lateral-force Procedure.

1629.8.1 General. Any structure may be, and certain structures defined below shall be, designed using the dynamic lateral-force procedures of Section 1631.

1629.8.2 Simplified static. The simplified static lateral-force procedure set forth in Section 1630.2.3 may be used for the following structures of Occupancy Category 4 or 5:

1. Buildings of any occupancy (including single-family dwellings) not more than three stories in height excluding basements, that use light-frame construction.

2. Other buildings not more than two stories in height excluding basements.

1629.8.3 Static. The static lateral force procedure of Section 1630 may be used for the following structures:

1. All structures, regular or irregular, in Seismic Zone 1 and in Occupancy Categories 4 and 5 in Seismic Zone 2.

2. Regular structures under 240 feet (73 152 mm) in height with lateral force resistance provided by systems listed in Table 16-N *[for BSC, HCD 1 & HCD 2] Table 16-N.1*, except where Section 1629.8.4, Item 4, applies.

3. Irregular structures not more than five stories or 65 feet (19 812 mm) in height.

4. Structures having a flexible upper portion supported on a rigid lower portion where both portions of the structure considered separately can be classified as being regular, the average story stiffness of the lower portion is at least 10 times the average story stiffness of the upper portion and the period of the entire structure is not greater than 1.1 times the period of the upper portion considered as a separate structure fixed at the base.

1629.8.4 Dynamic. The dynamic lateral-force procedure of Section 1631 shall be used for all other structures, including the following:

1. Structures 240 feet (73 152 mm) or more in height, except as permitted by Section 1629.8.3, Item 1.

2. Structures having a stiffness, weight or geometric vertical irregularity of Type 1, 2 or 3, as defined in Table 16-L, or structures having irregular features not described in Table 16-L or 16-M, except as permitted by Section 1630.4.2.

3. Structures over five stories or 65 feet (19 812 mm) in height in Seismic Zones 3 and 4 not having the same structural system throughout their height except as permitted by Section 1630.4.2.

4. Structures, regular or irregular, located on Soil Profile Type S_F, that have a period greater than 0.7 second. The analysis shall include the effects of the soils at the site and shall conform to Section 1631.2, Item 4.

1629.9 System Limitations.

1629.9.1 Discontinuity. Structures with a discontinuity in capacity, vertical irregularity Type 5 as defined in Table 16-L, shall not be over two stories or 30 feet (9144 mm) in height where the weak story has a calculated strength of less than 65 percent of the story above.

> **EXCEPTION:** Where the weak story is capable of resisting a total lateral seismic force of Ω_o times the design force prescribed in Section 1630.

1629.9.2 Undefined structural systems. For undefined structural systems not listed in Table 16-N *[for BSC, HCD 1 & HCD 2] Table 16-N.1*, the coefficient R shall be substantiated by approved cyclic test data and analyses. The following items shall be addressed when establishing R:

1. Dynamic response characteristics,
2. Lateral force resistance,
3. Overstrength and strain hardening or softening,
4. Strength and stiffness degradation,
5. Energy dissipation characteristics,
6. System ductility, and
7. Redundancy.

1629.9.3 Irregular features. All structures having irregular features described in Table 16-L or 16-M shall be designed to meet the additional requirements of those sections referenced in the tables.

1629.10 Alternative Procedures.

1629.10.1 General. Alternative lateral-force procedures using rational analyses based on well-established principles of mechanics may be used in lieu of those prescribed in these provisions.

1629.10.2 Seismic isolation. Seismic isolation, energy dissipation and damping systems may be used in the design of structures when approved by the building official and when special detailing is used to provide results equivalent to those obtained by the use of conventional structural systems. For alternate design procedures on seismic isolation systems, refer to Appendix Chapter 16, Division III, Earthquake Regulations for Seismic-isolated Structures.

SECTION 1630 — MINIMUM DESIGN LATERAL FORCES AND RELATED EFFECTS

1630.1 Earthquake Loads and Modeling Requirements.

1630.1.1 Earthquake loads. Structures shall be designed for ground motion producing structural response and seismic forces in any horizontal direction. The following earthquake loads shall be used in the load combinations set forth in Section 1612:

$$E = \rho E_h + E_v \qquad (30\text{-}1)$$

$$E_m = \Omega_o E_h \qquad (30\text{-}2)$$

WHERE:

E = the earthquake load on an element of the structure resulting from the combination of the horizontal component, E_h, and the vertical component, E_v.

E_h = the earthquake load due to the base shear, V, as set forth in Section 1630.2 or the design lateral force, F_p, as set forth in Section 1632.

E_m = the estimated maximum earthquake force that can be developed in the structure as set forth in Section 1630.1.1.

E_v = the load effect resulting from the vertical component of the earthquake ground motion and is equal to an addition of $0.5C_aID$ to the dead load effect, D, for Strength Design, and may be taken as zero for Allowable Stress Design.

Ω_o = the seismic force amplification factor that is required to account for structural overstrength, as set forth in Section 1630.3.1.

ρ = Reliability/Redundancy Factor as given by the following formula:

$$\rho = 2 - \frac{20}{r_{max}\sqrt{A_B}} \qquad (30\text{-}3)$$

For **SI:**

$$\rho = 2 - \frac{6.1}{r_{max}\sqrt{A_B}}$$

WHERE:

r_{max} = the maximum element-story shear ratio. For a given direction of loading, the element-story shear ratio is the ratio of the design story shear in the most heavily loaded single element divided by the total design story shear. For any given Story Level i, the element-story shear ratio is denoted as r_i. The maximum element-story shear ratio r_{max} is defined as the largest of the element story shear ratios, r_i, which occurs in any of the story levels at or below the two-thirds height level of the building.

For braced frames, the value of r_i is equal to the maximum horizontal force component in a single brace element divided by the total story shear.

For moment frames, r_i shall be taken as the maximum of the sum of the shears in any two adjacent columns in a moment frame bay divided by the story shear. For columns common to two bays with moment-resisting connections on opposite sides at Level i in the direction under consideration, 70 percent of the shear in that column may be used in the column shear summation.

For shear walls, r_i shall be taken as the maximum value of the product of the wall shear multiplied by $10/l_w$ (For **SI:** $3.05/l_w$) and divided by the total story shear, where l_w is the length of the wall in feet (m).

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

ρ shall not be taken less than 1.0 and need not be greater than 1.5, and A_B is the ground floor area of the structure in square feet (m^2). For special moment-resisting frames, except when used in dual systems, ρ shall not exceed 1.25. The number of bays of special moment-resisting frames shall be increased to reduce r, such that ρ is less than or equal to 1.25.

EXCEPTION: A_B may be taken as the average floor area in the upper setback portion of the building where a larger base area exists at the ground floor.

When calculating drift, or when the structure is located in Seismic Zone 0, 1 or 2, ρ shall be taken equal to 1.

The ground motion producing lateral response and design seismic forces may be assumed to act nonconcurrently in the direction of each principal axis of the structure, except as required by Section 1633.1.

Seismic dead load, *W,* is the total dead load and applicable portions of other loads listed below.

1. In storage and warehouse occupancies, a minimum of 25 percent of the floor live load shall be applicable.

2. Where a partition load is used in the floor design, a load of not less than 10 psf (0.48 kN/m^2) shall be included.

3. Design snow loads of 30 psf (1.44 kN/m^2) or less need not be included. Where design snow loads exceed 30 psf (1.44 kN/m^2), the design snow load shall be included, but may be reduced up to 75 percent where consideration of siting, configuration and load duration warrant when approved by the building official.

4. Total weight of permanent equipment shall be included.

1630.1.2 Modeling requirements. The mathematical model of the physical structure shall include all elements of the lateral-force-resisting system. The model shall also include the stiffness and strength of elements, which are significant to the distribution of forces, 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.

2. For steel moment frame systems, the contribution of panel zone deformations to overall story drift shall be included.

1630.1.3 *P*Δ effects. The resulting member forces and moments and the story drifts induced by $P\Delta$ effects shall be considered in the evaluation of overall structural frame stability and shall be evaluated using the forces producing the displacements of Δ_S. $P\Delta$ need not be considered when the ratio of secondary moment to primary moment does not exceed 0.10; the ratio may be evaluated for any story as the product of the total dead, floor live and snow load, as required in Section 1612, above the story times the seismic drift in that story divided by the product of the seismic shear in that story times the height of that story. In Seismic Zones 3 and 4, $P\Delta$ need not be considered when the story drift ratio does not exceed $0.02/R$.

1630.2 Static Force Procedure.

1630.2.1 Design base shear. The total design base shear in a given direction shall be determined from the following formula:

$$V = \frac{C_v\ I}{R\ T}\ W \qquad (30\text{-}4)$$

The total design base shear need not exceed the following:

$$V = \frac{2.5\ C_a\ I}{R}\ W \qquad (30\text{-}5)$$

The total design base shear shall not be less than the following:

$$V = 0.11\ C_a\ I\ W \qquad (30\text{-}6)$$

In addition, for Seismic Zone 4, the total base shear shall also not be less than the following:

$$V = \frac{0.8\ ZN_v\ I}{R}\ W \qquad (30\text{-}7)$$

1630.2.2 Structure period. The value of *T* shall be determined from one of the following methods:

1. **Method A:** For all buildings, the value *T* may be approximated from the following formula:

$$T = C_t\,(h_n)^{3/4} \qquad (30\text{-}8)$$

WHERE:

C_t = 0.035 (0.0853) for steel moment-resisting frames.

C_t = 0.030 (0.0731) for reinforced concrete moment-resisting frames and eccentrically braced frames.

C_t = 0.020 (0.0488) for all other buildings.

Alternatively, the value of C_t for structures with concrete or masonry shear walls may be taken as $0.1/\sqrt{A_c}$ (For **SI:** $0.0743/\sqrt{A_c}$ for A_c in m^2).

The value of A_c shall be determined from the following formula:

$$A_c = \Sigma A_e\left[0.2 + (D_e/h_n)^2\right] \qquad (30\text{-}9)$$

The value of D_e/h_n used in Formula (30-9) shall not exceed 0.9.

2. **Method B:** The fundamental period *T* may be calculated using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The analysis shall be in accordance with the requirements of Section 1630.1.2. The value of *T* from Method B shall not exceed a value 30 percent greater than the value of *T* obtained from Method A in Seismic Zone 4, and 40 percent in Seismic Zones 1, 2 and 3.

The fundamental period *T* may be computed by using the following formula:

$$T = 2\pi\sqrt{\left(\sum_{i=1}^{n} w_i\,\delta_i^{\ 2}\right) \div \left(g\sum_{i=1}^{n} f_i\,\delta_i\right)} \qquad (30\text{-}10)$$

The values of f_i represent any lateral force distributed approximately in accordance with the principles of Formulas (30-13), (30-14) and (30-15) or any other rational distribution. The elastic deflections, δ_i, shall be calculated using the applied lateral forces, f_i.

1630.2.3 Simplified design base shear.

1630.2.3.1 General. Structures conforming to the requirements of Section 1629.8.2 may be designed using this procedure.

1630.2.3.2 Base shear. The total design base shear in a given direction shall be determined from the following formula:

$$V = \frac{3.0\ C_a}{R}\ W \qquad (30\text{-}11)$$

where the value of C_a shall be based on Table 16-Q for the soil profile type. When the soil properties are not known in sufficient detail to determine the soil profile type, Type S_D shall be used in Seismic Zones 3 and 4, and Type S_E shall be used in Seismic Zones 1, 2A and 2B. In Seismic Zone 4, the Near-Source Factor, N_a, need not be greater than 1.3 if none of the following structural irregularities are present: Type 1, 4 or 5 of Table 16-L, or Type 1 or 4 of Table 16-M.

1630.2.3.3 ***[For BSC, HCD 1 & HCD 2] Distribution.***

1630.2.3.3.1 **Vertical distribution.** The forces at each level shall be calculated using the following formula:

$$F_x = \frac{3.0\ C_a}{R}\ w_i \qquad (30\text{-}12)$$

where the value of C_a shall be determined in Section 1630.2.3.2.

1630.2.3.3.2 [For BSC, HCD 1 & HCD 2] Horizontal Distribution. *Diaphragms constructed of untopped steel decking or wood structural panels or similar light-frame construction are permitted to be considered as flexible.*

1630.2.3.4 Applicability. Sections 1630.1.2, 1630.1.3, 1630.2.1, 1630.2.2, 1630.5, 1630.9, 1630.10 and 1631 shall not apply when using the simplified procedure.

EXCEPTION: For buildings with relatively flexible structural systems, the building official may require consideration of $P\Delta$ effects and drift in accordance with Sections 1630.1.3, 1630.9 and 1630.10. Δ_s shall be prepared using design seismic forces from Section 1630.2.3.2.

Where used, Δ_M shall be taken equal to 0.01 times the story height of all stories. In Section 1633.2.9, Formula (33-1) shall read $F_{px} = \frac{3.0\ C_a}{R}\ w_{px}$ and need not exceed $1.0\,C_a\,w_{px}$, but shall not be less than $0.5\,C_a\,w_{px}$. R and Ω_o shall be taken from Table 16-N *[for BSC, HCD 1 & HCD 2] Table 16-N.1.*

1630.3 Determination of Seismic Factors.

1630.3.1 Determination of Ω_o. For specific elements of the structure, as specifically identified in this code, the minimum design strength shall be the product of the seismic force overstrength factor Ω_o and the design seismic forces set forth in Section 1630. For both Allowable Stress Design and Strength Design, the Seismic Force Overstrength Factor, Ω_o, shall be taken from Table 16-N *[for BSC, HCD 1 & HCD 2] Table 16-N.1.*

1630.3.2 Determination of *R*. The notation *R* shall be taken from Table 16-N *[for BSC, HCD 1 & HCD 2] Table 16-N.1.*

1630.4 Combinations of Structural Systems.

1630.4.1 General. Where combinations of structural systems are incorporated into the same structure, the requirements of this section shall be satisfied.

1630.4.2 Vertical combinations.

1630.4.2.1 The value of *R* used in the design of any story shall be less than or equal to the value of *R* used in the given direction for the story above.

EXCEPTION: This requirement need not be applied to a story where the dead weight above that story is less than 10 percent of the total dead weight of the structure.

Structures may be designed using the procedures of this section under the following conditions:

1. The entire structure is designed using the lowest *R* of the lateral-force-resisting systems used, or

2. The following two-stage static analysis procedures may be used for structures conforming to Section 1629.8.3, Item 4.

   2.1 The flexible upper portion shall be designed as a separate structure, supported laterally by the rigid lower portion, using the appropriate values of *R* and ρ.

   2.2 The rigid lower portion shall be designed as a separate structure using the appropriate values of *R* and ρ. The reactions from the upper portion shall be those determined from the analysis of the upper portion amplified by the ratio of the *(R*/ρ) of the upper portion over *(R*/ρ) of the lower portion.

1630.4.2.2 [For BSC, HCD 1 & HCD 2] The value of R used in the design of any story shall be less than or equal to the value of R used in the given direction for the story above.

EXCEPTION: This requirement need not be applied to a story where the dead weight above that story is less than 10 percent of the total dead weight of the structure.

Structures may be designed using the procedures of this section under the following conditions:

1. The entire structure is designed using the lowest R of the lateral-force-resisting systems used, or

2. The following two-stage static analysis procedures may be used for structures conforming to Section 1629.8.3, Item 4.

   2.1 The flexible upper portion shall be designed as a separate structure, supported laterally by the rigid lower portion, using the appropriate values of R and ρ.

   2.2 The rigid lower portion shall be designed as a separate structure using the appropriate values of R and ρ. The reactions from the upper portion shall be those determined from the analysis of the upper portion multiplied by the ratio of the (R/ρ) of the upper portion over (R/ρ) of the lower portion. *This ratio shall not be taken less than 1.0.*

1630.4.3 Combinations along different axes. In Seismic Zones 3 and 4 where a structure has a bearing wall system in only one direction, the value of *R* used for design in the orthogonal direction shall not be greater than that used for the bearing wall system.

Any combination of bearing wall systems, building frame systems, dual systems or moment-resisting frame systems may be used to resist seismic forces in structures less than 160 feet (48 768 mm) in height. Only combinations of dual systems and special moment-resisting frames shall be used to resist seismic forces in structures exceeding 160 feet (48 768 mm) in height in Seismic Zones 3 and 4.

1630.4.4 Combinations along the same axis. For other than dual systems and shear wall-frame interactive systems in Seismic Zones 0 and 1, where a combination of different structural systems is utilized to resist lateral forces in the same direction, the value of *R* used for design in that direction shall not be greater than the least value for any of the systems utilized in that same direction.

1630.5 Vertical Distribution of Force. The total force shall be distributed over the height of the structure in conformance with Formulas (30-13), (30-14) and (30-15) in the absence of a more rigorous procedure.

$$V = F_t + \sum_{i=1}^{n} F_i \qquad (30\text{-}13)$$

The concentrated force F_t at the top, which is in addition to F_n, shall be determined from the formula:

$$F_t = 0.07\ T\ V \qquad (30\text{-}14)$$

The value of *T* used for the purpose of calculating F_t shall be the period that corresponds with the design base shear as computed using Formula (30-4). F_t need not exceed $0.25V$ and may be considered as zero where *T* is 0.7 second or less. The remaining portion of the base shear shall be distributed over the height of the structure, including Level *n,* according to the following formula:

$$F_x = \frac{(V - F_t)\, w_x\, h_x}{\sum_{i=1}^{n} w_i\, h_i} \qquad (30\text{-}15)$$

At each level designated as *x,* the force F_x shall be applied over the area of the building in accordance with the mass distribution at that level. Structural displacements and design seismic forces shall be calculated as the effect of forces F_x and F_t applied at the appropriate levels above the base.

1630.6 Horizontal Distribution of Shear. The design story shear, V_x, in any story is the sum of the forces F_t and F_x above that story. V_x shall be distributed to the various elements of the vertical lateral-force-resisting system in proportion to their rigidities, considering the rigidity of the diaphragm. See Section 1633.2.4 for rigid elements that are not intended to be part of the lateral-force-resisting systems.

Where diaphragms are not flexible, the mass at each level shall be assumed to be displaced from the calculated center of mass in each direction a distance equal to 5 percent of the building dimension at that level perpendicular to the direction of the force under consideration. The effect of this displacement on the story shear distribution shall be considered.

Diaphragms shall be considered flexible for the purposes of distribution of story shear and torsional moment when the maximum lateral deformation of the diaphragm is more than two times the average story drift of the associated story. This may be determined by comparing the computed midpoint in-plane deflection of the diaphragm itself under lateral load with the story drift of adjoining vertical-resisting elements under equivalent tributary lateral load.

1630.7 Horizontal Torsional Moments. Provisions shall be made for the increased shears resulting from horizontal torsion where diaphragms are not flexible. The most severe load combination for each element shall be considered for design.

The torsional design moment at a given story shall be the moment resulting from eccentricities between applied design lateral forces at levels above that story and the vertical-resisting elements in that story plus an accidental torsion.

The accidental torsional moment shall be determined by assuming the mass is displaced as required by Section 1630.6.

Where torsional irregularity exists, as defined in Table 16-M, the effects shall be accounted for by increasing the accidental torsion at each level by an amplification factor, A_x, determined from the following formula:

$$A_x = \left[\frac{\delta_{max}}{1.2\,\delta_{avg}}\right]^2 \qquad (30\text{-}16)$$

WHERE:

δ_{avg} = the average of the displacements at the extreme points of the structure at Level *x.*

δ_{max} = the maximum displacement at Level *x.*

The value of A_x need not exceed 3.0.

1630.8 Overturning.

1630.8.1 General. Every structure shall be designed to resist the overturning effects caused by earthquake forces specified in Section 1630.5. At any level, the overturning moments to be resisted shall be determined using those seismic forces (F_t and F_x) that act on levels above the level under consideration. At any level, the incremental changes of the design overturning moment shall be distributed to the various resisting elements in the manner prescribed in Section 1630.6. Overturning effects on every element shall be carried down to the foundation. See Sections 1612 and 1633 for combining gravity and seismic forces.

1630.8.2 Elements supporting discontinuous systems.

1630.8.2.1 General.

1630.8.2.1.1 Where any portion of the lateral-load-resisting system is discontinuous, such as for vertical irregularity Type 4 in Table 16-L or plan irregularity Type 4 in Table 16-M, concrete, masonry, steel and wood elements supporting such discontinuous systems shall have the design strength to resist the combination loads resulting from the special seismic load combinations of Section 1612.4. CA

> **EXCEPTIONS:** 1. The quantity E_m in Section 1612.4 need not exceed the maximum force that can be transferred to the element by the lateral-force-resisting system.
>
> 2. Concrete slabs supporting light-frame wood shear wall systems or light-frame steel and wood structural panel shear wall systems.

For Allowable Stress Design, the design strength may be determined using an allowable stress increase of 1.7 and a resistance factor, ϕ, of 1.0. This increase shall not be combined with the one-third stress increase permitted by Section 1612.3, but may be combined with the duration of load increase permitted in Chapter 23, Division III.

1630.8.2.1.2 [For BSC, HCD 1 & HCD 2] Where any portion of the lateral-load-resisting system is discontinuous, such as for vertical irregularity Type 4 in Table 16-L or plan irregularity Type 4 in Table 16-M, concrete, masonry, steel and wood elements *(i.e. columns, beams, trusses or slabs)* supporting such discontinuous systems shall have the design strength to resist the combination loads resulting from the special seismic load combinations of Section 1612.4. *The connections of such discontinued elements to the supporting members shall be adequate to transmit the forces for which the discontinuous elements were required to be designed.* CA CA CAC

> **EXCEPTIONS:** 1. The quantity E_m in Section 1612.4 need not exceed the maximum force that can be transferred to the element by the lateral-force-resisting system.
>
> 2. Concrete slabs supporting light-frame wood shear wall systems or light-frame steel and wood structural panel shear wall systems.

For Allowable Stress Design, the design strength may be determined using an allowable stress increase of 1.7 and a resistance factor, Φ, of 1.0. This increase shall not be combined with the one-third stress increase permitted by Section 1612.3, but may be combined with the duration of load increase permitted in Chapter 23, Division III.

1630.8.2.2 Detailing requirements in Seismic Zones 3 and 4.

1630.8.2.2.1 In Seismic Zones 3 and 4, elements supporting discontinuous systems shall meet the following detailing or member limitations: CA

1. Reinforced concrete elements designed primarily as axial-load members shall comply with Section 1921.4.4.5.

2. Reinforced concrete elements designed primarily as flexural members and supporting other than light-frame wood shear wall systems or light-frame steel and wood structural panel shear wall systems shall comply with Sections 1921.3.2 and 1921.3.3. Strength computations for portions of slabs designed as supporting elements shall include only those portions of the slab that comply with the requirements of these sections.

3. Masonry elements designed primarily as axial-load carrying members shall comply with Sections 2106.1.12.4, Item 1, and 2108.2.6.2.6.

4. Masonry elements designed primarily as flexural members shall comply with Section 2108.2.6.2.5.

5. Steel elements designed primarily as axial-load members shall comply with Sections 2213.5.2 and 2213.5.3.

6. Steel elements designed primarily as flexural members or trusses shall have bracing for both top and bottom beam flanges or chords at the location of the support of the discontinuous system and shall comply with the requirements of Section 2213.7.1.3.

7. Wood elements designed primarily as flexural members shall be provided with lateral bracing or solid blocking at each end of the element and at the connection location(s) of the discontinuous system.

1630.8.2.2.2 [For BSC, HCD 1 & HCD 2] In Seismic Zones 3 and 4, elements supporting discontinuous systems shall meet the following detailing or member limitations:

1. Reinforced concrete or reinforced masonry elements designed primarily as axial-load members shall comply with Section 1921.4.4.5.

2. Reinforced concrete elements designed primarily as flexural members and supporting other than light-frame wood shear wall systems or light-frame steel and wood structural panel shear wall systems shall comply with Sections 1921.3.2 and 1921.3.3. Strength computations for portions of slabs designed as supporting elements shall include only those portions of the slab that comply with the requirements of these sections.

3. Masonry elements designed primarily as axial-load carrying members shall comply with Sections 2106.1.12.4, Item 1, and 2108.2.6.2.6.

4. Masonry elements designed primarily as flexural members shall comply with Section 2108.2.6.2.5.

5. * * *

6. Steel elements designed primarily as flexural members or trusses shall have bracing for both top and bottom beam flanges or chords at the location of the support of the discontinuous system and shall comply with the requirements of *AISC-Seismic Part I, Section 9.4b*.

7. Wood elements designed primarily as flexural members shall be provided with lateral bracing or solid blocking at each end of the element and at the connection location(s) of the discontinuous systems.

1630.8.3 At foundation. See Sections 1629.1 and 1809.4 for overturning moments to be resisted at the foundation soil interface.

1630.9 Drift. Drift or horizontal displacements of the structure shall be computed where required by this code. For both Allowable Stress Design and Strength Design, the Maximum Inelastic Response Displacement, Δ_M, of the structure caused by the Design Basis Ground Motion shall be determined in accordance with this section. The drifts corresponding to the design seismic forces of Section 1630.2.1, Δ_S, shall be determined in accordance with Section 1630.9.1. To determine Δ_M, these drifts shall be amplified in accordance with Section 1630.9.2.

1630.9.1 Determination of Δ_S. A static, elastic analysis of the lateral force-resisting system shall be prepared using the design seismic forces from Section 1630.2.1. Alternatively, dynamic analysis may be performed in accordance with Section 1631. Where Allowable Stress Design is used and where drift is being computed, the load combinations of Section 1612.2 shall be used. The mathematical model shall comply with Section 1630.1.2. The resulting deformations, denoted as Δ_S, shall be determined at all critical locations in the structure. Calculated drift shall include translational and torsional deflections.

1630.9.2 Determination of Δ_M. The Maximum Inelastic Response Displacement, Δ_M, shall be computed as follows:

$$\Delta_M = 0.7\,R\Delta_S \qquad (30\text{-}17)$$

EXCEPTION: Alternatively, Δ_M may be computed by nonlinear time history analysis in accordance with Section 1631.6.

The analysis used to determine the Maximum Inelastic Response Displacement Δ_M shall consider $P\Delta$ effects.

1630.10 Story Drift Limitation.

1630.10.1 General. Story drifts shall be computed using the Maximum Inelastic Response Displacement, Δ_M.

1630.10.2 Calculated. Calculated story drift using Δ_M shall not exceed 0.025 times the story height for structures having a fundamental period of less than 0.7 second. For structures having a fundamental period of 0.7 second or greater, the calculated story drift shall not exceed 0.020 times the story height.

EXCEPTIONS: 1. These drift limits may be exceeded when it is demonstrated that greater drift can be tolerated by both structural elements and nonstructural elements that could affect life safety. The drift used in this assessment shall be based upon the Maximum Inelastic Response Displacement, Δ_M.

2. There shall be no drift limit in single-story steel-framed structures classified as Groups B, F and S Occupancies or Group H, Division 4 or 5 Occupancies. In Groups B, F and S Occupancies, the primary use shall be limited to storage, factories or workshops. Minor accessory uses shall be allowed in accordance with the provisions of Section 302. Structures on which this exception is used shall not have equipment attached to the structural frame or shall have such equipment detailed to accommodate the additional drift. Walls that are laterally supported by the steel frame shall be designed to accommodate the drift in accordance with Section 1633.2.4.

1630.10.3 Limitations. The design lateral forces used to determine the calculated drift may disregard the limitations of Formulas (30-6) and may be based on the period determined from Formula (30-10) neglecting the 30 or 40 percent limitations of Section 1630.2.2, Item 2.

1630.11 Vertical Component. The following requirements apply in Seismic Zones 3 and 4 only. Horizontal cantilever components shall be designed for a net upward force of $0.7C_aIW_p$.

In addition to all other applicable load combinations, horizontal prestressed components shall be designed using not more than 50 percent of the dead load for the gravity load, alone or in combination with the lateral force effects.

SECTION 1631 — DYNAMIC ANALYSIS PROCEDURES

1631.1 General. Dynamic analyses procedures, when used, shall conform to the criteria established in this section. The analysis shall be based on an appropriate ground motion representation and shall be performed using accepted principles of dynamics.

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anchors or shallow cast-in-place anchors. Shallow anchors are those with an embedment length-to-diameter ratio of less than 8. When anchorage is constructed of nonductile materials, or by use of adhesive, R_p shall equal 1.0.

The design lateral forces determined using Formula (32-1) or (32-2) shall be distributed in proportion to the mass distribution of the element or component.

Forces determined using Formula (32-1) or (32-2) shall be used to design members and connections that transfer these forces to the seismic-resisting systems. Members and connection design shall use the load combinations and factors specified in Section 1612.2 or 1612.3. The Reliability/Redundancy Factor, ρ, may be taken equal to 1.0.

For applicable forces and Component Response Modification Factors in connectors for exterior panels and diaphragms, refer to Sections 1633.2.4, 1633.2.8 and 1633.2.9.

Forces shall be applied in the horizontal directions, which result in the most critical loadings for design.

[For OSHPD 2] The design and detailing of anchorages or restraints for architectural components and mechanical and electrical systems shall accommodate the structure drifts as computed in accordance with Division III—Earthquake Design, Section 1630.9.2 or determined by the wind forces given in Division II—Wind Design multiplied by 2.

1632.3 Specifying Lateral Forces. Design specifications for equipment shall either specify the design lateral forces prescribed herein or reference these provisions.

1632.4 Relative Motion of Equipment Attachments. For equipment in Categories 1 and 2 buildings as defined in Table 16-K, the lateral-force design shall consider the effects of relative motion of the points of attachment to the structure, using the drift based upon Δ_M.

1632.5 Alternative Designs. Where an approved national standard or approved physical test data provide a basis for the earthquake-resistant design of a particular type of equipment or other nonstructural component, such a standard or data may be accepted as a basis for design of the items with the following limitations:

1. These provisions shall provide minimum values for the design of the anchorage and the members and connections that transfer the forces to the seismic-resisting system.
2. The force, F_p, and the overturning moment used in the design of the nonstructural component shall not be less than 80 percent of the values that would be obtained using these provisions.

SECTION 1633 — DETAILED SYSTEMS DESIGN REQUIREMENTS

1633.1 General. All structural framing systems shall comply with the requirements of Section 1629. Only the elements of the designated seismic-force-resisting system shall be used to resist design forces. The individual components shall be designed to resist the prescribed design seismic forces acting on them. The components shall also comply with the specific requirements for the material contained in Chapters 19 through 23. In addition, such framing systems and components shall comply with the detailed system design requirements contained in Section 1633.

All building components in Seismic Zones 2, 3 and 4 shall be designed to resist the effects of the seismic forces prescribed herein and the effects of gravity loadings from dead, floor live and snow loads.

Consideration shall be given to design for uplift effects caused by seismic loads.

In Seismic Zones 2, 3 and 4, provision shall be made for the effects of earthquake forces acting in a direction other than the principal axes in each of the following circumstances:

The structure has plan irregularity Type 5 as given in Table 16-M.

The structure has plan irregularity Type 1 as given in Table 16-M for both major axes.

A column of a structure forms part of two or more intersecting lateral-force-resisting systems.

> **EXCEPTION:** If the axial load in the column due to seismic forces acting in either direction is less than 20 percent of the column axial load capacity.

The requirement that orthogonal effects be considered may be satisfied by designing such elements for 100 percent of the prescribed design seismic forces in one direction plus 30 percent of the prescribed design seismic forces in the perpendicular direction. The combination requiring the greater component strength shall be used for design. Alternatively, the effects of the two orthogonal directions may be combined on a square root of the sum of the squares (SRSS) basis. When the SRSS method of combining directional effects is used, each term computed shall be assigned the sign that will result in the most conservative result.

1633.2 Structural Framing Systems.

1633.2.1 General. Four types of general building framing systems defined in Section 1629.6 are recognized in these provisions and shown in Table 16-N *[for BSC, HCD 1 & HCD2] Table 16-N.1*. Each type is subdivided by the types of vertical elements used to resist lateral seismic forces. Special framing requirements are given in this section and in Chapters 19 through 23.

1633.2.2 Detailing for combinations of systems. For components common to different structural systems, the more restrictive detailing requirements shall be used.

1633.2.3 Connections. Connections that resist design seismic forces shall be designed and detailed on the drawings.

1633.2.4 Deformation compatibility. All structural framing elements and their connections, not required by design to be part of the lateral-force-resisting system, shall be designed and/or detailed to be adequate to maintain support of design dead plus live loads when subjected to the expected deformations caused by seismic forces. $P\Delta$ effects on such elements shall be considered. Expected deformations shall be determined as the greater of the Maximum Inelastic Response Displacement, Δ_M, considering $P\Delta$ effects determined in accordance with Section 1630.9.2 or the deformation induced by a story drift of 0.0025 times the story height. When computing expected deformations, the stiffening effect of those elements not part of the lateral-force-resisting system shall be neglected.

For elements not part of the lateral-force-resisting system, the forces induced by the expected deformation may be considered as ultimate or factored forces. When computing the forces induced by expected deformations, the restraining effect of adjoining rigid structures and nonstructural elements shall be considered and a rational value of member and restraint stiffness shall be used. Inelastic deformations of members and connections may be considered in the evaluation, provided the assumed calculated capacities are consistent with member and connection design and detailing.

For concrete and masonry elements that are part of the lateral-force-resisting system, the assumed flexural and shear stiffness properties shall not exceed one half of the gross section properties unless a rational cracked-section analysis is performed. Additional deformations that may result from foundation flexibility and diaphragm deflections shall be considered. For concrete elements not part of the lateral-force-resisting system, see Section 1921.7.

1633.2.4.1 Adjoining rigid elements. Moment-resisting frames and shear walls may be enclosed by or adjoined by more rigid elements, provided it can be shown that the participation or failure of the more rigid elements will not impair the vertical and lateral-load-resisting ability of the gravity load and lateral-force-resisting systems. The effects of adjoining rigid elements shall be considered when assessing whether a structure shall be designated regular or irregular in Section 1629.5.1.

1633.2.4.2 Exterior elements. Exterior nonbearing, nonshear wall panels or elements that are attached to or enclose the exterior shall be designed to resist the forces per Formula (32-1) or (32–2) and shall accommodate movements of the structure based on Δ_M and temperature changes. Such elements shall be supported by means of cast-in-place concrete or by mechanical connections and fasteners in accordance with the following provisions:

1. Connections and panel joints shall allow for a relative movement between stories of not less than two times story drift caused by wind, the calculated story drift based on Δ_M or $^1/_2$ inch (12.7 mm), whichever is greater.

2. Connections to permit movement in the plane of the panel for story drift shall be sliding connections using slotted or oversize holes, connections that permit movement by bending of steel, or other connections providing equivalent sliding and ductility capacity.

3. Bodies of connections shall have sufficient ductility and rotation capacity to preclude fracture of the concrete or brittle failures at or near welds.

4. The body of the connection shall be designed for the force determined by Formula (32-2), where R_p = 3.0 and a_p = 1.0.

5. All fasteners in the connecting system, such as bolts, inserts, welds and dowels, shall be designed for the forces determined by Formula (32-2), where R_p = 1.0 and a_p = 1.0.

6. Fasteners embedded in concrete shall be attached to, or hooked around, reinforcing steel or otherwise terminated to effectively transfer forces to the reinforcing steel.

1633.2.5 Ties and continuity. All parts of a structure shall be interconnected and the connections shall be capable of transmitting the seismic force induced by the parts being connected. As a minimum, any smaller portion of the building shall be tied to the remainder of the building with elements having at least a strength to resist 0.5 C_aI times the weight of the smaller portion.

A positive connection for resisting a horizontal force acting parallel to the member shall be provided for each beam, girder or truss. This force shall not be less than 0.5 C_aI times the dead plus live load.

1633.2.6 Collector elements.

|| CA ***1633.2.6.1*** 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 forces determined in accordance with Formula (33-1). In addition, collector elements, splices, and their connections to resisting elements shall have the design strength to resist the combined loads resulting from the special seismic load of Section 1612.4.

> **EXCEPTION:** In structures, or portions thereof, braced entirely by light-frame wood shear walls or light-frame steel and wood structural panel shear wall systems, collector elements, splices and connections to resisting elements need only be designed to resist forces in accordance with Formula (33-1).

The quantity E_M 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. For Allowable Stress Design, the design strength may be determined using an allowable stress increase of 1.7 and a resistance factor, ϕ, of 1.0. This increase shall not be combined with the one-third stress increase permitted by Section 1612.3, but may be combined with the duration of load increase permitted in Division III of Chapter 23.

1633.2.6.2 [For OSHPD 2] 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. CA ||

Collector elements, splices and their connections to resisting elements shall resist the forces determined in accordance with Formula (33-1). In addition, collector elements, splices, and their connections to resisting elements shall have the design strength to resist the combined loads resulting from the special seismic load of Section 1612.4.

> **EXCEPTION:** In structures, or portions thereof, braced entirely by light-frame wood shear walls or light-frame steel and wood structural panel shear wall systems, collector elements, splices and connections to resisting elements need only be designed to resist forces in accordance with Formula (33-1).

The quantity E_m 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. For Allowable Stress Design, the design strength may be determined using an allowable stress increase of 1.7 and *for LRFD* a resistance factor, ϕ, of 1.0. This increase shall not be combined with the one-third stress increase permitted by Section 1612.3, but may be combined with the duration of load increase permitted in Division III of Chapter 23. CA || CA ||

1633.2.7 Concrete frames. Concrete frames required by design to be part of the lateral-force-resisting system shall conform to the following:

1. In Seismic Zones 3 and 4 they shall be special moment-resisting frames.

2. In Seismic Zone 2 they shall, as a minimum, be intermediate moment-resisting frames.

1633.2.8 Anchorage of concrete or masonry walls. Concrete or masonry walls shall be anchored to all floors and roofs that provide out-of-plane lateral support of the wall. The anchorage shall provide a positive direct connection between the wall and floor or roof construction capable of resisting the larger of the horizontal forces specified in this section and Sections 1611.4 and 1632. In addition, in Seismic Zones 3 and 4, diaphragm to wall anchorage using embedded straps shall have the straps attached to or hooked around the reinforcing steel or otherwise terminated to effectively transfer forces to the reinforcing steel. Requirements for developing anchorage forces in diaphragms are given in Section 1633.2.9. Diaphragm deformation shall be considered in the design of the supported walls.

1633.2.8.1 Out-of-plane wall anchorage to flexible diaphragms. This section shall apply in Seismic Zones 3 and 4 where flexible diaphragms, as defined in Section 1630.6, provide lateral support for walls.

1. Elements of the wall anchorage system shall be designed for the forces specified in Section 1632 where R_p = 3.0 and a_p = 1.5.

In Seismic Zone 4, the value of F_p used for the design of the elements of the wall anchorage system shall not be less than 420 pounds per lineal foot (6.1 kN per lineal meter) of wall substituted for E.

See Section 1611.4 for minimum design forces in other seismic zones.

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

3. 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 be that specified in Section 1633.2.8.1, Item 1.

4. The strength design forces for steel elements of the wall anchorage system shall be 1.4 times the forces otherwise required by this section.

5. The strength design forces for wood elements of the wall anchorage system shall be 0.85 times the force otherwise required by this section and these wood elements shall have a minimum actual net thickness of $2^1/_2$ inches (63.5 mm).

1633.2.9 Diaphragms.

1. 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 element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

2. Floor and roof diaphragms shall be designed to resist the forces determined in accordance with the following formula:

$$F_{px} = \frac{F_t + \sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad (33\text{-}1)$$

The force F_{px} determined from Formula (33-1) need not exceed $1.0C_aIw_{px}$, but shall not be less than $0.5C_aIw_{px}$.

When the diaphragm is required to transfer design seismic forces from the vertical-resisting elements above the diaphragm to other vertical-resisting elements below the diaphragm due to offset in the placement of the elements or to changes in stiffness in the vertical elements, these forces shall be added to those determined from Formula (33-1).

3. Design seismic forces for flexible diaphragms providing lateral supports for walls or frames of masonry or concrete shall be determined using Formula (33-1) based on the load determined in accordance with Section 1630.2 using a R not exceeding 4.

4. Diaphragms supporting concrete or masonry walls shall have continuous ties or struts between diaphragm chords to distribute the anchorage forces specified in Section 1633.2.8. Added chords of subdiaphragms may be used to form subdiaphragms to transmit the anchorage forces to the main continuous crossties. The maximum length-to-width ratio of the wood structural subdiaphragm shall be $2^1/_2$:1.

5. Where wood diaphragms are used to laterally support concrete or masonry walls, the anchorage shall conform to Section 1633.2.8. In Seismic Zones 2, 3 and 4, anchorage shall not be accomplished by use of toenails or nails subject to withdrawal, wood ledgers or framing shall not be used in cross-grain bending or cross-grain tension, and the continuous ties required by Item 4 shall be in addition to the diaphragm sheathing.

6. Connections of diaphragms to the vertical elements in structures in Seismic Zones 3 and 4, having a plan irregularity of Type 1, 2, 3 or 4 in Table 16-M, shall be designed without considering either the one-third increase or the duration of load increase considered in allowable stresses for elements resisting earthquake forces.

7. In structures in Seismic Zones 3 and 4 having a plan irregularity of Type 2 in Table 16-M, diaphragm chords and drag members shall be designed considering independent movement of the projecting wings of the structure. Each of these diaphragm elements shall be designed for the more severe of the following two assumptions:

Motion of the projecting wings in the same direction.

Motion of the projecting wings in opposing directions.

EXCEPTION: This requirement may be deemed satisfied if the procedures of Section 1631 in conjunction with a three-dimensional model have been used to determine the lateral seismic forces for design.

1633.2.10 Framing below the base. The strength and stiffness of the framing between the base and the foundation shall not be less than that of the superstructure. The special detailing requirements of Chapters 19 and 22, as appropriate, shall apply to columns supporting discontinuous lateral-force-resisting elements and to SMRF, IMRF, EBF, STMF and MMRWF system elements below the base, which are required to transmit the forces resulting from lateral loads to the foundation.

1633.2.11 Building separations. All structures shall be separated from adjoining structures. Separations shall allow for the displacement Δ_M. Adjacent buildings on the same property shall be separated by at least Δ_{MT} where

$$\Delta_{MT} = \sqrt{(\Delta_{M1})^2 + (\Delta_{M2})^2} \qquad (33\text{-}2)$$

and Δ_{M1} and Δ_{M2} are the displacements of the adjacent buildings.

When a structure adjoins a property line not common to a public way, that structure shall also be set back from the property line by at least the displacement Δ_M of that structure.

EXCEPTION: Smaller separations or property line setbacks may be permitted when justified by rational analyses based on maximum expected ground motions.

SECTION 1634 — NONBUILDING STRUCTURES

1634.1 General.

1634.1.1 Scope. Nonbuilding structures include all self-supporting structures other than buildings that carry gravity loads and resist the effects of earthquakes. Nonbuilding structures shall be designed to provide the strength required to resist the displacements induced by the minimum lateral forces specified in this section. Design shall conform to the applicable provisions of other sections as modified by the provisions contained in Section 1634.

1634.1.2 Criteria. The minimum design seismic forces prescribed in this section are at a level that produce displacements in a fixed base, elastic model of the structure, comparable to those expected of the real structure when responding to the Design Basis Ground Motion. Reductions in these forces using the coefficient R is permitted where the design of nonbuilding structures provides sufficient strength and ductility, consistent with the provisions specified herein for buildings, to resist the effects of seismic ground motions as represented by these design forces.

When applicable, design strengths and other detailed design criteria shall be obtained from other sections or their referenced standards. The design of nonbuilding structures shall use the load combinations or factors specified in Section 1612.2 or 1612.3. For nonbuilding structures designed using Section 1634.3, 1634.4 or 1634.5, the Reliability/Redundancy Factor, ρ, may be taken as 1.0.

When applicable design strengths and other design criteria are not contained in or referenced by this code, such criteria shall be obtained from approved national standards.

1634.1.3 Weight *W*. The weight, *W*, for nonbuilding structures shall include all dead loads as defined for buildings in Section 1630.1.1. For purposes of calculating design seismic forces in nonbuilding structures, *W* shall also include all normal operating contents for items such as tanks, vessels, bins and piping.

1634.1.4 Period. The fundamental period of the structure shall be determined by rational methods such as by using Method B in Section 1630.2.2.

1634.1.5 Drift. The drift limitations of Section 1630.10 need not apply to nonbuilding structures. Drift limitations shall be established for structural or nonstructural elements whose failure would cause life hazards. $P\Delta$ effects shall be considered for structures whose calculated drifts exceed the values in Section 1630.1.3.

1634.1.6 Interaction effects. In Seismic Zones 3 and 4, structures that support flexible nonstructural elements whose combined weight exceeds 25 percent of the weight of the structure shall be designed considering interaction effects between the structure and the supported elements.

1634.2 Lateral Force. Lateral-force procedures for nonbuilding structures with structural systems similar to buildings (those with structural systems which are listed in Table 16-N *[for BSC, HCD 1 & HCD 2] Table 16-N.1*) shall be selected in accordance with the provisions of Section 1629.

> **EXCEPTION:** Intermediate moment-resisting frames (IMRF) may be used in Seismic Zones 3 and 4 for nonbuilding structures in Occupancy Categories 3 and 4 if (1) the structure is less than 50 feet (15 240 mm) in height and (2) the value *R* used in reducing calculated member forces and moments does not exceed 2.8.

1634.3 Rigid Structures. Rigid structures (those with period *T* less than 0.06 second) and their anchorages shall be designed for the lateral force obtained from Formula (34-1).

$$V = 0.7C_a\ IW \qquad (34\text{-}1)$$

The force *V* shall be distributed according to the distribution of mass and shall be assumed to act in any horizontal direction.

1634.4 Tanks with Supported Bottoms. Flat bottom tanks or other tanks with supported bottoms, founded at or below grade, shall be designed to resist the seismic forces calculated using the procedures in Section 1634 for rigid structures considering the entire weight of the tank and its contents. Alternatively, such tanks may be designed using one of the two procedures described below:

1. A response spectrum analysis that includes consideration of the actual ground motion anticipated at the site and the inertial effects of the contained fluid.

2. A design basis prescribed for the particular type of tank by an approved national standard, provided that the seismic zones and occupancy categories shall be in conformance with the provisions of Sections 1629.4 and 1629.2, respectively.

1634.5 Other Nonbuilding Structures. Nonbuilding structures that are not covered by Sections 1634.3 and 1634.4 shall be designed to resist design seismic forces not less than those determined in accordance with the provisions in Section 1630 with the following additions and exceptions:

1. The factors *R* and Ω_o shall be as set forth in Table 16-P. The total design base shear determined in accordance with Section 1630.2 shall not be less than the following:

$$V = 0.56C_aIW \qquad (34\text{-}2)$$

Additionally, for Seismic Zone 4, the total base shear shall also not be less than the following:

$$V = \frac{1.6\ ZN_v\ I}{R}\ W \qquad (34\text{-}3)$$

2. The vertical distribution of the design seismic forces in structures covered by this section may be determined by using the provisions of Section 1630.5 or by using the procedures of Section 1631.

> **EXCEPTION:** For irregular structures assigned to Occupancy Categories 1 and 2 that cannot be modeled as a single mass, the procedures of Section 1631 shall be used.

3. Where an approved national standard provides a basis for the earthquake-resistant design of a particular type of nonbuilding structure covered by this section, such a standard may be used, subject to the limitations in this section:

The seismic zones and occupancy categories shall be in conformance with the provisions of Sections 1629.4 and 1629.2, respectively.

The values for total lateral force and total base overturning moment used in design shall not be less than 80 percent of the values that would be obtained using these provisions.

SECTION 1635 — EARTHQUAKE-RECORDING INSTRUMENTATIONS

For earthquake-recording instrumentations, see Appendix Chapter 16, Division II.

TABLE 16-L—VERTICAL STRUCTURAL IRREGULARITIES

IRREGULARITY TYPE AND DEFINITION	REFERENCE SECTION
1. **Stiffness irregularity—soft story** A soft story is one in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above.	1629.8.4, Item 2
2. **Weight (mass) irregularity** Mass irregularity shall be considered to exist where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	1629.8.4, Item 2
3. **Vertical geometric irregularity** Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral-force-resisting system in any story is more than 130 percent of that in an adjacent story. One-story penthouses need not be considered.	1629.8.4, Item 2
4. **In-plane discontinuity in vertical lateral-force-resisting element** An in-plane offset of the lateral-load-resisting elements greater than the length of those elements.	1630.8.2
5. **Discontinuity in capacity—weak story** A weak story is one in which the story strength is less than 80 percent 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.	1629.9.1

TABLE 16-M—PLAN STRUCTURAL IRREGULARITIES

IRREGULARITY TYPE AND DEFINITION	REFERENCE SECTION
1. **Torsional irregularity—to be considered when diaphragms are not flexible** Torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of the structure.	1633.1; 1633.2.9, Item 6
2. **Re-entrant corners** Plan configurations of a structure and its lateral-force-resisting system contain re-entrant corners, where both projections of the structure beyond a re-entrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.	1633.2.9, Items 6 and 7
3. **Diaphragm discontinuity** Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50 percent from one story to the next.	1633.2.9, Item 6
4. **Out-of-plane offsets** Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements.	1630.8.2; 1633.2.9, Item 6; 2213.9.1
5. **Nonparallel systems** The vertical lateral-load-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral-force-resisting system.	1633.1

TABLE 16-N—STRUCTURAL SYSTEMS[1]
(For occupancies regulated by BSC and HCD 1 & HCD2 use Table 16.1-N)

BASIC STRUCTURAL SYSTEM[2]	LATERAL-FORCE-RESISTING SYSTEM DESCRIPTION	*R*	Ω_o	HEIGHT LIMIT FOR SEISMIC ZONES 3 AND 4 (feet)
				× 304.8 for mm
1. Bearing wall system	1. Light-framed walls with shear panels			
	a. Wood structural panel walls for structures three stories or less	5.5	2.8	65
	b. All other light-framed walls	4.5	2.8	65
	2. Shear walls			
	a. Concrete	4.5	2.8	160
	b. Masonry	4.5	2.8	160
	3. Light steel-framed bearing walls with tension-only bracing	2.8	2.2	65
	4. Braced frames where bracing carries gravity load			
	a. Steel	4.4	2.2	160
	b. Concrete[3]	2.8	2.2	—
	c. Heavy timber	2.8	2.2	65
2. Building frame system	1. Steel eccentrically braced frame (EBF)	7.0	2.8	240
	2. Light-framed walls with shear panels			
	a. Wood structural panel walls for structures three stories or less	6.5	2.8	65
	b. All other light-framed walls	5.0	2.8	65
	3. Shear walls			
	a. Concrete	5.5	2.8	240
	b. Masonry	5.5	2.8	160
	4. Ordinary braced frames			
	a. Steel	5.6	2.2	160
	b. Concrete[3]	5.6	2.2	—
	c. Heavy timber	5.6	2.2	65
	5. Special concentrically braced frames			
	a. Steel	6.4	2.2	240
3. Moment-resisting frame system	1. Special moment-resisting frame (SMRF)			
	a. Steel	8.5	2.8	N.L.
	b. Concrete[4]	8.5	2.8	N.L.
	2. Masonry moment-resisting wall frame (MMRWF)	6.5	2.8	160
	3. Concrete intermediate moment-resisting frame (IMRF)[5]	5.5	2.8	—
	4. Ordinary moment-resisting frame (OMRF)			
	a. Steel[6]	4.5	2.8	160
	b. Concrete[7]	3.5	2.8	—
	5. Special truss moment frames of steel (STMF)	6.5	2.8	240
4. Dual systems	1. Shear walls			
	a. Concrete with SMRF	8.5	2.8	N.L.
	b. Concrete with steel OMRF	4.2	2.8	160
	c. Concrete with concrete IMRF[5]	6.5	2.8	160
	d. Masonry with SMRF	5.5	2.8	160
	e. Masonry with steel OMRF	4.2	2.8	160
	f. Masonry with concrete IMRF[3]	4.2	2.8	—
	g. Masonry with masonry MMRWF	6.0	2.8	160
	2. Steel EBF			
	a. With steel SMRF	8.5	2.8	N.L.
	b. With steel OMRF	4.2	2.8	160
	3. Ordinary braced frames			
	a. Steel with steel SMRF	6.5	2.8	N.L.
	b. Steel with steel OMRF	4.2	2.8	160
	c. Concrete with concrete SMRF[3]	6.5	2.8	—
	d. Concrete with concrete IMRF[3]	4.2	2.8	—
	4. Special concentrically braced frames			
	a. Steel with steel SMRF	7.5	2.8	N.L.
	b. Steel with steel OMRF	4.2	2.8	160
5. Cantilevered column building systems	1. Cantilevered column elements	2.2	2.0	35[7]
6. Shear wall-frame interaction systems	1. Concrete[8]	5.5	2.8	160
7. Undefined systems	See Sections 1629.6.7 and 1629.9.2	—	—	—

N.L.—no limit

[1]See Section 1630.4 for combination of structural systems.

[2]Basic structural systems are defined in Section 1629.6.

[3]Prohibited in Seismic Zones 3 and 4.

[4]Includes precast concrete conforming to Section 1921.2.7.

[5]Prohibited in Seismic Zones 3 and 4, except as permitted in Section 1634.2.

[6]Ordinary moment-resisting frames in Seismic Zone 1 meeting the requirements of Section 2214.6 may use a *R* value of 8.

[7] Total height of the building including cantilevered columns.

[8]Prohibited in Seismic Zones 2A, 2B, 3 and 4. See Section 1633.2.7.

TABLE 16-N.1—[For BSC and HCD 1 & HCD 2] STRUCTURAL SYSTEMS[1]

BASIC STRUCTURAL SYSTEM[2]	LATERAL-FORCE-RESISTING SYSTEM DESCRIPTION	R	Ω_o	HEIGHT LIMIT FOR SEISMIC ZONES 3 AND 4 (feet) x 304.8 for mm
1. Bearing wall system	1. Light-framed walls with shear panels			
	a. Wood structural panel walls for structures three stories or less	5.5	2.8	65
	b. All other light-framed walls	4.5	2.8	65
	2. Shear walls			
	a. Concrete	4.5	2.8	160
	b. Masonry	4.5	2.8	160
	3. Light steel-framed bearing walls with tension-only bracing	2.8	2.2	65
	4. Braced frames where bracing carries gravity load			
	a. Steel	4.4	2.2	160
	b. Concrete[3]	2.8	2.2	—
	c. Heavy timber	2.8	2.2	65
2. Building frame system	1. Steel eccentrically braced frame (EBF)	7.0	2.8	240
	2. Light-framed walls with shear panels.			
	a. Wood structural panel walls for structures three stories or less	6.5	2.8	65
	b. All other light-framed walls	5.0	2.8	65
	3. Shear walls			
	a. Concrete	5.5	2.8	240
	b. Masonry	5.5	2.8	160
	4. Ordinary braced frames			
	a. Steel [6]	*5*	*2*	*35 [6]*
	b. Concrete[3]	5.6	—	—
	c. Heavy timber	5.6	2.2	65
	5. Special concentrically braced frames			
	a. Steel	6.4	2.2	240
3. Moment-resisting frame system	1. Special moment-resisting frame (SMRF)			
	a. Steel	8.5	2.8	N.L.
	b. Concrete[4]	8.5	2.8	N.L.
	2. Masonry moment-resisting wall frame (MMRWF)	6.5	2.8	160
	3. Intermediate moment-resisting frame (IMRF)			
	a. Steel[6]	*4.5*	*2.8*	*35*[6]
	b. Concrete[5]	*5.5*	*2.8*	—
	4. Ordinary moment-resisting frame (OMRF)			
	a. Steel [6]	*3.5*	*2.8*	—
	b. Concrete[8]	3.5	2.8	—
	5. Special truss moment frames of steel (STMF)	6.5	2.8	240
4. Dual systems	1. Shear walls			
	a. Concrete with SMRF	8.5	2.8	N.L.
	b. Concrete with steel OMRF *(Not Permitted)*	4.2	2.8	160
	c. Concrete with concrete IMRF [5]	6.5	2.8	160
	d. Masonry with SMRF	5.5	2.8	160
	e. Masonry with steel OMRF *(Not Permitted)*	4.2	2.8	160
	f. Masonry with concrete IMRF [3]	4.2	2.8	—
	g. Masonry with masonry MMRWF	6.0	2.8	160
	2. Steel EBF			
	a. With steel SMRF	8.5	2.8	N.L.
	b. With steel OMRF *(Not Permitted)*	4.2	2.8	160
	3. Ordinary braced frames *(Not Permitted)*			
	a. Steel with steel SMRF	6.5	2.8	N.L.
	b. Steel with steel OMRF	4.2	2.8	160
	c. Concrete with concrete SMRF[3]	6.5	2.8	—
	d. Concrete with concrete IMRF[3]	4.2	2.8	—
	4. Special concentrically braced frames			
	a. Steel with steel SMRF	7.5	2.8	N.L.
	b. Steel with steel OMRF *(Not Permitted)*	4.2	2.8	160
	5. Steel IMRF *(Not permitted)*			
5. Cantilevered column building systems	1. Cantilevered column elements	2.2	2.0	35[7]
6. Shear wall-frame interaction systems	1. Concrete[8]	5.5	2.8	160
7. Undefined systems	See Sections 1629.6.7 and 1629.9.2	—	—	—

N.L.– no limit

[1]See Section 1630.4 for combination of structural systems.

[2]Basic structural systems are defined in Section 1629.6.

[3]Prohibited in Seismic Zones 3 and 4.

[4]Includes precast concrete conforming to Section 1921.2.7.

[5]Prohibited in Seismic Zones 3 and 4, except as permitted in Section 1634.2.

[6]*Unless otherwise approved by the enforcement agency, in Seismic Zone 4 :*

6.1 Steel IMRF are permitted for buildings 35 ft. or less in height and the dead load of the roof, walls or floors not exceeding 35 psf each; or for single-story buildings 60 ft. or less in height with dead load of the roof or walls not exceeding 15 psf each where the moment joints of field connections are constructed of bolted end plates; or single-family dwellings using light frame construction with R = 3.0 and Ω_o = 2.2.

6.2 Steel OMRF are permitted for buildings 35 ft or less in height with the dead load of the roof, walls or floors not exceeding 15 psf each; or single-story buildings 60 ft or less in height with the dead load of the roof or walls not exceeding 15 psf each and where the moment joints of field connections are constructed of bolted end plates.

6.3 Steel Ordinary Braced Frames are permitted for buildings 35 ft or less in height; or penthouse structures; or single-story buildings 60 ft or less in height with the dead load of the roof or walls not exceeding 15 psf each.

[7]Total height of the building including cantilevered columns.

[8]Prohibited in Seismic Zones 2A, 2B, 3 and 4. See Section 1633.2.7.

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TABLE 16-O—HORIZONTAL FORCE FACTORS, a_p AND R_p

ELEMENTS OF STRUCTURES AND NONSTRUCTURAL COMPONENTS AND EQUIPMENT[1]	a_p	R_p	FOOTNOTE
1. Elements of Structures			
A. Walls including the following:			
(1) Unbraced (cantilevered) parapets.	2.5	3.0	
(2) Exterior walls at or above the ground floor and parapets braced above their centers of gravity.	1.0	3.0	2
(3) All interior-bearing and nonbearing walls.	1.0	3.0	2
B. Penthouse (except when framed by an extension of the structural frame).	2.5	4.0	
C. Connections for prefabricated structural elements other than walls. See also Section 1632.2.	1.0	3.0	3
2. Nonstructural Components			
A. Exterior and interior ornamentations and appendages.	2.5	3.0	
B. Chimneys, stacks and trussed towers supported on or projecting above the roof:			
(1) Laterally braced or anchored to the structural frame at a point below their centers of mass.	2.5	3.0	
(2) Laterally braced or anchored to the structural frame at or above their centers of mass.	1.0	3.0	
C. Signs and billboards.	2.5	3.0	
D. Storage racks (include contents) over 6 feet (1829 mm) tall. ***[For BSC]*** *Storage racks (include contents) with upper storage level more than 5 feet (1524 mm) in height.*	2.5	4.0	4 *4.1*
E. Permanent floor-supported cabinets and book stacks more than 6 feet (1829 mm) in height (include contents).	1.0	3.0	5
F. Anchorage and lateral bracing for suspended ceilings and light fixtures.	1.0	3.0	3, 6, 7, 8
G. Access floor systems. ***[For BSC]***	1.0	3.0	4, 5, 9 *4.1, 5, 9*
H. Masonry or concrete fences over 6 feet (1829 mm) high.	1.0	3.0	
I. Partitions.	1.0	3.0	
3. Equipment			
A. Tanks and vessels (include contents), including support systems.	1.0	3.0	
B. Electrical, mechanical and plumbing equipment and associated conduit and ductwork and piping.	1.0	3.0	5, 10, 11, 12, 13, 14, 15, 16
C. Any flexible equipment laterally braced or anchored to the structural frame at a point below their center of mass.	2.5	3.0	5, 10, 14, 15, 16
D. Anchorage of emergency power supply systems and essential communications equipment. Anchorage and support systems for battery racks and fuel tanks necessary for operation of emergency equipment. See also Section 1632.2.	1.0	3.0	17, 18
E. Temporary containers with flammable or hazardous materials.	1.0	3.0	19
4. Other Components			
A. Rigid components with ductile material and attachments.	1.0	3.0	1
B. Rigid components with nonductile material or attachments.	1.0	1.5	1
C. Flexible components with ductile material and attachments.	2.5	3.0	1
D. Flexible components with nonductile material or attachments.	2.5	1.5	1

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[1]See Section 1627 for definitions of flexible components and rigid components.

[2]See Sections 1633.2.4 and 1633.2.8 for concrete and masonry walls and Section 1632.2 for connections for panel connectors for panels.

[3]Applies to Seismic Zones 2, 3 and 4 only.

[4]Ground supported steel storage racks may be designed using the provisions of Section 1634. Chapter 22, Division VI, may be used for design, provided seismic design forces are equal to or greater than those specified in Section 1632.2 or 1634.2, as appropriate.

[4.1] ***[For BSC]*** Ground supported steel storage racks may be designed using the provisions of Section 1634. Chapter *22A*, Division *X*, may be used for design, provided seismic design forces are equal to or greater than those specified in Section 1632.2 or 1634.2, as appropriate. CA

[5]Only attachments, anchorage or restraints need be designed.

[6]Ceiling weight shall include all light fixtures and other equipment or partitions that are laterally supported by the ceiling. For purposes of determining the seismic force, a ceiling weight of not less than 4 psf (0.19 kN/m^2) shall be used.

[7]Ceilings constructed of lath and plaster or gypsum board screw or nail attached to suspended members that support a ceiling at one level extending from wall to wall need not be analyzed, provided the walls are not over 50 feet (15 240 mm) apart.

[8]Light fixtures and mechanical services installed in metal suspension systems for acoustical tile and lay-in panel ceilings shall be independently supported from the structure above as specified in UBC Standard 25-2, Part III.

[9]W_p for access floor systems shall be the dead load of the access floor system plus 25 percent of the floor live load plus a 10-psf (0.48 kN/m^2) partition load allowance.

[10]Equipment includes, but is not limited to, boilers, chillers, heat exchangers, pumps, air-handling units, cooling towers, control panels, motors, switchgear, transformers and life-safety equipment. It shall include major conduit, ducting and piping, which services such machinery and equipment and fire sprinkler systems. See Section 1632.2 for additional requirements for determining a_p for nonrigid or flexibly mounted equipment.

[11]Seismic restraints may be omitted from piping and duct supports if all the following conditions are satisfied:

[11.1]Lateral motion of the piping or duct will not cause damaging impact with other systems.

[11.2]The piping or duct is made of ductile material with ductile connections.

[11.3]Lateral motion of the piping or duct does not cause impact of fragile appurtenances (e.g., sprinkler heads) with any other equipment, piping or structural member.

(Continued)

FOOTNOTES TO TABLE 16-O—(Continued)

[11.4]Lateral motion of the piping or duct does not cause loss of system vertical support.

[11.5]Rod-hung supports of less than 12 inches (305 mm) in length have top connections that cannot develop moments.

[11.6]Support members cantilevered up from the floor are checked for stability.

[12]Seismic restraints may be omitted from electrical raceways, such as cable trays, conduit and bus ducts, if all the following conditions are satisfied:

[12.1]Lateral motion of the raceway will not cause damaging impact with other systems.

[12.2]Lateral motion of the raceway does not cause loss of system vertical support.

[12.3]Rod-hung supports of less than 12 inches (305 mm) in length have top connections that cannot develop moments.

[12.4]Support members cantilevered up from the floor are checked for stability.

[13]Piping, ducts and electrical raceways, which must be functional following an earthquake, spanning between different buildings or structural systems shall be sufficiently flexible to withstand relative motion of support points assuming out-of-phase motions.

[14]Vibration isolators supporting equipment shall be designed for lateral loads or restrained from displacing laterally by other means. Restraint shall also be provided, which limits vertical displacement, such that lateral restraints do not become disengaged. a_p and R_p for equipment supported on vibration isolators shall be taken as 2.5 and 1.5, respectively, except that if the isolation mounting frame is supported by shallow or expansion anchors, the design forces for the anchors calculated by Formula (32-1), (32-2) or (32-3) shall be additionally multiplied by a factor of 2.0.

[15]Equipment anchorage shall not be designed such that lateral loads are resisted by gravity friction (e.g., friction clips).

[16]Expansion anchors, which are required to resist seismic loads in tension, shall not be used where operational vibrating loads are present.

[17]Movement of components within electrical cabinets, rack- and skid-mounted equipment and portions of skid-mounted electromechanical equipment that may cause damage to other components by displacing, shall be restricted by attachment to anchored equipment or support frames.

[18]Batteries on racks shall be restrained against movement in all directions due to earthquake forces.

[19]Seismic restraints may include straps, chains, bolts, barriers or other mechanisms that prevent sliding, falling and breach of containment of flammable and toxic materials. Friction forces may not be used to resist lateral loads in these restraints unless positive uplift restraint is provided which ensures that the friction forces act continuously.

TABLE 16-P—*R* AND Ω_o FACTORS FOR NONBUILDING STRUCTURES

STRUCTURE TYPE	R	Ω_o
1. Vessels, including tanks and pressurized spheres, on braced or unbraced legs.	2.2	2.0
2. Cast-in-place concrete silos and chimneys having walls continuous to the foundations.	3.6	2.0
3. Distributed mass cantilever structures such as stacks, chimneys, silos and skirt-supported vertical vessels.	2.9	2.0
4. Trussed towers (freestanding or guyed), guyed stacks and chimneys.	2.9	2.0
5. Cantilevered column-type structures.	2.2	2.0
6. Cooling towers.	3.6	2.0
7. Bins and hoppers on braced or unbraced legs.	2.9	2.0
8. Storage racks.	3.6	2.0
9. Signs and billboards.	3.6	2.0
10. Amusement structures and monuments.	2.2	2.0
11. All other self-supporting structures not otherwise covered.	2.9	2.0

TABLE 16-Q—SEISMIC COEFFICIENT C_a

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
S_F	See Footnote 1				

[1]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

Chapter 16A *[For DSA/SS, OSHPD 1 & 4]*
STRUCTURAL DESIGN REQUIREMENTS

NOTE: This chapter has been revised in its entirety.

Division I—GENERAL DESIGN REQUIREMENTS

SECTION 1601A — SCOPE

1601A.1 *This chapter is applicable to community colleges, public schools and state-owned or state-leased essential services buildings regulated by the Division of the State Architect, Office of Regulatory Services.*

1601A.2 Scope. *This chapter is applicable to hospitals, skilled nursing facilities, intermediate-care facilities and correctional treatment centers regulated by the Office of Statewide Health Planning and Development.*

> ***EXCEPTION [For OSHPD 2]:*** *Single-story Type V skilled nursing or intermediate care facilities utilizing wood-frame or light-steel-frame construction as defined in Health and Safety Code Section 129725, which shall comply with UBC Chapter 16 and any applicable amendments therein.*

For structures regulated by the Office of Statewide Health Planning and Development, the scope of application [OSHPD 1, 2, 3 and 4] is defined as follows:

OSHPD 1. *General acute-care hospitals. For Structural Regulations: Skilled nursing facilities and/or intermediate care facilities except those skilled nursing facilities and intermediate care facilities of single-story, Type V, wood or light-steel frame construction.*

OSHPD 2. *Skilled nursing facilities and intermediate care facilities. For structural regulations: Single-story, Type V skilled nursing and/or intermediate-care facilities utilizing wood or light steel-frame construction.*

OSHPD 3. *Licensed clinics.*

OSHPD 4. *Correctional Treatment Centers.*

1601A.2.1 Existing hospitals, skilled nursing facilities, intermediate-care facilities and correctional treatment centers regulated by the Office of Statewide Health Planning and Development [for OSHPD 1, 2 and 4]. *See Section 7-109, Part 1, Title 24, California Code of Regulations and Division VI-R, Chapter 16A, Part 2, Title 24, California Building Code.*

1601A.3 Existing Public School Buildings. *See Sections 4-308 and 4-309, Part 1, Title 24, California Code of Regulations, for requirements governing the design and construction of alterations, additions or repairs to existing public school buildings.*

1601A.4 Existing State-owned or State-leased Essential Service Buildings. *See Section 4-206 and 4-207, Part 1, Title 24, California Code of Regulations.*

SECTION 1602A — DEFINITIONS

The following terms are defined for use in this code:

ALLOWABLE STRESS DESIGN is a method of proportioning structural elements such that computed stresses produced in the elements by the allowable stress load combinations do not exceed specified allowable stress (also called working stress design).

BALCONY, EXTERIOR, is an exterior floor system projecting from a structure and supported by that structure, with no additional independent supports.

DEAD LOADS consist of the weight of all materials and fixed equipment incorporated into the building or other structure.

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

FACTORED LOAD is the product of a load specified in Sections 1606A through 1611A and a load factor. See Section 1612A.2 for combinations of factored loads.

HOSPITAL BUILDING *[for OSHPD 1, 2, 3 and 4]. Any building defined in Section 129725, Health and Safety Code.*

LIMIT STATE is a condition in which a structure or component is judged either to be no longer useful for its intended function (serviceability limit state) or to be unsafe (strength limit state).

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

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

STRENGTH DESIGN is a method of proportioning structural elements such that the computed forces produced in the elements by the factored load combinations do not exceed the factored element strength. The term "strength design" is used in the design of concrete and masonry structures.

SECTION 1603A — NOTATIONS

D = dead load.

E = earthquake load set forth in Section 1630A.1.

E_m = estimated maximum earthquake force that can be developed in the structure as set forth in Section 1630A.1.1.

F = load due to fluids.

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

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

L_r = roof live load, including any permitted live load reduction.

P = ponding load.

S = snow load.

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

W = load due to wind pressure.

SECTION 1604A — STANDARDS

The standards listed below are recognized standards (see Section 3504).

1. Wind Design.

1.1 ASCE 7, Chapter 6, Minimum Design Loads for Buildings and Other Structures

1.2 ANSI EIA/TIA 222-E, Structural Standards for Steel Antenna Towers and Antenna Supporting Structures

1.3 ANSI/NAAMM FP1001, Guide Specifications for the Design Loads of Metal Flagpoles

SECTION 1605A — DESIGN

1605A.1 General. Buildings and other structures and all portions thereof shall be designed and constructed to sustain, within the limitations specified in this code, all loads set forth in Chapter 16A and elsewhere in this code, combined in accordance with Section 1612A. Design shall be in accordance with Strength Design, Load and Resistance Factor Design or Allowable Stress Design methods, as permitted by the applicable materials chapters.

EXCEPTION: Unless otherwise required by the building official, buildings or portions thereof that are constructed in accordance with the conventional light-framing requirements specified in Chapter 23A of this code shall be deemed to meet the requirements of this section.

1605A.2 Rationality. Any system or method of construction to be used shall be based on a rational analysis in accordance with well-established principles of mechanics. Such analysis shall result in a system that provides a complete load path capable of transferring all loads and forces from their point of origin to the load-resisting elements. The analysis shall include, but not be limited to, the provisions of Sections 1605A.2.1 through 1605A.2.3.

1605A.2.1 Distribution of horizontal shear. The total lateral force shall be distributed to the various vertical elements of the lateral-force-resisting system in proportion to their rigidities considering the rigidity of the horizontal bracing system or diaphragm. Rigid elements that are assumed not to be part of the lateral-force-resisting system may be incorporated into buildings, provided that their effect on the action of the system is considered and provided for in the design.

Provision shall be made for the increased forces induced on resisting elements of the structural system resulting from torsion due to eccentricity between the center of application of the lateral forces and the center of rigidity of the lateral-force-resisting system. For accidental torsion requirements for seismic design, see Section 1630A.6.

1605A.2.2 Stability against overturning. Every structure shall be designed to resist the overturning effects caused by the lateral forces specified in this chapter. See Section 1611A.6 for retaining walls, Section *1621A* for wind and Section *1630A.8* for seismic.

1605A.2.3 Anchorage. Anchorage of the roof to walls and columns, and of walls and columns to foundations, shall be provided to resist the uplift and sliding forces that result from the application of the prescribed forces.

Concrete and masonry walls shall be anchored to all floors, roofs and other structural elements that provide lateral support for the wall. Such anchorage shall provide a positive direct connection capable of resisting the horizontal forces specified in this chapter but not less than the minimum forces in Section 1611A.4. In addition, in Seismic Zones 3 and 4, diaphragm to wall anchorage using embedded straps shall have the straps attached to or hooked around the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet (1219 mm). Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall. See Sections 1632A, 1633A.2.8 and 1633A.2.9 for earthquake design requirements.

1605A.3 Erection of Structural Framing. Walls and structural framing shall be erected true and plumb in accordance with the design.

1605A.4 Alternate Method. *Acceptance and approval by the enforcement agency of design, materials or types of construction other than those recognized in these regulations shall be dependent on rational analysis, or if rational analysis is not applicable, on tests establishing physical characteristics and demonstrating ability to meet prescribed performance criteria for safety and durability. Where tests are required to substantiate the adequacy of alternate designs, they shall be conducted or witnessed by a qualified independent testing laboratory as directed by the project architect or engineer. The test program and testing method shall be subject to approval by the enforcement agency.*

1605A.5 Construction Procedures. *Where unusual erection or construction procedures are considered essential by the project architect or structural engineer in order to accomplish the intent of the design or influence the design, such procedures shall be indicated on the plans or in the specifications and shall have the prior approval of the enforcement agency.*

SECTION 1606A — DEAD LOADS

1606A.1 General. Dead loads shall be as defined in Section 1602A and this section.

1606A.2 Partition Loads. Floors in office buildings and other buildings where partition locations are subject to change shall be designed to support, in addition to all other loads, a uniformly distributed dead load equal to 20 pounds per square foot (psf) (0.96 kN/m^2) of floor area.

EXCEPTION: Access floor systems shall be designed to support, in addition to all other loads, a uniformly distributed dead load not less than 10 psf (0.48 kN/m^2) of floor area.

SECTION 1607A — LIVE LOADS

1607A.1 General. Live loads shall be the maximum loads expected by the intended use or occupancy but in no case shall be less than the loads required by this section.

1607A.2 Critical Distribution of Live Loads. Where structural members are arranged to create continuity, members shall be designed using the loading conditions, which would cause maximum shear and bending moments. This requirement may be satisfied in accordance with the provisions of Section 1607A.3.2 or 1607A.4.2, where applicable.

1607A.3 Floor Live Loads.

1607A.3.1 General. Floors shall be designed for the unit live loads as set forth in Table 16A-A. These loads shall be taken as the minimum live loads in pounds per square foot of horizontal projection to be used in the design of buildings for the occupancies listed, and loads at least equal shall be assumed for uses not listed in this section but that create or accommodate similar loadings.

Where it can be determined in designing floors that the actual live load will be greater than the value shown in Table 16A-A, the actual live load shall be used in the design of such buildings or portions thereof. Special provisions shall be made for machine and apparatus loads.

1607A.3.2 Distribution of uniform floor loads. Where uniform floor loads are involved, consideration may be limited to full dead load on all spans in combination with full live load on adjacent spans and alternate spans.

1607A.3.3 Concentrated loads. Provision shall be made in designing floors for a concentrated load, *L,* as set forth in Table 16A-A placed upon any space $2^1/_2$ feet (762 mm) square, wherever this load upon an otherwise unloaded floor would produce stresses greater than those caused by the uniform load required therefor.

Provision shall be made in areas where vehicles are used or stored for concentrated loads, *L,* consisting of two or more loads spaced 5 feet (1524 mm) nominally on center without uniform live loads. Each load shall be 40 percent of the gross weight of the maximum-size vehicle to be accommodated. Parking garages for the storage of private or pleasure-type motor vehicles with no repair or refueling shall have a floor system designed for a concentrated load of not less than 2,000 pounds (8.9 kN) acting on an area of 20 square inches (12 903 mm^2) without uniform live loads. The condition of concentrated or uniform live load, combined in accordance with Section 1612A.2 or 1612A.3 as appropriate, producing the greatest stresses shall govern.

Areas other than garages subject to unlimited, uncontrolled vehicle access shall be designed for a minimum loading of HS 20-44 in accordance with the Standard Specifications for Highway Bridges published by the American Association of State Highway and Transportation Officials.

1607A.3.4 Special loads. Provision shall be made for the special vertical and lateral loads as set forth in Table 16A-B.

1607A.3.5 Live loads posted. *The live loads used in the design of floor and other areas listed in Category 8, 9, 10, 11, 17, 18, 19 or 20 of Table 16A-A, or for other special-purpose areas shall be conspicuously posted in that part of each story in which they apply using durable metal signs. The sign shall be in letters not less than 1 inch (25 mm) high on contrasting background.*

1607A.3.5.1 [For DSA-SS]. *The owner or school board shall be responsible for keeping the actual load below the allowable limits.*

1607A.3.5.2 [For OSHPD 1 & 4]. *The hospital owner or hospital governing board shall be responsible for keeping the actual load below the allowable limits.*

1607A.4 Roof Live Loads.

1607A.4.1 General. Roofs shall be designed for the unit live loads, L_r, set forth in Table 16A-C. The live loads shall be assumed to act vertically upon the area projected on a horizontal plane. *The design dead loads shall provide for the weight of at least one reroofing in addition to other applicable loadings if the new roofing can be applied over the original roofing without its removal.*

1607A.4.2 Distribution of loads. Where uniform roof loads are involved in the design of structural members arranged to create continuity, consideration may be limited to full dead loads on all spans in combination with full roof live loads on adjacent spans and on alternate spans.

> **EXCEPTION:** Alternate span loading need not be considered where the uniform roof live load is 20 psf (0.96 kN/m^2) or more or where load combinations, including snow load, result in larger members or connections.

For those conditions where light-gage metal preformed structural sheets serve as the support and finish of roofs, roof structural members arranged to create continuity shall be considered adequate if designed for full dead loads on all spans in combination with the most critical one of the following superimposed loads:

1. Snow load in accordance with Section 1614A.
2. The uniform roof live load, L_r, set forth in Table 16A-C on all spans.
3. A concentrated gravity load, L_r, of 2,000 pounds (8.9 kN) placed on any span supporting a tributary area greater than 200 square feet (18.58 m^2) to create maximum stresses in the member, whenever this loading creates greater stresses than those caused by the uniform live load. The concentrated load shall be placed on the member over a length of $2^1/_2$ feet (762 mm) along the span. The concentrated load need not be applied to more than one span simultaneously.
4. Water accumulation as prescribed in Section 1611A.7.

1607A.4.3 Unbalanced loading. Unbalanced loads shall be used where such loading will result in larger members or connections. Trusses and arches shall be designed to resist the stresses caused by unit live loads on one half of the span if such loading results in reverse stresses, or stresses greater in any portion than the stresses produced by the required unit live load on the entire span. For roofs whose structures are composed of a stressed shell, framed or solid, wherein stresses caused by any point loading are distributed throughout the area of the shell, the requirements for unbalanced unit live load design may be reduced 50 percent.

1607A.4.4 Special roof loads. Roofs to be used for special purposes shall be designed for appropriate loads as approved by the *enforcement agency.*

Greenhouse roof bars, purlins and rafters shall be designed to carry a 100-pound-minimum (444.8 N) concentrated load, L_r, in addition to the uniform live load.

Uncovered open-frame roof structures shall be designed for a vertical live load of not less than 10 pounds per square foot (0.48 kN/m^2) of the total area encompassed by the framework.

1607A.5 Reduction of Live Loads. The design live load determined using the unit live loads as set forth in Table 16A-A for floors and Table 16A-C, Method 2, for roofs may be reduced on any member supporting more than 150 square feet (13.94 m^2), including flat slabs, except for floors in places of public assembly and for live loads greater than 100 psf (4.79 kN/m^2), in accordance with the following formula:

$$R = r\,(A - 150) \qquad (7A\text{-}1)$$

For **SI:**

$$R = r\,(A - 13.94)$$

The reduction shall not exceed 40 percent for members receiving load from one level only, 60 percent for other members or *R,* as determined by the following formula:

$$R = 23.1\,(1 + D/L) \qquad (7A\text{-}2)$$

WHERE:

- A = area of floor or roof supported by the member, square feet (m^2).
- D = dead load per square foot (m^2) of area supported by the member.
- L = unit live load per square foot (m^2) of area supported by the member.
- R = reduction in percentage.
- r = rate of reduction equal to 0.08 percent for floors. See Table 16A-C for roofs.

For storage loads exceeding 100 psf (4.79 kN/m^2), no reduction shall be made, except that design live loads on columns may be reduced 20 percent.

The live load reduction shall not exceed 40 percent in garages for the storage of private pleasure cars having a capacity of not more than nine passengers per vehicle.

1607A.6 Alternate Floor Live Load Reduction. *Not adopted by the State of California.*

SECTION 1608A — SNOW LOADS

Snow loads shall be determined in accordance with Chapter 16A, Division II.

SECTION 1609A — WIND LOADS

Wind loads shall be determined in accordance with Chapter 16A, Division III.

SECTION 1610A — EARTHQUAKE LOADS

Earthquake loads shall be determined in accordance with Chapter 16A, Division IV.

SECTION 1611A — OTHER MINIMUM LOADS

1611A.1 General. In addition to the other design loads specified in this chapter, structures shall be designed to resist the loads specified in this section and the special loads set forth in Table 16A-B.

1611A.2 Other Loads. Buildings and other structures and portions thereof shall be designed to resist all loads due to applicable fluid pressures, *F,* lateral soil pressures, *H,* ponding loads, *P,* and self-straining forces, *T.* See Section 1611A.7 for ponding loads for roofs.

1611A.3 Impact Loads. Impact loads shall be included in the design of any structure where impact loads occur.

1611A.4 Anchorage of Concrete and Masonry Walls. Concrete and masonry walls shall be anchored as required by Section 1605A.2.3. Such anchorage shall be capable of resisting the load combinations of Section 1612A.2 or 1612A.3 using the greater of the wind or earthquake loads required by this chapter or a minimum horizontal force of 280 pounds per linear foot (4.09 kN/m) of wall, substituted for *E*.

1611A.5 Interior Wall Loads. Interior walls, permanent partitions and temporary partitions that exceed 6 feet (1829 mm) in height shall be designed to resist all loads to which they are subjected but not less than a load, *L*, of 5 psf (0.24 kN/m^2) applied perpendicular to the walls. The 5 psf (0.24 kN/m^2) load need not be applied simultaneously with wind or seismic loads. The deflection of such walls under a load of 5 psf (0.24 kN/m^2) shall not exceed $^1/_{240}$ of the span for walls with brittle finishes and $^1/_{120}$ of the span for walls with flexible finishes. See Table 16A-O for earthquake design requirements where such requirements are more restrictive.

EXCEPTION: Flexible, folding or portable partitions are not required to meet the load and deflection criteria but must be anchored to the supporting structure to meet the provisions of this code.

1611A.6 Retaining Walls. Retaining walls shall be designed to resist loads due to the lateral pressure of retained material in accordance with accepted engineering practice. *Retaining walls higher than 12 feet (3658 mm), as measured from the top of the foundation, shall be designed to resist the additional earth pressure caused by seismic ground shaking.* Walls retaining drained soil, where the surface of the retained soil is level, shall be designed for a load, *H,* equivalent to that exerted by a fluid weighing not less than 30 psf per foot of depth (4.71 kN/m^2/m) and having a depth equal to that of the retained soil. Any surcharge shall be in addition to the equivalent fluid pressure.

Retaining walls shall be designed to resist sliding by at least 1.5 times the lateral force and overturning by at least 1.5 times the overturning moment, using allowable stress design loads.

The resultant of the vertical loads and lateral pressures acting on the wall and its base shall pass through the middle half of the bottom of the footing.

Retaining walls shall be restrained against sliding by friction of the base against the earth, by passive resistance of the soil or by a combination of the two. When used, keys may be assumed to lower the plane of frictional resistance and depth of passive resistance to the level of the bottom of the key. Passive resistance pressures shall be assumed to act on a vertical plane located at the toe of the footing. Overturning shall be computed about the bottom of the spread footing. Passive resistance on the face of the wall may be included in computing resistance to overturning. Frictional resistance on the face of the wall may be included in computing resistance to overturning, except when lateral loads include seismic forces. See Section 1611A.13 for overturning provisions for free-standing walls.

Gravity-type retaining walls utilizing precast concrete units may be used as an alternative to the conventional cantilever retaining systems only after they have been accepted by the enforcement agency.

1611A.7 Water Accumulation. All roofs shall be designed with sufficient slope or camber to ensure adequate drainage after the long-term deflection from dead load or shall be designed to resist ponding load, *P,* combined in accordance with Section 1612A.2 or 1612A.3. Ponding load shall include water accumulation from any source, including snow, due to deflection. See Section 1506 and Table 16A-C, Footnote 3, for drainage slope. See Section *1613A* for deflection criteria.

1611A.8 Hydrostatic Uplift. All foundations, slabs and other footings subjected to water pressure shall be designed to resist a uniformly distributed uplift load, *F,* equal to the full hydrostatic pressure.

1611A.9 Flood-resistant Construction. For flood-resistant construction requirements, where specifically adopted, see Appendix Chapter 31, Division I.

1611A.10 Heliport and Helistop Landing Areas. In addition to other design requirements of this chapter, heliport and helistop landing or touchdown areas shall be designed for the following loads, combined in accordance with Section 1612A.2 or 1612A.3:

1. Dead load plus actual weight of the helicopter.

2. Dead load plus a single concentrated impact load, *L*, covering 1 square foot (0.093 m^2) of 0.75 times the fully loaded weight of the helicopter if it is equipped with hydraulic-type shock absorbers, or 1.5 times the fully loaded weight of the helicopter if it is equipped with a rigid or skid-type landing gear.

3. The dead load plus a uniform live load, *L*, of 100 psf (4.8 kN/m^2). The required live load may be reduced in accordance with Section 1607A.5 or 1607A.6.

1611A.11 Prefabricated Construction.

1611A.11.1 Connections. Every device used to connect prefabricated assemblies shall be designed as required by this code and shall be capable of developing the strength of the members connected, except in the case of members forming part of a structural frame designed as specified in this chapter. Connections shall be capable of withstanding uplift forces as specified in this chapter.

design approach used in the design of the structure, provided load combinations of Section 1612A.3 are utilized *and the foundation conforms with the requirements of Section 1633A.2.12*.

1629A.2 Occupancy Categories. For purposes of earthquake-resistant design, each structure shall be placed in one of the occupancy categories listed in Table 16A-K. Table 16A-K assigns importance factors, I and I_p, and structural observation requirements for each category.

1629A.3 Site Geology and Soil Characteristics. Each site shall be assigned a soil profile type based on properly substantiated geotechnical data using the site categorization procedure set forth in Division V, Section 1636A and Table 16A-J.

> **EXCEPTION:** When the soil properties are not known in sufficient detail to determine the soil profile type, Type S_D shall be used. Soil Profile Type S_E or S_F need not be assumed unless the building official determines that Type S_E or S_F may be present at the site or in the event that Type S_E or S_F is established by geotechnical data.

1629A.3.1 Soil profile type. Soil Profile Types S_A, S_B, S_C, S_D and S_E are defined in Table 16A-J and Soil Profile Type S_F is defined as soils requiring site-specific evaluation as follows:

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

2. Peats and/or highly organic clays, where the thickness of peat or highly organic clay exceeds 10 feet (3048 mm).

3. Very high plasticity clays with a plasticity index, $PI > 75$, where the depth of clay exceeds 25 feet (7620 mm).

4. Very thick soft/medium stiff clays, where the depth of clay exceeds 120 feet (36 576 mm).

1629A.4 Site Seismic Hazard Characteristics. Seismic hazard characteristics for the site shall be established based on the seismic zone and proximity of the site to active seismic sources, site soil profile characteristics and the structure's importance factor.

1629A.4.1 Seismic zone. Each site shall be assigned a seismic zone in accordance with Figure 16A-2. Each structure shall be assigned a seismic zone factor Z, in accordance with Table 16A-I.

Seismic Zone 4 shall include all of the area within the boundaries of the state of California except for those areas designated as being in Seismic Zone 3. Seismic Zone 3 shall include all of the area within the following boundaries:

Alpine County, Amador County, Butte County, Calaveras County, Colusa County, El Dorado County, the portion of Fresno County northeast of Interstate Highway 5 and west of longitude 118°45', Glenn County, the portion of Inyo County east of longitude 116°30', the portion of Kings County northeast of Interstate Highway 5, Lassen County, Madera County, Mariposa County, the portion of Merced County northeast of Interstate Highway 5, Modoc County, Nevada County, Placer County, Plumas County, the portion of Riverside County east of longitude 115°30', the portion of Sacramento County north of latitude 38°15', and east of longitude 121°35', the portion of San Bernardino County east of longitude 116°00', the portion of San Joaquin County northeast of Interstate Highway 580, Shasta County, Sierra County, Siskiyou County, the portion of Stanislaus County northeast of Interstate Highway 5, Sutter County, Tehema County, the portion of Trinity County east of the South Fork Trinity River, the portion of Tulare County west of longitude 118°30', Tuolumne County, Yolo County and Yuba County.

1629A.4.2 Seismic Zone 4 near-source factor. In Seismic Zone 4, each site shall be assigned a near-source factor in accordance with Table 16A-S and the Seismic Source Type set forth in Table 16A-U. The value of N_a used to determine C_a need not exceed 1.1 for structures complying with all the following conditions:

1. The soil profile type is S_A, S_B, S_C or S_D.

2. $\rho = 1.0$.

3. Except in single-story structures, Group R, Division 3 and Group U, Division 1 Occupancies, moment frame systems designated as part of the lateral-force-resisting system shall be special moment-resisting frames.

4. The exceptions to Section 2213A.7.5 shall not apply, except for columns in one-story buildings or columns at the top story of multistory buildings.

5. None of the following structural irregularities is present: Type 1, 4 or 5 of Table 16A-L, and Type 1 or 4 of Table 16A-M.

1629A.4.3 Seismic response coefficients. Each structure shall be assigned a seismic coefficient, C_a, in accordance with Table 16A-Q and a seismic coefficient, C_v, in accordance with Table 16A-R.

1629A.5 Configuration Requirements.

1629A.5.1 General. Each structure shall be designated as being structurally regular or irregular in accordance with Sections 1629A.5.2 and 1629A.5.3.

1629A.5.2 Regular structures. Regular structures have no significant physical discontinuities in plan or vertical configuration or in their lateral-force-resisting systems such as the irregular features described in Section 1629A.5.3.

1629A.5.3 Irregular structures.

1. Irregular structures have significant physical discontinuities in configuration or in their lateral-force-resisting systems. Irregular features include, but are not limited to, those described in Tables 16A-L and 16A-M.

2. Structures having any of the features listed in Table 16A-L shall be designated as if having a vertical irregularity.

3. Structures having any of the features listed in Table 16A-M shall be designated as having a plan irregularity.

1629A.6 Structural Systems.

1629A.6.1 General. Structural systems shall be classified as one of the types listed in Table 16A-N and defined in this section.

1629A.6.2 Bearing wall system. A structural system without a complete vertical load-carrying space frame. Bearing walls or bracing systems provide support for all or most gravity loads. Resistance to lateral load is provided by shear walls or braced frames.

1629A.6.3 Building frame system. A structural system with an essentially complete space frame providing support for gravity loads. Resistance to lateral load is provided by shear walls or braced frames.

1629A.6.4 Moment-resisting frame system. A structural system with an essentially complete space frame providing support for gravity loads. Moment-resisting frames provide resistance to lateral load primarily by flexural action of members.

1629A.6.5 Dual system. A structural system with the following features:

1. An essentially complete space frame that provides support for gravity loads.

2. ***Resistance to lateral load.***

2.1 *[For DSA/SS/]* Resistance to lateral load is provided by shear walls or braced frames and moment-resisting frames (SMRF, * * * MMRWF or steel OMRF). The moment-resisting frames shall be designed to * * * resist at least 25 percent of the design base shear.

2.2 *[For OSHPD 1 & 4]* Resistance to lateral load is provided by shear walls or braced frames and moment-resisting frames *(SMRF or MMRWF)*. The moment-resisting frames shall be designed to * * * resist at least 25 percent of the design base shear.

3. The two systems shall be designed to resist the total design base shear in proportion to their relative rigidities considering the interaction of the dual system at all levels.

4. If the complete system analysis specified in Item 3 shows the moment-resisting frame resists less than 25 percent of the design base shear, the forces in the moment-resisting frame shall be ratioed up by a factor of $0.25V/V_F$, where V_F is the portion of the base shear carried by the moment-resisting frame.

1629A.6.6 Cantilevered column system. A structural system relying on cantilevered column elements for lateral resistance.

1629A.6.7 Undefined structural system. A structural system not listed in Table 16A-N.

1629A.6.8 Nonbuilding structural system. A structural system conforming to Section 1634A.

1629A.7 Height Limits. Height limits for the various structural systems in Seismic Zones 3 and 4 are given in Table 16A-N.

> **EXCEPTION:** Regular structures may exceed these limits by not more than 50 percent for unoccupied structures, which are not accessible to the general public.

1629A.8 Selection of Lateral-force Procedure.

1629A.8.1 General. Any structure may be, and certain structures defined below shall be, designed using the dynamic lateral-force procedures of Section 1631A.

1629A.8.2 Simplified static. *[Not adopted by OSHPD.]* The simplified static lateral-force procedure set forth in Section 1630A.2.3 may be used for the following structures of Occupancy Category 4 or 5:

1. Buildings of any occupancy (including single-family dwellings) not more than three stories in height excluding basements, that use light-frame construction.

2. Other buildings not more than two stories in height excluding basements.

1629A.8.3 Static. The static lateral force procedure of Section 1630A may be used for the following structures:

1. *Not adopted by OSHPD and DSA.*

2. Regular structures under 240 feet (73 152 mm) in height with lateral force resistance provided by systems listed in Table 16A-N, except where Section 1629A.8.4, Item 4, applies.

3. Irregular structures *with flexible diaphragms not more than three stories or 30 feet (9144 mm) in height.*

4. Structures having a flexible upper portion supported on a rigid lower portion where both portions of the structure considered separately can be classified as being regular, the average story stiffness of the lower portion is at least 10 times the average story stiffness of the upper portion and the period of the entire structure is not greater than 1.1 times the period of the upper portion considered as a separate structure fixed at the base.

5. Wood-frame structures having wood shear walls and wood diaphragms.

6. Irregular structures with reentrant corners, plan irregularity Type 2, Table 16A-M, which are otherwise eligible for static analysis.

1629A.8.4 Dynamic. The dynamic lateral-force procedure of Section 1631A shall be used for all other structures, including the following:

1. Structures 240 feet (73 152 mm) or more in height * * *.

2. Structures having a *plan or* vertical irregularity as defined in Table 16A-L *or 16A-M*, except as permitted by *Section 1629A.8.3 and* Section 1630A.4.2.

3. Structures over five stories or 65 feet (19 812 mm) in height in Seismic Zones 3 and 4 not having the same structural system throughout their height except as permitted by Section 1630A.4.2.

4. Structures, regular or irregular, *except those defined in Section 1629A.8.3, Items 3 and 5,* located on Soil Profile Type S_F, that have a period greater than *0.5* second *as calculated in accordance with Method B in Section 1630A.2.2*. The analysis shall include the effects of the soils at the site and shall conform to Section 1631A.2, Item 4.

1629A.9 System Limitations.

1629A.9.1 Discontinuity. Structures with a discontinuity in capacity, vertical irregularity Type 5 as defined in Table 16A-L, *are not permitted.*

1629A.9.2 Undefined structural systems. For undefined structural systems not listed in Table 16A-N, the coefficient R shall be substantiated by approved cyclic test data and analyses. The following items shall be addressed when establishing R:

1. Dynamic response characteristics,
2. Lateral force resistance,
3. Overstrength and strain hardening or softening,
4. Strength and stiffness degradation,
5. Energy dissipation characteristics,
6. System ductility, and
7. Redundancy.

Undefined systems shall comply with Section 1631A.2.2.

1629A.9.3 Irregular features. All structures having irregular features described in Table 16A-L or 16A-M shall be designed to meet the additional requirements of those sections referenced in the tables.

1629A.9.4 Severe Soft Story. *Structures with a Severe Soft Story vertical irregularity Type 1b, as defined in Table 16A-L, are not permitted.*

1629A.9.5 Severe torsional irregularity. *Structures with a severe torsional irregularity, plan irregularity Type 1b, as defined in Table 16A-M are not permitted if N_a or N_v is greater than 1.0.*

1629A.10 Alternative Procedures.

1629A.10.1 General. Alternative lateral-force procedures using rational analyses based on well-established principles of mechanics may be used in lieu of those prescribed in these provisions *when approved by the enforcement agency.*

1629A.10.2 Seismic isolation. Seismic isolation, energy dissipation and damping systems may be used in the design of structures when approved by the *enforcement agency* and when special detailing is used to provide results equivalent to those obtained by the use of conventional structural systems.

For alternate design procedures on seismic isolation systems, refer to Appendix Chapter 16A, Division VII, Earthquake Regulations for Seismic-Isolated Structures.

SECTION 1630A — MINIMUM DESIGN LATERAL FORCES AND RELATED EFFECTS

1630A.1 Earthquake Loads and Modeling Requirements.

1630A.1.1 Earthquake loads. *Any structure which does not have a highly irregular shape, large differences in lateral resistance or stiffness between adjacent stories, or other unusual structural features which could significantly affect the dynamic response, may be designed and constructed to resist the minimum lateral seismic forces set forth in the provisions of this section. The equivalent static lateral seismic forces assumed to act on parts or portions of structures and their anchorage shall be as set forth in Section 1632A. The equivalent static lateral seismic forces assumed to act on nonstructural components and their anchorage shall be as set forth in Section 1632A.* Structures shall be designed for ground motion producing structural response and seismic forces in any horizontal direction. The following earthquake loads shall be used in the load combinations set forth in Section 1612A:

$$E = \rho E_h \pm E_v \qquad (30A\text{-}1)$$

$$E_m = \Omega_o E_h \qquad (30A\text{-}2)$$

WHERE:

E = the earthquake load on an element of the structure resulting from the combination of the horizontal component, E_h, and the vertical component, E_v.

E_h = the earthquake load due to the base shear, V, as set forth in Section 1630A.2 or the design lateral force, F_p, as set forth in Section 1632A.

E_m = the estimated maximum earthquake force that can be developed in the structure as set forth in Section 1630A.1.1.

E_v = the load effect resulting from the vertical component of the earthquake ground motion and is equal to * * * $0.5C_aID$ *applied* to the dead load effect, D, for Strength Design, and may be taken as zero for Allowable Stress Design.

> ***EXCEPTION:*** *See Section 1632A.2 for the definition of E_v for nonstructural components.*

Ω_o = the seismic force amplification factor that is required to account for structural overstrength, as set forth in Section 1630A.3.1.

ρ = Reliability/Redundancy Factor as given by the following formula:

$$\rho = 2 - \frac{20}{r_{max}\sqrt{A_B}} \qquad (30A\text{-}3)$$

For **SI:**

$$\rho = 2 - \frac{6.1}{r_{max}\sqrt{A_B}}$$

WHERE:

r_{max} = the maximum element-story shear ratio. For a given direction of loading, the element-story shear ratio is the ratio of the design story shear in the most heavily loaded single element divided by the total design story shear. For any given Story Level i, the element-story shear ratio is denoted as r_i. The maximum element-story shear ratio r_{max} is defined as the largest of the element story shear ratios, r_i, which occurs in any of the story levels at or below the two-thirds height level of the building.

For braced frames, the value of r_i is equal to the maximum horizontal force component in a single brace element divided by the total story shear.

For moment frames, r_i shall be taken as the maximum of the sum of the shears in any two adjacent columns in a moment frame bay divided by the story shear. For columns common to two bays with moment-resisting connections on opposite sides at Level i in the direction under consideration, 70 percent of the shear in that column may be used in the column shear summation.

For shear walls, r_i shall be taken as the maximum value of the product of the wall shear multiplied by $10/l_w$ (For **SI:** $3.05/l_w$) and divided by the total story shear, where l_w is the length of the wall in feet (m). *[For OSHPD 1 & 4] The value of the ratio of $10/L_w$ need not be taken as greater than 1.0 for light-framed construction.*

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

ρ shall not be taken less than 1.0 and need not be greater than 1.5, and A_B is the ground floor area of the structure in square feet (m^2). For special moment-resisting frames, except when used in dual systems, ρ shall not exceed 1.25. The number of bays of special moment-resisting frames shall be increased to reduce r, such that ρ is less than or equal to 1.25.

> **EXCEPTION:** A_B may be taken as the average floor area in the upper setback portion of the building where a larger base area exists at the ground floor.

When calculating drift, or when the structure is located in Seismic Zone 0, 1 or 2, ρ shall be taken equal to 1.

The ground motion producing lateral response and design seismic forces may be assumed to act nonconcurrently in the direction of each principal axis of the structure, except as required by Section 1633A.1.

Seismic dead load, W, is the total dead load *located above the base* and applicable portions of other loads listed below.

1. In storage and warehouse occupancies, a minimum of 25 percent of the floor live load shall be applicable.

2. Where a partition load is used in the floor design, a load of not less than 10 psf (0.48 kN/m^2) shall be included.

3. Design snow loads of 30 psf (1.44 kN/m^2) or less need not be included. Where design snow loads exceed 30 psf (1.44 kN/m^2), the design snow load shall be included, but may be reduced up to 75 percent where consideration of siting, configuration and load duration warrant when approved by the *enforcement agency.*

4. Total weight of permanent equipment shall be included.

5. Where buildings provide lateral support for walls retaining earth, and the exterior grades on opposite sides of the building differ by more than 6 feet (1829 mm), the load combination of the seismic increment of earth pressure due to earthquake acting on the higher side, as determined by a civil engineer qualified in soils engineering plus the difference in earth pressures shall be added to the lateral forces provided in this section.

1630A.1.2 Modeling requirements. The mathematical model of the physical structure shall include all elements of the lateral-force-resisting system. The model shall also include the stiffness and strength of elements, which are significant to the distribution of forces, and shall represent the spatial distribution of the mass and stiffness of the structure, *including diaphragm and foundation stiffness.* 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.

2. For steel moment frame systems, the contribution of panel zone deformations to overall story drift shall be included.

1630A.1.3 $P\Delta$ effects. The resulting member forces and moments and the story drifts induced by $P\Delta$ effects shall be considered in the evaluation of overall structural frame stability and shall be evaluated using the forces producing the displacements of Δ_S. $P\Delta$ need not be considered when the ratio of secondary moment to primary moment does not exceed 0.10; the ratio may be evaluated for any story as the product of the total dead, floor live and snow load, as required in Section 1612A, above the story times the seismic drift in that story divided by the product of the seismic shear in that story times the height of that story. In Seismic Zones 3 and 4, $P\Delta$ need not be considered when the story drift ratio does not exceed $0.02/R$.

1630A.2 Static Force Procedure.

1630A.2.1 Design base shear. The total design base shear in a given direction shall be determined from the following formula:

$$V = \frac{C_v I}{R T} W \tag{30A-4}$$

The total design base shear need not exceed the following:

$$V = \frac{2.5 C_a I}{R} W \tag{30A-5}$$

The total design base shear shall not be less than the following:

$$V = 0.11 C_a I W \tag{30A-6}$$

In addition, for Seismic Zone 4, the total base shear shall also not be less than the following:

$$V = \frac{0.8 Z N_v I}{R} W \tag{30A-7}$$

1630A.2.2 Structure period. The value of T shall be determined from one of the following methods:

1. **Method A:** For all buildings, the value T may be approximated from the following formula:

$$T_A = \frac{C_t (h_n)^{3/4}}{I N_v} \tag{30A-8}$$

WHERE:

C_t = 0.035 (0.0853) for steel moment-resisting frames.

C_t = 0.030 (0.0731) for reinforced concrete moment-resisting frames and eccentrically braced frames.

C_t = 0.020 (0.0488) for all other buildings.

Alternatively, the value of C_t for structures with concrete or masonry shear walls may be taken as $0.1/\sqrt{A_c}$ (For **SI:** $0.0743/\sqrt{A_c}$ for A_c in m^2).

The value of A_c shall be determined from the following formula:

$$A_c = \Sigma A_e \left[0.2 + (D_e/h_n)^2\right] \tag{30A-9}$$

The value of D_e/h_n used in Formula (30A-9) shall not exceed 0.9.

The value of T computed by Method A shall not be taken as larger than the value of T given by Method B. If Method B is not used to compute T, then the value of T shall be taken as:

$$\frac{T_A}{I N_v}$$

2. **Method B:** The fundamental period T may be calculated using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The analysis shall be in accordance with the requirements of Section 1630A.1.2. The value of T from Method B shall not exceed a value 30 percent greater than the value of T obtained from Method A in Seismic Zone 4, and 40 percent in Seismic Zones 1, 2 and 3.

The fundamental period T may be computed by using the following formula:

$$T = 2\pi \sqrt{\left(\sum_{i=1}^{n} w_i \delta_i^{\,2}\right) \div \left(g \sum_{i=1}^{n} f_i \delta_i\right)} \tag{30A-10}$$

The values of f_i represent any lateral force distributed approximately in accordance with the principles of Formulas (30A-13), (30A-14) and (30A-15) or any other rational distribution. The elastic deflections, δ_i, shall be calculated using the applied lateral forces, f_i.

1630A.2.3 Simplified design base shear. *[Not adopted by OSHPD]*

1630A.2.3.1 General. Structures conforming to the requirements of Section 1629A.8.2 may be designed using this procedure.

1630A.2.3.2 Base shear. The total design base shear in a given direction shall be determined from the following formula:

$$V = \frac{3.0 C_a}{R} W \tag{30A-11}$$

where the value of C_a shall be based on Table 16A-Q for the soil profile type. When the soil properties are not known in sufficient detail to determine the soil profile type, Type S_D shall be used in Seismic Zones 3 and 4, and Type S_E shall be used in Seismic Zones 1, 2A and 2B. In Seismic Zone 4, the Near-Source Factor, N_a, need not be greater than 1.3 if none of the following structural irregularities are present: Type 1, 4 or 5 of Table 16A-L, or Type 1 or 4 of Table 16A-M.

1630A.2.3.3 Vertical distribution. The forces at each level shall be calculated using the following formula:

$$F_x = \frac{3.0 C_a}{R} w_i \tag{30A-12}$$

where the value of C_a shall be determined in Section 1630A.2.3.2.

1630A.2.3.4 Applicability. Sections 1630A.1.2, 1630A.1.3, 1630A.2.1, 1630A.2.2, 1630A.5, 1630A.9, 1630A.10 and 1631A shall not apply when using the simplified procedure.

> **EXCEPTION:** For buildings with relatively flexible structural systems, the building official may require consideration of $P\Delta$ effects and drift in accordance with Sections 1630A.1.3, 1630A.9 and 1630A.10. Δ_s shall be prepared using design seismic forces from Section 1630A.2.3.2.

Where used, Δ_M shall be taken equal to 0.01 times the story height of all stories. In Section 1633A.2.9, Formula (33A-1) shall read $F_{px} = \frac{3.0 C_a}{R} w_{px}$ and need not exceed 1.0 C_a w_{px}, but shall not be less than 0.5 C_a w_{px}. R and Ω_o shall be taken from Table 16A-N.

1630A.3 Determination of Seismic Factors.

1630A.3.1 Determination of Ω_o. For specific elements of the structure, as specifically identified in this code, the minimum design strength shall be the product of the seismic force overstrength factor Ω_o and the design seismic forces set forth in Section 1630A. For both Allowable Stress Design and Strength Design, the Seismic Force Overstrength Factor, Ω_o, shall be taken from Table 16A-N.

1630A.3.2 Determination of R. The notation R shall be taken from Table 16A-N.

1630A.4 Combinations of Structural Systems.

1630A.4.1 General. Where combinations of structural systems are incorporated into the same structure, the requirements of this section shall be satisfied.

1630A.4.2 Vertical combinations. The value of R used in the design of any story shall be less than or equal to the value of R used in the given direction for the story above.

> **EXCEPTION:** This requirement need not be applied to a story where the dead weight above that story is less than 10 percent of the total dead weight of the structure.

Structures may be designed using the procedures of this section under the following conditions:

1. The entire structure is designed using the lowest R of the lateral-force-resisting systems used, or

2. Two-stage analysis procedures may be used *providing the structure complies with the following:*

   2.1 The flexible upper portion shall be designed as a separate structure, supported laterally by the rigid lower portion, using the appropriate values of R and ρ.

   2.2 The rigid lower portion shall be designed as a separate structure using the appropriate values of R and ρ. The reactions from the upper portion shall be those determined from the analysis of the upper portion amplified by the ratio of the (R/ρ) of the upper portion over (R/ρ) of the lower portion. *This ratio shall not be less than one.*

   2.3 Where design of elements of the upper portion are governed by special seismic load combinations, the special loads shall be considered in the design of the lower portion.

   2.4 The lower portion shall have a stiffness at least 10 times the upper portion.

   2.5 The period of the entire structure shall not be greater than 1.1 times the period of the upper portion considered as a separate structure fixed at the base.

   2.6 The detailing requirements required by the lateral system of the upper portion shall be used for structural components common to the structural system of lower portion.

   2.7 If the separate models are used to design the upper and lower portions, the model boundary conditions of the upper portion shall be compatible with actual strength and stiffness of the supporting elements of the lower portion.

1630A.4.3 Combinations along different axes. In Seismic Zones 3 and 4 where a structure has a bearing wall system in only one direction, the value of R used for design in the orthogonal direction shall not be greater than that used for the bearing wall system.

Any combination of bearing wall systems, building frame systems, dual systems or moment-resisting frame systems may be used to resist seismic forces in structures less than 160 feet (48 768 mm) in height. Only combinations of dual systems and special moment-resisting frames shall be used to resist seismic forces in structures exceeding 160 feet (48 768 mm) in height in Seismic Zones 3 and 4.

1630A.4.4 Combinations along the same axis. For other than dual systems and shear wall-frame interactive systems in Seismic Zones 0 and 1, where a combination of different structural systems is utilized to resist lateral forces in the same direction, the value of R used for design in that direction shall not be greater than the least value for any of the systems utilized in that same direction.

1630A.5 Vertical Distribution of Force. The total force shall be distributed over the height of the structure in conformance with Formulas (30A-13), (30A-14) and (30A-15) in the absence of a more rigorous procedure.

$$V = F_t + \sum_{i=1}^{n} F_i \tag{30A-13}$$

The concentrated force F_t at the top, which is in addition to F_n, shall be determined from the formula:

$$F_t = 0.07\ T\ V \tag{30A-14}$$

The value of T used for the purpose of calculating F_t shall be the period that corresponds with the design base shear as computed using Formula (30A-4). F_t need not exceed $0.25V$ and may be considered as zero where T is 0.7 second or less. The remaining portion of the base shear shall be distributed over the height of the structure, including Level n, according to the following formula:

$$F_x = \frac{(V - F_t)\, w_x\, h_x}{\sum_{i=1}^{n} w_i\, h_i} \tag{30A-15}$$

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution at that level. Structural displacements and design seismic forces shall be calculated as the effect of forces F_x and F_t applied at the appropriate levels above the base.

1630A.6 Horizontal Distribution of Shear. The design story shear, V_x, in any story is the sum of the forces F_t and F_x above that story. V_x shall be distributed to the various elements of the vertical lateral-force-resisting system in proportion to their rigidities, considering the rigidity of the diaphragm. See Section 1633A.2.4 for rigid elements that are not intended to be part of the lateral-force-resisting systems.

Where diaphragms are not flexible, the mass at each level shall be assumed to be displaced from the calculated center of mass in each direction a distance equal to 5 percent of the building dimension at that level perpendicular to the direction of the force under consideration. The effect of this displacement on the story shear distribution shall be considered.

Diaphragms shall be considered flexible for the purposes of distribution of story shear and torsional moment when the maximum lateral deformation of the diaphragm is more than two times the average story drift of the associated story. This may be determined by comparing the computed midpoint in-plane deflection of the diaphragm itself under lateral load with the story drift of adjoining vertical-resisting elements under equivalent tributary lateral load.

1630A.7 Horizontal Torsional Moments. Provisions shall be made for the increased shears resulting from horizontal torsion where diaphragms are not flexible. The most severe load combination for each element shall be considered for design.

The torsional design moment at a given story shall be the moment resulting from eccentricities between applied design lateral forces at levels above that story and the vertical-resisting elements in that story plus an accidental torsion.

The accidental torsional moment shall be determined by assuming the mass is displaced as required by Section 1630A.6.

Where torsional irregularity exists, as defined in Table 16A-M, the effects shall be accounted for by increasing the accidental torsion at each level by an amplification factor, A_x, determined from the following formula:

$$A_x = \left[\frac{\Delta_{max}}{1.2\Delta_{avg}}\right]^2 \tag{30A-16}$$

WHERE:

Δ_{avg} = the average of the *interstory drift* at the extreme points of the structure at Level x.

Δ_{max} = the maximum *interstory drift* at Level x.

The value of A_x need not exceed 3.0.

1630A.8 Overturning.

1630A.8.1 General. Every structure shall be designed to resist the overturning effects caused by earthquake forces specified in Section 1630A.5. At any level, the overturning moments to be resisted shall be determined using those seismic forces (F_t and F_x) that act on levels above the level under consideration. At any level, the incremental changes of the design overturning moment shall be distributed to the various resisting elements in the manner prescribed in Section 1630A.6. Overturning effects on every element shall be carried down to the foundation. See Sections 1612A and 1633A for combining gravity and seismic forces.

1630A.8.2 Elements supporting discontinuous systems.

1630A.8.2.1 General. Where any portion of the lateral-load-resisting system is discontinuous, such as for vertical irregularity Type 4 in Table 16A-L or plan irregularity Type 4 in Table 16A-M, concrete, masonry, steel and wood elements supporting such discontinuous systems shall have the design strength to resist the combination loads resulting from the special seismic load combinations of Section 1612A.4.

EXCEPTIONS: 1. The quantity E_m in Section 1612A.4 need not exceed the maximum force that can be transferred to the element by the lateral-force-resisting system.

2. Concrete slabs supporting light-frame wood shear wall systems or light-frame steel and wood structural panel shear wall systems.

For Allowable Stress Design, the design strength may be determined using an allowable stress increase of 1.7 and a resistance factor, ϕ, of 1.0. This increase shall not be combined with the one-third stress increase permitted by Section 1612A.3, but may be combined with the duration of load increase permitted in Chapter 23A, Division III.

1630A.8.2.2 Detailing requirements in Seismic Zones 3 and 4. In Seismic Zones 3 and 4, elements supporting discontinuous systems shall meet the following detailing or member limitations:

1. Reinforced concrete elements designed primarily as axial-load members shall comply with Section 1921A.4.4.5.

2. Reinforced concrete elements designed primarily as flexural members and supporting other than light-frame wood shear wall systems or light-frame steel and wood structural panel shear wall systems shall comply with Sections 1921A.3.2 and 1921A.3.3. Strength computations for portions of slabs designed as supporting elements shall include only those portions of the slab that comply with the requirements of these sections.

3. Masonry elements designed primarily as axial-load carrying members shall comply with Sections 2106A.1.12.4, Item 1, and 2108A.2.6.2.6.

4. Masonry elements designed primarily as flexural members shall comply with Section 2108A.2.6.2.5.

5. Steel elements designed primarily as axial-load members shall comply with Sections 2213A.5.2 and 2213A.5.3.

6. Steel elements designed primarily as flexural members or trusses shall have bracing for both top and bottom beam flanges or chords at the location of the support of the discontinuous system and shall comply with the requirements of Section 2213A.7.1.3.

7. Wood elements designed primarily as flexural members shall be provided with lateral bracing or solid blocking at each end of the element and at the connection location(s) of the discontinuous system.

CA **1630A.8.3 At foundation.** See Sections 1629A.1, *1633A.2.12* and 1809A.4 for overturning moments to be resisted at the foundation soil interface.

1630A.9 Drift. Drift or horizontal displacements of the structure shall be computed where required by this code. For both Allowable Stress Design and Strength Design, the Maximum Inelastic Response Displacement, Δ_M, of the structure caused by the Design Basis Ground Motion shall be determined in accordance with this section. The drifts corresponding to the design seismic forces of Section 1630A.2.1, Δ_S, shall be determined in accordance with Section 1630A.9.1. To determine Δ_M, these drifts shall be amplified in accordance with Section 1630A.9.2.

1630A.9.1 Determination of Δ_S. A static, elastic analysis of the lateral force-resisting system shall be prepared using the design seismic forces from Section 1630A.2.1. Alternatively, dynamic analysis may be performed in accordance with Section 1631A. Where Allowable Stress Design is used and where drift is being computed, the load combinations of Section *1612A.2* shall be used. The mathematical model shall comply with Section 1630A.1.2. The resulting deformations, denoted as Δ_S, shall be determined at all critical locations in the structure. Calculated drift shall include translational and torsional deflections. CA

1630A.9.2 Determination of Δ_M. The Maximum Inelastic Response Displacement, Δ_M, shall be computed as follows:

$$\Delta_M = 0.7\ R\Delta_S \qquad (30A\text{-}17)$$

EXCEPTION: Alternatively, Δ_M may be computed by nonlinear time history analysis in accordance with Section 1631A.6.

The analysis used to determine the Maximum Inelastic Response Displacement Δ_M shall consider $P\Delta$ effects.

1630A.10 Story Drift Limitation.

1630A.10.1 General. Story drifts shall be computed using the Maximum Inelastic Response Displacement, Δ_M.

1630A.10.2 Calculated. Calculated story drift using Δ_M shall not exceed 0.025 times the story height for structures having a fundamental period of less than 0.7 second. For structures having a fundamental period of 0.7 second or greater, the calculated story drift shall not exceed 0.020 times the story height.

EXCEPTIONS: 1. These drift limits may be exceeded when it is demonstrated that greater drift can be tolerated by both structural elements and nonstructural elements that could affect life safety *or continued operation*. The drift used in this assessment shall be based upon the Maximum Inelastic Response Displacement, Δ_M. CAC

2. There shall be no drift limit in single-story steel-framed structures classified as Groups B, F and S Occupancies or Group H, Division 4 or 5 Occupancies. In Groups B, F and S Occupancies, the primary use shall be limited to storage, factories or workshops. Minor accessory uses shall be allowed in accordance with the provisions of Section 302. Structures on which this exception is used shall not have equipment attached to the structural frame or shall have such equipment detailed to accommodate the additional drift. Walls that are laterally supported by the steel frame shall be designed to accommodate the drift in accordance with Section 1633A.2.4.

1630A.10.3 Limitations. The design lateral forces used to determine the calculated drift may disregard the limitations of Formula (30A-6) and may be based on the period determined from Formula (30A-10) neglecting the 30 or 40 percent limitations of Section 1630A.2.2, Item 2.

1630A.11 Vertical Component. The following requirements apply in Seismic Zones 3 and 4 only. Horizontal cantilever components shall be designed for a net upward force of $0.7C_aIW_p$.

In addition to all other applicable load combinations, horizontal prestressed components shall be designed using not more than 50 percent of the dead load for the gravity load, alone or in combination with the lateral force effects.

SECTION 1631A — DYNAMIC ANALYSIS PROCEDURES

1631A.1 General. Dynamic analyses procedures, when used, shall conform to the criteria established in this section. The analysis shall be based on an appropriate ground motion representation and shall be performed using accepted principles of dynamics. Structures that are designed in accordance with this section shall comply with all other applicable requirements of these provisions.

1631A.2 Ground Motion. *The dynamic analysis shall be based on the maximum probable ground motions prescribed in the Supplemental Ground-response Reports described in Section 1637A.2. The maximum probable* ground motion representation shall, as a minimum, be one having a 10-percent probability of being exceeded in 50 years, shall not be reduced by the quantity R and may be one of the following:

1. An elastic design response spectrum constructed in accordance with Figure 16A-3, using the values of C_a and C_v consistent with the specific site. The design acceleration ordinates shall be multiplied by the acceleration of gravity, 386.4 in./sec.^{2} (9.815 m/sec.^{2}). *This spectrum may be used for regular structures only.*

2. A site-specific elastic design response spectrum based on the geologic, tectonic, seismologic and soil characteristics associated with the specific site. The spectrum shall be developed for a damping ratio of 0.05, unless a different value is shown to be consistent with the anticipated structural behavior at the intensity of shaking established for the site. *The site-specific response spectra shall be used for irregular structures and for all structures located on Soil Profile Type S_F.*

3. Ground motion time histories developed for the specific site shall be representative of actual earthquake motions. Response spectra from time histories, either individually or in combination, shall approximate the site design spectrum conforming to Section 1631A.2, Item 2.

4. For structures on Soil Profile Type S_F, the following requirements shall apply when required by Section 1629A.8.4, Item 4:

   4.1 The ground motion representation shall be developed in accordance with Items 2 and 3.

   4.2 Possible amplification of building response due to the effects of soil-structure interaction and lengthening of building period caused by inelastic behavior shall be considered.

1631A.2.1 Vertical ground motion. The vertical component of ground motion may be defined by scaling corresponding horizontal accelerations by a factor of two-thirds. Alternative factors may be used when substantiated by site-specific data. Where the Near Source Factor, N_a, is greater than 1.0, site-specific vertical response spectra shall be used in lieu of the factor of two-thirds.

1631A.2.2 Upper bound earthquake. *The "upper bound earthquake" ground motion is defined as the motion having a 10 percent probability of being exceeded in a 100-year period or maximum level of motion which may ever be expected at the building site within the known geological framework. Where required by other sections of this code, structures shall be designed to sustain the upperbound earthquake motion, including the $P\Delta$ effects, without forming a story collapse mechanism along any frameline. Such structures shall have sufficient ductility and strength to undergo the displacement caused by the upper bound earthquake motion without collapse. For irregular or unusual structures located in an area having large site-specific ground motion, criteria as determined by the project architect or structural engineer and approved by the enforcement agency will be required to demonstrate safety against collapse from the upper bound earthquake motion.*

1631A.3 Mathematical Model. A mathematical model of the physical structure shall represent the spatial distribution of the mass and stiffness of the structure to an extent that is adequate for the calculation of the significant features of its dynamic response. A three-dimensional model shall be used for the dynamic analysis of structures with highly irregular plan configurations such as those having a plan irregularity defined in Table 16A-M and having a rigid or semirigid diaphragm. The stiffness properties used in the analysis and general mathematical modeling shall be in accordance with Section 1630A.1.2. *The mathematical model of buildings with diaphragm discontinuities, as defined in Table 16A-M, Item 3, shall explicitly include the effect of diaphragm stiffness.*

1631A.4 Description of Analysis Procedures.

1631A.4.1 Response spectrum analysis. An elastic dynamic analysis of a structure utilizing the peak dynamic response of all modes having a significant contribution to total structural response. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve which correspond to the modal periods. Maximum modal contributions are combined in a statistical manner to obtain an approximate total structural response.

1631A.4.2 Time-history analysis. An analysis of the dynamic response of a structure at each increment of time when the base is subjected to a specific ground motion time history.

1631A.5 Response Spectrum Analysis.

1631A.5.1 Response spectrum representation and interpretation of results. The ground motion representation shall be in accordance with Section 1631A.2. The corresponding response parameters, including forces, moments and displacements, shall be denoted as Elastic Response Parameters. Elastic Response Parameters may be reduced in accordance with Section 1631A.5.4.

1631A.5.2 Number of modes. The requirement of Section 1631A.4.1 that all significant modes be included may be satisfied by demonstrating that for the modes considered, at least 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction.

1631A.5.3 Combining modes. The peak member forces, displacements, story forces, story shears and base reactions for each mode shall be combined by recognized methods. When three-dimensional models are used for analysis, modal interaction effects shall be considered when combining modal maxima.

1631A.5.4 Reduction of Elastic Response Parameters for design. Elastic Response Parameters may be reduced for purposes of design in accordance with the following items, with the limitation that in no case shall the Elastic Response Parameters be reduced such that the corresponding design base shear is less than the Elastic Response Base Shear divided by the value of R.

1. For all regular structures, * * * Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than *100* percent of the base shear determined in accordance with Section 1630A.2.

2. *Not adopted by DSA/SS and OSHPD.*

3. *[For OSHPD 1 & 4 and DSA/SS] For irregular structures with vertical irregularity Types 1a, 2 or 5, as defined in Table 16A-L, or irregular structures with plan irregularity Type 1B, as defined in Table 16A-M, Elastic Response Parameters, may be reduced such that the corresponding design base shear is not less*

than 125 percent of the base shear determined in accordance with Section 1630A.2.

> ***EXCEPTION:*** *The Elastic Response Parameters for structures with Vertical Irregularity Types 1a or 2, as defined in Table 16A-L, or plan irregularity Type 1b; as defined in Table 16A-M, may be reduced such that the corresponding design base shear is not less than 100 percent of the base shear determined in accordance with Section 1630A.2, if no interstory drift ratio under design lateral load is greater than 130 percent of the interstory drift ratio of the story immediately above. Torsional effects need not be considered in the calculation of story drifts for the purposes of this determination. The story drift ratio relationships for the top two stories of the structures are not required to be evaluated.*

4. For all *other* structures, ∗ ∗ ∗ Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than *100* percent of the base shear determined in accordance with Section 1630A.2.

The reduced design seismic forces shall be used for design in accordance with Section 1612A.

1631A.5.5 Directional effects. Directional effects for horizontal ground motion shall conform to the requirements of Section 1630A.1. The effects of vertical ground motions on horizontal cantilevers and prestressed elements shall be considered in accordance with Section 1630A.11. Alternately, vertical seismic response may be determined by dynamic response methods; in no case shall the response used for design be less than that obtained by the static method.

1631A.5.6 Torsion. The analysis shall account for torsional effects, including accidental torsional effects as prescribed in Section 1630A.7. Where three-dimensional models are used for analysis, effects of accidental torsion shall be accounted for by appropriate adjustments in the model such as adjustment of mass locations, or by equivalent static procedures such as provided in Section 1630A.6.

1631A.5.7 Dual systems. Where the lateral forces are resisted by a dual system as defined in Section 1629A.6.5, the combined system shall be capable of resisting the base shear determined in accordance with this section. The moment-resisting frame shall conform to Section 1629A.6.5, Item 2, and may be analyzed using either the procedures of Section 1630A.5 or those of Section 1631A.5.

1631A.6 Time-history Analysis.

1631A.6.1 Time history. Time-history analysis shall be performed with pairs of appropriate horizontal ground-motion time-history components that shall be selected and scaled from not less than three recorded events. Appropriate time histories shall have magnitudes, fault distances and source mechanisms that are consistent with those that control the design-basis earthquake (or maximum capable earthquake). Where three appropriate recorded ground-motion time-history pairs are not available, appropriate simulated ground-motion time-history pairs may be used to make up the total number required. For each pair of horizontal ground-motion components, the square root of the sum of the squares (SRSS) of the 5 percent-damped site-specific spectrum of the scaled horizontal components shall be constructed. The motions shall be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5 percent-damped spectrum of the design-basis earthquake for periods from $0.2T$ second to $1.5T$ seconds. Each pair of time histories shall be applied simultaneously to the model considering torsional effects.

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

1631A.6.2 Elastic time-history analysis. Elastic time history shall conform to Sections 1631A.1, 1631A.2, 1631A.3, 1631A.5.2, 1631A.5.4, 1631A.5.5, 1631A.5.6, 1631A.5.7 and 1631A.6.1. Response parameters from elastic time-history analysis shall be denoted as Elastic Response Parameters. All elements shall be designed using Strength Design. Elastic Response Parameters may be scaled in accordance with Section 1631A.5.4.

1631A.6.3 Nonlinear time-history analysis.

1631A.6.3.1 Nonlinear time history. Nonlinear time-history analysis shall meet the requirements of Section 1629A.10, and time histories shall be developed and results determined in accordance with the requirements of Section 1631A.6.1. Capacities and characteristics of nonlinear elements shall be modeled consistent with test data or substantiated analysis, considering the Importance Factor. The maximum inelastic response displacement shall not be reduced and shall comply with Section 1630A.10.

1631A.6.3.2 Design review. *[Not adopted by OSHPD]* When nonlinear time-history analysis is used to justify a structural design, a design review of the lateral-force-resisting system shall be performed by an independent engineering team, including persons licensed in the appropriate disciplines and experienced in seismic analysis methods. The lateral-force-resisting system design review shall include, but not be limited to, the following:

1. Reviewing the development of site-specific spectra and ground-motion time histories.

2. Reviewing the preliminary design of the lateral-force-resisting system.

3. Reviewing the final design of the lateral-force-resisting system and all supporting analyses.

The engineer of record shall submit with the plans and calculations a statement by all members of the engineering team doing the review stating that the above review has been performed.

SECTION 1632A — LATERAL FORCE ON ELEMENTS OF STRUCTURES, NONSTRUCTURAL COMPONENTS AND EQUIPMENT SUPPORTED BY STRUCTURES

1632A.1 General. Elements of structures and their attachments, permanent nonstructural components and their attachments, and the attachments for permanent equipment supported by a structure shall be designed to resist the total design seismic forces prescribed in Section 1632A.2.

Attachments shall include anchorages and required bracing *of components, including legs, platforms, frames, skids or other elements providing gravity or lateral support to the equipment or component. Welded, bolted or other intermittent connections, such as inserts for anchorage of nonstructural components, shall not be allowed the one-third increase in allowable stresses permitted in Section 1612A.3.2.* Friction resulting from gravity loads shall not be considered to provide resistance to seismic forces.

When the structural failure of the lateral-force-resisting systems of nonrigid equipment would cause a life hazard, such systems shall be designed to resist the seismic forces prescribed in Section 1632A.2.

When permissible design strengths and other acceptance criteria are not contained in or referenced by this code, such criteria shall be obtained from approved national standards subject to the approval of the *enforcement agency.*

1632A.2 Design for Total Lateral Force. The total design lateral seismic force, F_p, shall be determined from the following formula:

$$F_p = 4.0\, C_a\, I_p\, W_p \qquad (32A\text{-}1)$$

Alternatively, F_p may be calculated using the following formula:

$$F_p = \frac{a_p\, C_a\, I_p}{R_p} \left(1 + 2\frac{h_x}{h_r} \right) W_p \qquad (32A\text{-}2)$$

Except that:

$$F_p \text{ shall not be less than } 0.7 C_a I_p W_p \text{ and need not be more than } 4 C_a I_p W_p \qquad (32A\text{-}3)$$

WHERE:

h_x is the element or component attachment elevation with respect to grade. h_x shall not be taken less than 0.0. *The value of h_x/h_r need not exceed 1.0.*

h_r is the structure roof elevation with respect to grade.

a_p is the in-structure Component Amplification Factor that varies from 1.0 to 2.5.

A value for a_p shall be selected from Table 16A-O. Alternatively, this factor may be determined based on the dynamic properties or empirical data of the component and the structure that supports it. The value shall not be taken less than 1.0. *For elevator-related equipment, see Part 7, Title 24, California Code of Regulations in addition to Table 16A-O.*

R_p is the Component Response Modification Factor that shall be taken from Table 16A-O, except that R_p for anchorages shall equal 1.5 for shallow expansion anchor bolts, shallow chemical anchors or shallow cast-in-place anchors. Shallow anchors are those with an embedment length-to-diameter ratio of less than 8. When anchorage is constructed of nonductile materials, or by use of adhesive, R_p shall equal 1.0.

The design lateral forces determined using Formula (32A-1) or (32A-2) shall be distributed in proportion to the mass distribution of the element or component. *To determine the out-of-plane loading for elements such as walls or wall panels that have points of attachment at two or more different elevations, the following procedure may be used. For the vertical span of the element having a unit weight w_p between two successive attachment elevations h_x and h_{x+i} evaluate the force coefficients F_p and W_p at each of the two points, observing the minimum and maximum limits, and compute the average of the two values. The resulting average coefficient times the unit weight w_p provides the distributed seismic load for the span between the attachment points, and this load may be extended to the top of any wall parapet above the roof attachment point at h_r.*

Forces determined using Formula (32A-1) or (32A-2) shall be used to design members and connections that transfer these forces to the seismic-resisting systems. Members and connection design shall use the load combinations and factors specified in Section 1612A.2 or 1612A.3. *The vertical earthquake effect E_v, shall be applied simultaneously with the horizontal earthquake effect E_h. E_v shall be taken as $0.5C_aIW_p$ for Strength Design and $0.35C_aIW_p$ for Allowable Stress Design. E_v shall be applied to the dead load effect D, in a manner that produces the most severe member and connection demands.* The Reliability/Redundancy Factor, ρ, may be taken equal to 1.0. *A continuous load path of sufficient strength and stiffness between the component and the supporting structure shall be verified. Local elements of the supporting structure shall be verified for the component loads where they control the design of the elements or their connections. The component loads shall be those determined using Formula (32A-1) or (32A-2), except that modifications to R_p due to anchorage conditions need not be considered.*

For applicable forces and Component Response Modification Factors in connectors for exterior panels and diaphragms, refer to Sections 1633A.2.4, 1633A.2.8 and 1633A.2.9.

Forces shall be applied in the horizontal directions, which result in the most critical loadings for design.

The design and detailing of anchorages or restraints for architectural components and mechanical and electrical systems shall accommodate the structure drifts as computed in accordance with Section 1633A.2.4 or determined by the wind forces given in Division III—Wind Design multiplied by 2.

1632A.3 Specifying Lateral Forces. Design specifications for equipment shall either specify the design lateral forces prescribed herein or reference these provisions.

1632A.4 Relative Motion of Equipment Attachments. For equipment in Categories 1 and 2 buildings as defined in Table 16A-K, the lateral-force design shall consider the effects of relative motion of the points of attachment to the structure, using the drift based upon Δ_M.

1632A.5 Alternative Designs. Where an approved national standard or approved physical test data provide a basis for the earthquake-resistant design of a particular type of equipment or other nonstructural component, such a standard or data may be accepted as a basis for design of the items with the following limitations:

1. These provisions shall provide minimum values for the design of the anchorage and the members and connections that transfer the forces to the seismic-resisting system.

2. The force, F_p, and the overturning moment used in the design of the nonstructural component shall not be less than 80 percent of the values that would be obtained using these provisions.

1632A.6 HVAC Ductwork, Plumbing/Piping and Conduit Systems. *All pipes, duct and conduit shall be braced to resist the forces prescribed in Section 1632A.2. Ductwork shall be constructed in accordance with provisions contained in Part 4, Title 24, California Mechanical Code. Pipes and their connections constructed of ductile materials (copper, ductile iron, steel, or aluminum with brazed or welded connections) shall have brace spacing not exceeding that specified in Section 1632A.5 or other standards approved by the enforcement agency. Pipes and their connections, constructed of nonductile materials (e.g., cast iron, no-hub pipe, and plastic), or with screwed connections shall have the brace spacing reduced to one-half of the spacing allowed for ductile material in accordance with Section 1632A.5 or other standards approved by the enforcement agency.*

Seismic restraints may be omitted for the following conditions, where flexible connections are provided between components or equipment and the associated ductwork, piping and conduit:

1. Fuel, medical gas, and vacuum piping less than 1 inch (25 mm) inside diameter.

2. All other piping less than 2.5 inches (64 mm) diameter, or

3. All piping suspended by individual hangers 12 inches (305 mm) or less in length from the top of the pipe to the bottom of the structural support for the hanger, where the hangers are detailed to avoid bending of the hangers and their connections, or

4. All electrical conduit less than 2.5 inches (64 mm) trade size, or

5. All rectangular air-handling ducts less than 6 square feet (0.56 m²) in cross-sectional area, or

6. All round air-handling ducts less than 28 inches (711 mm) in diameter, or

7. All ducts suspended by hangers 12 inches (305 mm) or less in length from the top of the duct to the bottom of the structural support for the hanger, where the hangers are detailed to avoid bending of the hangers and their connections.

Where lateral restraints are omitted, the piping, ducts or conduit shall be installed such that lateral motion of the piping, duct or conduit will not cause damaging impact with other systems or structural members, or loss of vertical support.

1632A.6.1 All trapeze assemblies supporting pipes, ducts and conduit shall be braced to resist the forces of Section 1632A.2, considering the total weight of the elements on the trapeze.

Pipes, ducts and conduit supported by a trapeze where none of those elements would individually be braced need not be braced if connections to the pipe/conduit/ductwork or directional changes do not restrict the movement of the trapeze. If this flexibility is not provided, bracing will be required when the aggregate weight of the pipes and conduit exceed 10 pounds/feet (146 N/m). The weight shall be determined assuming all pipes and conduit are filled with water.

SECTION 1633A — DETAILED SYSTEMS DESIGN REQUIREMENTS

1633A.1 General. All structural framing systems shall comply with the requirements of Section 1629A. Only the elements of the designated seismic-force-resisting system shall be used to resist design forces. The individual components shall be designed to resist the prescribed design seismic forces acting on them. The components shall also comply with the specific requirements for the material contained in Chapters 19A through 23A. In addition, such framing systems and components shall comply with the detailed system design requirements contained in Section 1633A.

All building components in Seismic Zones ∗ ∗ ∗ 3 and 4 shall be designed to resist the effects of the seismic forces prescribed herein and the effects of gravity loadings from dead, floor live and snow loads.

Consideration shall be given to design for uplift effects caused by seismic loads.

In Seismic Zones ∗ ∗ ∗ 3 and 4, provision shall be made for the effects of earthquake forces acting in a direction other than the principal axes in each of the following circumstances:

The structure has plan irregularity Type 5 as given in Table 16A-M.

The structure has plan irregularity Type 1 as given in Table 16A-M for both major axes.

A column of a structure forms part of two or more intersecting lateral-force-resisting systems.

EXCEPTION: If the axial load in the column due to seismic forces acting in either direction is less than 20 percent of the column axial load capacity.

The requirement that orthogonal effects be considered may be satisfied by designing such elements for 100 percent of the prescribed design seismic forces in one direction plus 30 percent of the prescribed design seismic forces in the perpendicular direction. The combination requiring the greater component strength shall be used for design. Alternatively, the effects of the two orthogonal directions may be combined on a square root of the sum of the squares (SRSS) basis. When the SRSS method of combining directional effects is used, each term computed shall be assigned the sign that will result in the most conservative result.

1633A.2 Structural Framing Systems.

1633A.2.1 General. Four types of general building framing systems defined in Section 1629A.6 are recognized in these provisions and shown in Table 16A-N. Each type is subdivided by the types of vertical elements used to resist lateral seismic forces. Special framing requirements are given in this section and in Chapters 19A through 23A.

1633A.2.2 Detailing for combinations of systems. For components common to different structural systems, the more restrictive detailing requirements shall be used.

1633A.2.3 Connections. Connections that resist design seismic forces shall be designed and detailed on the drawings.

1633A.2.4 Deformation compatibility. All structural framing elements and their connections, not required by design to be part of the lateral-force-resisting system, shall be designed and/or detailed to be adequate to maintain support of design dead plus live loads when subjected to the expected deformations caused by seismic forces. $P\Delta$ effects on such elements shall be considered. Expected deformations shall be determined as the greater of the Maximum Inelastic Response Displacement, Δ_M, considering $P\Delta$ effects determined in accordance with Section 1630A.9.2 or the deformation induced by a story drift of 0.0025 times the story height. When computing expected deformations, the stiffening effect of those elements not part of the lateral-force-resisting system shall be neglected.

For elements not part of the lateral-force-resisting system, the forces induced by the expected deformation may be considered as ultimate or factored forces. When computing the forces induced by expected deformations, the restraining effect of adjoining rigid structures and nonstructural elements shall be considered and a rational value of member and restraint stiffness shall be used. Inelastic deformations of members and connections may be considered in the evaluation, provided the assumed calculated capacities are consistent with member and connection design and detailing.

For concrete and masonry elements that are part of the lateral-force-resisting system, the assumed flexural and shear stiffness properties shall not exceed one half of the gross section properties unless a rational cracked-section analysis is performed. Additional deformations that may result from foundation flexibility and diaphragm deflections shall be considered. For concrete elements not part of the lateral-force-resisting system, see Section 1921A.7.

1633A.2.4.1 Adjoining rigid elements. Moment-resisting frames and shear walls may be enclosed by or adjoined by more rigid elements, provided it can be shown that the participation or failure of the more rigid elements will not impair the vertical and lateral-load-resisting ability of the gravity load and lateral-force-resisting systems. The effects of adjoining rigid elements shall be considered when assessing whether a structure shall be designated regular or irregular in Section 1629A.5.1.

1633A.2.4.2 Exterior elements. Exterior nonbearing, nonshear wall panels or elements that are attached to or enclose the exterior shall be designed to resist the forces per Formula (32A-1) or (32A-2) and shall accommodate movements of the structure based on Δ_M and temperature changes. Such elements shall be supported by means of cast-in-place concrete or by mechanical connections and fasteners in accordance with the following provisions:

1. Connections and panel joints shall allow for a relative movement between stories of not less than two times story drift caused

by wind, the calculated story drift based on Δ_M or $^1/_2$ inch (12.7 mm), whichever is greater.

2. Connections to permit movement in the plane of the panel for story drift shall be sliding connections using slotted or oversize holes, connections that permit movement by bending of steel, or other connections providing equivalent sliding and ductility capacity.

3. Bodies of connections shall have sufficient ductility and rotation capacity to preclude fracture of the concrete or brittle failures at or near welds.

4. The body of the connection shall be designed for the force determined by Formula (32A-2), where R_p = 3.0 and a_p = 1.0.

5. All fasteners in the connecting system, such as bolts, inserts, welds and dowels, shall be designed for the forces determined by Formula (32A-2), where R_p = 1.0 and a_p = 1.0.

6. Fasteners embedded in concrete shall be attached to, or hooked around, reinforcing steel or otherwise terminated to effectively transfer forces to the reinforcing steel.

1633A.2.5 Ties and continuity. All parts of a structure shall be interconnected and the connections shall be capable of transmitting the seismic force induced by the parts being connected. As a minimum, any smaller portion of the building shall be tied to the remainder of the building with elements having at least a strength to resist 0.5 C_aI times the weight of the smaller portion.

A positive connection for resisting a horizontal force acting parallel to the member shall be provided for each beam, girder or truss. This force shall not be less than 0.5 C_aI times the dead plus live load.

1633A.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.

Collector elements, splices and their connections to resisting elements shall resist the forces determined in accordance with Formula (33A-1). In addition, collector elements, splices, and their connections to resisting elements shall have the design strength to resist the combined loads resulting from the special seismic load of Section 1612A.4.

> **EXCEPTION:** In structures, or portions thereof, braced entirely by light-frame wood shear walls or light-frame steel and wood structural panel shear wall systems, collector elements, splices and connections to resisting elements need only be designed to resist forces in accordance with Formula (33A-1).

The quantity E_m 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. For Allowable Stress Design, the design strength may be determined using an allowable stress increase of 1.7 and *for LRFD* a resistance factor, ϕ, of 1.0. This increase shall not be combined with the one-third stress increase permitted by Section 1612A.3, but may be combined with the duration of load increase permitted in Division III of Chapter 23A.

1633A.2.7 Concrete frames. Concrete frames required by design to be part of the lateral-force-resisting system shall conform to the following:

1. In Seismic Zones 3 and 4 they shall be special moment-resisting frames.

2. *Not adopted by the State of California.*

1633A.2.8 Anchorage of concrete or masonry walls. Concrete or masonry walls shall be anchored to all floors and roofs that provide out-of-plane lateral support of the wall. The anchorage shall provide a positive direct connection between the wall and floor or roof construction capable of resisting the larger of the horizontal forces specified in this section and Sections 1611A.4 and 1632A. In addition, in Seismic Zones 3 and 4, diaphragm to wall anchorage using embedded straps shall have the straps attached to or hooked around the reinforcing steel or otherwise terminated to effectively transfer forces to the reinforcing steel. Requirements for developing anchorage forces in diaphragms are given in Section 1633A.2.9. Diaphragm deformation shall be considered in the design of the supported walls.

1633A.2.8.1 Out-of-plane wall anchorage to flexible diaphragms. This section shall apply in Seismic Zones 3 and 4 where flexible diaphragms, as defined in Section 1630A.6, provide lateral support for walls.

1. Elements of the wall anchorage system shall be designed for the forces specified in Section 1632A where R_p = 3.0 and a_p = 1.5.

In Seismic Zone 4, the value of F_p used for the design of the elements of the wall anchorage system shall not be less than 420 pounds per lineal foot (6.1 kN per lineal meter) of wall substituted for E.

See Section 1611A.4 for minimum design forces in other seismic zones.

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

3. 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 be that specified in Section 1633A.2.8.1, Item 1.

4. The strength design forces for steel elements of the wall anchorage system shall be 1.4 times the forces otherwise required by this section.

5. The strength design forces for wood elements of the wall anchorage system shall be 0.85 times the force otherwise required by this section and these wood elements shall have a minimum actual net thickness of $2^1/_2$ inches (63.5 mm).

1633A.2.9 Diaphragms.

1. 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 element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

2. Floor and roof diaphragms shall be designed to resist the forces determined in accordance with the following formula:

$$F_{px} = \frac{F_t + \sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad \text{(33A-1)}$$

The force F_{px} determined from Formula (33A-1) need not exceed $1.0C_aIw_{px}$, but shall not be less than $0.5C_aIw_{px}$.

When the diaphragm is required to transfer design seismic forces from the vertical-resisting elements above the diaphragm to other vertical-resisting elements below the diaphragm due to offset in the placement of the elements or to changes in stiffness in the vertical elements, these forces shall be added to those determined from Formula (33A-1).

3. Design seismic forces for flexible diaphragms providing lateral supports for walls or frames of masonry or concrete shall be determined using Formula (33A-1) based on the load determined in accordance with Section 1630A.2 using a R not exceeding 4.

4. Diaphragms supporting concrete or masonry walls shall have continuous ties or struts between diaphragm chords to distribute the anchorage forces specified in Section 1633A.2.8. Added chords of subdiaphragms may be used to form subdiaphragms to transmit the anchorage forces to the main continuous crossties. The maximum length-to-width ratio of the wood structural subdiaphragm shall be $2^1/_2$:1.

5. Where wood diaphragms are used to laterally support concrete or masonry walls, the anchorage shall conform to Section 1633A.2.8. In Seismic Zones ✳ ✳ ✳ 3 and 4, anchorage shall not be accomplished by use of toenails or nails subject to withdrawal, wood ledgers or framing shall not be used in cross-grain bending or cross-grain tension, and the continuous ties required by Item 4 shall be in addition to the diaphragm sheathing.

6. Connections of diaphragms to the vertical elements in structures in Seismic Zones 3 and 4, having a plan irregularity of Type 1, 2, 3 or 4 in Table 16A-M, shall be designed without considering either the one-third increase or the duration of load increase considered in allowable stresses for elements resisting earthquake forces.

7. In structures in Seismic Zones 3 and 4 having a plan irregularity of Type 2 in Table 16A-M, diaphragm chords and drag members shall be designed considering independent movement of the projecting wings of the structure. Each of these diaphragm elements shall be designed for the more severe of the following two assumptions:

Motion of the projecting wings in the same direction.

Motion of the projecting wings in opposing directions.

EXCEPTION: This requirement may be deemed satisfied if the procedures of Section 1631A in conjunction with a three-dimensional model have been used to determine the lateral seismic forces for design.

1633A.2.10 Framing below the base. The strength and stiffness of the framing between the base and the foundation shall not be less than that of the superstructure. The special detailing requirements of Chapters 19A and 22A, as appropriate, shall apply to columns supporting discontinuous lateral-force-resisting elements and to SMRF, ✳ ✳ ✳ EBF, STMF and MMRWF system elements below the base, which are required to transmit the forces resulting from lateral loads to the foundation.

1633A.2.11 Building separations. All structures shall be separated from adjoining structures. Separations shall allow for the displacement Δ_M. Adjacent buildings on the same property shall be separated by at least Δ_{MT} where

$$\Delta_{MT} = \sqrt{(\Delta_{M1})^2 + (\Delta_{M2})^2} \qquad (33A\text{-}2)$$

and Δ_{M1} and Δ_{M2} are the displacements of the adjacent buildings.

When a structure adjoins a property line not common to a public way, that structure shall also be set back from the property line by at least the displacement Δ_M of that structure.

EXCEPTION: Smaller separations or property line setbacks may be permitted when justified by rational analyses based on maximum expected ground motions.

1633A.2.12 Foundations and superstructure-to-foundation connections. *The foundation shall be capable of transmitting the design base shear and the overturning forces from the structure into the supporting soil.*

In addition, the foundation and the connection of the superstructure elements to the foundation shall have the strength to resist, in addition to gravity loads, the lesser of the following seismic loads:

1. The strength of the superstructure elements.

2. The maximum forces that would occur in the fully yielded structural system.

3. Ω_o times the forces in the superstructure elements due to the seismic forces as prescribed in this chapter.

EXCEPTIONS: *1. Where structures are designed using $R \leq 2.2$ such as for inverted pendulum-type structures.*

2. When it can be demonstrated that inelastic deformation of the foundation and superstructure-to-foundation connection will not result in a weak story or cause collapse of the structure.

3. Where the basic structural system as described in Table 16A-N consists of light-framed walls with shear panels.

Where moment resistance is assumed at the base of the superstructure elements, the rotation and flexural deformation of the foundation as well as deformation of the superstructure-to-foundation connection shall be considered in the drift and deformation compatibility analyses.

1633A.2.13 Requirements for elevators. *In addition to all of the requirements contained in Part 7, Title 24, California Code of Regulations, the design of elevators in schools and hospitals shall meet the following requirements.*

1633A.2.13.1 *The design of guide rail support-bracket fastenings and the supporting structural framing shall be in accordance with Section 3030 (k), Part 7, Title 24, using the weight of the counterweight or maximum weight of the car plus not more than 40 percent of its rated load. The seismic forces shall be assumed to be distributed one third to the top guiding members and two thirds to the bottom guiding members of cars and counterweights, unless other substantiating data are provided. Minimum seismic forces shall be 0.5g acting in any horizontal direction, using allowable stress design.*

Retainer plates are required for both car and counterweight, designed in accordance with Section 3032 (c), Part 7, Title 24, California Code of Regulations. Retainer plates are required at the top and bottom of the car and counterweight, except where safety devices acceptable to the enforcement agency are provided which meet all requirements of the retainer plates, including full engagement of the machined portion of the rail. The design of the car and counterweight guide rails for seismic forces shall be based on the following requirements:

1. The lateral forces using allowable stress design shall be based on horizontal acceleration of 0.5g for all buildings.

2. W_p shall equal the weight of the counterweight or the maximum weight of the car plus not less than 40 percent of its rated load.

3. With the car or counterweight located in the most adverse position, the stress in the rail shall not exceed the limitations specified in these regulations, nor shall the deflection of the rail relative to its supports exceed the deflection listed below:

RAIL SIZE (weight per foot of length, pounds)	*WIDTH OF MACHINED SURFACE (inches)*	*ALLOWABLE RAIL DEFLECTION (inches)*
8	$1^1/_4$	0.20
11	$1^1/_2$	0.30
12	$1^3/_4$	0.40
15	$1^{31}/_{32}$	0.50
$18^1/_2$	$1^{31}/_{32}$	0.50
$22^1/_2$	2	0.50
30	$2^1/_4$	0.50

*For **SI:** 1 inch = 25 mm, 1 foot = 305 mm.*

NOTE: *Deflection limitations are given to maintain a consistent factor of safety against disengagement of retainer plates from the guide rails during an earthquake.*

4. Where guide rails are continuous over supports and rail joints are within 2 feet (610 mm) of their supporting brackets, a simple span may be assumed.

5. The use of spreader brackets is allowed.

6. Cab stabilizers and counterweight frames shall be designed to withstand a lateral load equal to 0.5g using allowable stress design.

SECTION 1634A — NONBUILDING STRUCTURES

1634A.1 General.

1634A.1.1 Scope. Nonbuilding structures include all self-supporting structures other than buildings that carry gravity loads and resist the effects of earthquakes. Nonbuilding structures shall be designed to provide the strength required to resist the displacements induced by the minimum lateral forces specified in this section. Design shall conform to the applicable provisions of other sections as modified by the provisions contained in Section 1634A.

1634A.1.2 Criteria. The minimum design seismic forces prescribed in this section are at a level that produce displacements in a fixed base, elastic model of the structure, comparable to those expected of the real structure when responding to the Design Basis Ground Motion. Reductions in these forces using the coefficient R is permitted where the design of nonbuilding structures provides sufficient strength and ductility, consistent with the provisions specified herein for buildings, to resist the effects of seismic ground motions as represented by these design forces.

When applicable, design strengths and other detailed design criteria shall be obtained from other sections or their referenced standards. The design of nonbuilding structures shall use the load combinations or factors specified in Section 1612A.2 or 1612A.3. For nonbuilding structures designed using Section 1634A.3, 1634A.4 or 1634A.5, the Reliability/Redundancy Factor, ρ, may be taken as 1.0.

When applicable design strengths and other design criteria are not contained in or referenced by this code, such criteria shall be obtained from approved national standards.

1634A.1.3 Weight W. The weight, W, for nonbuilding structures shall include all dead loads as defined for buildings in Section 1630A.1.1. For purposes of calculating design seismic forces in nonbuilding structures, W shall also include all normal operating contents for items such as tanks, vessels, bins and piping.

1634A.1.4 Period. The fundamental period of the structure shall be determined by rational methods such as by using Method B in Section 1630A.2.2.

1634A.1.5 Drift. The drift limitations of Section 1630A.10 need not apply to nonbuilding structures. Drift limitations shall be established for structural or nonstructural elements whose failure would cause life hazards. $P\Delta$ effects shall be considered for structures whose calculated drifts exceed the values in Section 1630A.1.3.

1634A.1.6 Interaction effects. In Seismic Zones 3 and 4, structures that support flexible nonstructural elements whose combined weight exceeds 25 percent of the weight of the structure shall be designed considering interaction effects between the structure and the supported elements.

1634A.2 Lateral Force. Lateral-force procedures for nonbuilding structures with structural systems similar to buildings (those with structural systems which are listed in Table 16A-N) shall be selected in accordance with the provisions of Section 1629A. *

1634A.3 Rigid Structures. Rigid structures (those with period T less than 0.06 second) and their anchorages shall be designed for the lateral force obtained from Formula (34A-1).

$$V = 0.7C_a IW \qquad (34A\text{-}1)$$

The force V shall be distributed according to the distribution of mass and shall be assumed to act in any horizontal direction.

1634A.4 Tanks with Supported Bottoms. Flat bottom tanks or other tanks with supported bottoms, founded at or below grade, shall be designed to resist the seismic forces calculated using the procedures in Section 1634A for rigid structures considering the entire weight of the tank and its contents. Alternatively, such tanks may be designed using one of the two procedures described below:

1. A response spectrum analysis that includes consideration of the actual ground motion anticipated at the site and the inertial effects of the contained fluid.

2. A design basis prescribed for the particular type of tank by an approved national standard, provided that the seismic zones and occupancy categories shall be in conformance with the provisions of Sections 1629A.4 and 1629A.2, respectively.

1634A.5 Other Nonbuilding Structures. Nonbuilding structures that are not covered by Sections 1634A.3 and 1634A.4 shall be designed to resist design seismic forces not less than those determined in accordance with the provisions in Section 1630A with the following additions and exceptions:

1. The factors R and Ω_o shall be as set forth in Table 16A-P. The total design base shear determined in accordance with Section 1630A.2 shall not be less than the following:

$$V = 0.56C_a IW \qquad (34A\text{-}2)$$

Additionally, for Seismic Zone 4, the total base shear shall also not be less than the following:

$$V = \frac{1.6\ ZN_v\ I}{R} W \qquad (34A\text{-}3)$$

2. The vertical distribution of the design seismic forces in structures covered by this section may be determined by using the provisions of Section 1630A.5 or by using the procedures of Section 1631A.

> **EXCEPTION:** For irregular structures assigned to Occupancy Categories 1 and 2 that cannot be modeled as a single mass, the procedures of Section 1631A shall be used.

3. Where an approved national standard provides a basis for the earthquake-resistant design of a particular type of nonbuilding structure covered by this section, such a standard may be used, subject to the limitations in this section:

The seismic zones and occupancy categories shall be in conformance with the provisions of Sections 1629A.4 and 1629A.2, respectively.

The values for total lateral force and total base overturning moment used in design shall not be less than 80 percent of the values that would be obtained using these provisions.

SECTION 1635A — EARTHQUAKE-RECORDING INSTRUMENTATIONS

For earthquake-recording instrumentations, see Appendix Chapter 16, Division II.

Division V—SOIL PROFILE TYPES

SECTION 1636A — SITE CATEGORIZATION PROCEDURE

1636A.1 Scope. This division describes the procedure for determining Soil Profile Types S_A through S_F in accordance with Table 16A-J.

1636A.2 Definitions. Soil profile types are defined as follows:

S_A Hard rock with measured shear wave velocity, $\bar{v}_s > 5{,}000$ ft./sec. (1500 m/s).

S_B Rock with 2,500 ft./sec. $< \bar{v}_s \leq$ 5,000 ft./sec. (760 m/s $< \bar{v}_s \leq$ 1500 m/s).

S_C Very dense soil and soft rock with 1,200 ft./sec. $< \bar{v}_s \leq$ 2,500 ft./sec. (360 m/s $\bar{v}_s \leq$ 760 m/s) or with either $\bar{N} > 50$ or $\bar{s}_u \geq$ 2,000 psf (100 kPa).

S_D Stiff soil with 600 ft./sec. $\leq \bar{v}_s \leq$ 1,200 ft./sec. (180 m/s $\leq \bar{v}_s \leq$ 360 m/s) or with $15 \leq \bar{N} \leq 50$ or 1,000 psf $\leq \bar{s}_u \leq$ 2,000 psf (50 kPa $\leq \bar{s}_u \leq$ 100 kPa).

S_E A soil profile with $\bar{v}_s <$ 600 ft./sec. (180 m/s) or any profile with more than 10 ft. (3048 mm) of soft clay defined as soil with $PI > 20$, $w_{mc} \geq 40$ percent and $s_u <$ 500 psf (25 kPa).

S_F Soils requiring site-specific evaluation:

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

2. Peats and/or highly organic clays [$H >$ 10 ft. (3048 mm) of peat and/or highly organic clay where H = thickness of soil].

3. Very high plasticity clays [$H >$ 25 ft. (7620 mm) with $PI >$ 75].

4. Very thick soft/medium stiff clays [$H >$ 120 ft. (36 580 mm)].

> **EXCEPTION:** When the soil properties are not known in sufficient detail to determine the soil profile type, Type S_D shall be used. Soil Profile Type S_E need not be assumed unless the building official determines that Soil Profile Type S_E may be present at the site or in the event that Type S_E is established by geotechnical data.

The criteria set forth in the definition for Soil Profile Type S_F requiring site-specific evaluation shall be considered. If the site corresponds to this criteria, the site shall be classified as Soil Profile Type S_F and a site-specific evaluation shall be conducted.

1636A.2.1 $\bar{v}_s$, Average shear wave velocity. $\bar{v}_s$ shall be determined in accordance with the following formula:

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \quad \text{(36A-1)}$$

WHERE:

d_i = thickness of Layer i in feet (m).

v_{si} = shear wave velocity in Layer i in ft./sec. (m/sec).

1636A.2.2 $\bar{N}$, average field standard penetration resistance and $\bar{N}_{CH}$, average standard penetration resistance for cohesionless soil layers. $\bar{N}$ and $\bar{N}_{CH}$ shall be determined in accordance with the following formula:

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \quad \text{(36A-2)}$$

and

$$\bar{N}_{CH} = \frac{d_s}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \quad \text{(36A-3)}$$

WHERE:

d_i = thickness of Layer i in feet (mm).

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

N_i = the standard penetration resistance of soil layer in accordance with approved nationally recognized standards.

1636A.2.3 $\bar{s}_u$, Average undrained shear strength. $\bar{s}_u$ shall be determined in accordance with the following formula:

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{n} \frac{d_i}{S_{ui}}} \quad \text{(36A-4)}$$

WHERE:

d_c = the total thickness $(100 - d_s)$ of cohesive soil layers in the top 100 feet (30 480 mm).

S_{ui} = the undrained shear strength in accordance with approved nationally recognized standards, not to exceed 5,000 psf (250 kPa).

1636A.2.4 Soft clay profile, S_E. The existence of a total thickness of soft clay greater than 10 feet (3048 mm) shall be investigated where a soft clay layer is defined by $s_u <$ 500 psf (24 kPa), $w_{mc} \geq 40$ percent and $PI > 20$. If these criteria are met, the site shall be classified as Soil Profile Type S_E.

1636A.2.5 Soil profiles S_C, S_D and S_E. Sites with Soil Profile Types S_C, S_D and S_E shall be classified by using one of the following three methods with $\bar{v}_s$, $\bar{N}$ and $\bar{s}_u$ computed in all cases as specified in Section 1636A.2.

1. $\bar{v}_s$ for the top 100 feet (30 480 mm) ($\bar{v}_s$ method).
2. $\bar{N}$ for the top 100 feet (30 480 mm) ($\bar{N}$ method).
3. $\bar{N}_{CH}$ for cohesionless soil layers ($PI < 20$) in the top 100 feet (30 480 mm) and average $\bar{s}_u$ for cohesive soil layers ($PI > 20$) in the top 100 feet (30 480 mm) ($\bar{s}_u$ method).

1636A.2.6 Rock profiles, S_A and S_B. The shear wave velocity for rock, Soil Profile Type S_B, shall be either measured on site or estimated by a geotechnical engineer, engineering geologist or seismologist for competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Soil Profile Type S_C.

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

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

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

SECTION 1637A — SITE DATA FOR HOSPITALS, PUBLIC ELEMENTARY AND SECONDARY SCHOOLS AND STATE-OWNED OR STATE-LEASED ESSENTIAL SERVICES BUILDINGS

1637A.1 Engineering geologic reports.

1637A.1.1 *Geologic and earthquake engineering reports shall be required for all proposed construction.*

> ***EXCEPTIONS:*** *1. Reports are not required for one-story, wood-frame and light-steel-frame buildings of Type II or Type V construction and 4,000 square feet ($371 m^2$) or less in floor area; nonstructural, associated structural or nonrequired structural alterations and incidental structural additions or alterations, and structural repairs for other than earthquake damage.*
>
> *2. A previous report for a specific site may be resubmitted, provided that a reevaluation is made and the report is found to be currently appropriate.*

1637A.1.2 *The purpose of the engineering geologic report shall be to identify geologic and seismic conditions that may require project mitigations. The reports shall contain data which provide an assessment of the nature of the site and potential for earthquake damage based on appropriate investigations of the regional and site geology, project foundation conditions and the potential seismic shaking at the site. The report shall be prepared by a California-certified engineering geologist in consultation with a California-registered geotechnical engineer. The engineering geologic report shall not contain design criteria, but shall contain basic data to be used for a preliminary earthquake engineering evaluation of the project.*

The preparation of the engineering geologic report shall be done in conformance with the most recent Division of Mines and Geology (DMG) Notes 44 and 42; Guidelines for Preparing Engineering Geologic Reports, and Guidelines to Geologic/Seismic Reports, respectively. Upper-bound earthquakes, proposed in the Engineering Geologic Report, must be fully supported by satisfactory data and analysis. In addition, the most recent version of DMG Special Publication 42, Fault Rupture Hazard Zones in California, shall be considered for project sites proposed within an Alquist-Priolo special studies zone.

The report shall include, but shall not be limited to, the following:

1. Geologic investigation.

2. Evaluation of the known active and potentially active faults, both regional and local, including estimates of their upper-bound earthquakes and estimates of the peak ground accelerations at the site resulting from these earthquakes.

3. Evaluation of slope stability at or near the site, and the liquefaction and settlement potential of the earth materials in the foundation.

1637A.1.3 *The engineering geologic report shall be submitted to the enforcement agency for review and approval. The review shall determine whether potential geologic problems and hazards are adequately identified and described in order to provide a timely completion of the subsequent geotechnical report, described in Section 1637A.2.1. The enforcement agency, with consultation of its advisors, may require additional information, analysis and/or clarification of potential geologic problems affecting the proposed building site before approval is given. The results of the approved engineering geologic report shall be used as a basis for further investigations for the geotechnical report. Approval of the engineering geologic report by the enforcement agency shall be required prior to the submission of the geotechnical report.*

1637A.2 Geotechnical and Supplemental Ground-response Reports.

1637A.2.1 Geotechnical report.

1637A.2.1.1 *The geotechnical report shall provide completed evaluations of the foundation conditions of the site and the potential geologic/seismic hazards affecting the site. The geotechnical report shall include, but shall not be limited to, site-specific evaluations of design criteria related to the nature and extent of foundation materials, groundwater conditions, liquefaction potential, settlement potential and slope stability. The report shall contain the results of the analyses of problem areas identified in the engineering geologic report. The geotechnical report shall incorporate estimates of the characteristics of site ground motion provided in the engineering geologic report. The estimates of ground motion shall not be structural design criteria, but shall be provided to characterize the seismic environment of the site, with consideration of the upper-bound earthquakes reported in the engineering geologic report. The ground-motion estimates shall include, but shall not be limited to, peak ground motions and predominant period. The estimates should be derived by accepted methods of current seismological practice and fully documented in the geologic report.*

The geotechnical report shall be prepared by a geotechnical engineer registered in the state of California with the advice of the certified engineering geologist and other technical experts, as necessary. The approved engineering geologic report shall be submitted with or as part of the geotechnical report.

1637A.2.1.2 *The geotechnical report shall be submitted to the enforcement agency for review and approval. The review shall determine whether potential geologic hazards and foundation problems have been adequately evaluated. The enforcement agency, with the consultation of its advisors, may require additional information, analysis or clarification of potential geotechnical issues affecting the proposed building site before approving the geotechnical report.*

Approval of the geotechnical report by the enforcement agency shall be required prior to the approval of the supplemental ground-response report, if required, as described in Section 1637A.2.2. The results of the geotechnical report shall be used as a guide for further investigations for the supplemental ground-response report.

1637A.2.2 Supplemental ground-response report. *A supplemental ground-response report may be required, containing a ground-motion element and an advanced geotechnical element.*

1637A.2.2.1 *The ground-motion element shall be prepared when required by the approved geotechnical report, or when required for dynamic analysis procedures described under Section 1631A.2. The ground-motion element shall be prepared by a registered civil engineer or geophysicist (depending on the scope of the element), or engineering geologist licensed in the state of California, and having professional specialization in earthquake analyses. The ground-motion element shall present a detailed characterization of earthquake ground motions for the site, which* *incorporates data given in the geotechnical report. The level of ground motion considered by the ground-motion element shall be as described in Section 1631A.2. The characterization of ground motion in the ground-motion element shall be given, according to the requirements of the analysis, in terms of:*

1. Peak acceleration, bracketed duration and predominant period.

2. Elastic structural response spectra.

3. Time-history plot of predicted ground motion at the site.

4. Other analyses in conformance with accepted engineering and seismological practice.

1637A.2.2.2 The advanced geotechnical element shall contain the results of dynamic geotechnical analyses specified by the approved geotechnical report.

The supplemental ground-response report shall be submitted to the enforcement agency for review and approval. The review shall determine whether the ground-motion response evaluations of the site are adequately represented. The enforcement agency, under consultation with its advisors, may require additional information, analysis or clarification of potential ground-response issues reported in the supplemental ground-response report for the proposed building site.

SECTION 1638A [FOR OSHPD 1 & 4] — ADDITIONS, ALTERATIONS, REPAIRS AND SEISMIC RETROFIT TO EXISTING BUILDINGS OR STRUCTURES

Existing hospital buildings (as defined in Section 7-111, Part 1, Title 24, Building Standards Administrative Code).

***NOTE:** Alterations to lateral shear force-resisting capacity and story lateral shear forces shall be considered to be cumulative for purposes of defining incidental or minor alterations or additions. The percentage of cumulative changes shall be based on as-built conditions existing on March 7, 1973.*

1638A.1 ***Alterations**. For this section, alterations include any additions, alterations, repairs, and/or seismic retrofits to an existing hospital building or portions thereof. The provisions of Section 3403, "Additions, Alterations or Repairs" of Chapter 34 of the California Building Code shall apply for hospital buildings.*

1638A.2 ***Seismic Retrofit**. Any seismic retrofits of hospital buildings required by Article 2 and Article 11, Chapter 6, Part 1, Title 24, shall meet the requirements of Sections 1640A through 1649A.*

> ***EXCEPTION:** Hospital buildings evaluated to SPC 1 due to deficiencies identified by Article 10, Chapter 6, Part 1, Title 24, may be upgraded to SPC 2 by altering, repairing or seismically retrofitting these conditions in accordance with the requirements of Sections 1640A through 1649A.*

1638A.3 Alterations, additions and repairs to existing buildings or structures not required by Chapter 6, Part 1, Title 24.

***1638A.3.1 Approved existing buildings.** Structural alterations or repairs may be made to approved buildings provided the entire building, as modified, including the structural alterations or repairs, conforms to Sections 1640A through 1649A requirements for the seismic structural performance category (SPC) of the building as determined in Chapter 6, Part 1, Title 24. Additions shall conform to the requirements of these regulations for new construction.*

1638A.3.2 Pre-1973 buildings.

***1638A.3.2.1 Incidental structural alteration, additions or repairs.** The existing structural elements affected by the alteration, addition or repair shall conform or shall be made to conform to the vertical load requirements of these regulations. Incidental structural additions will be permitted provided the additions meet these regulations for new construction using the importance factor, I, equal to or greater than 1.0. Alterations or repairs to the existing lateral load-resisting system must meet the requirements of Sections 1640A through 1649A.*

***1638A.3.2.2 Minor structural alterations, additions or repairs.** Minor structural alterations, additions or repairs will be permitted provided they meet the following: Alterations to existing gravity and/or lateral load-resisting system shall be made to conform to the requirements of Section 1640A through 1649A; or additions shall meet all of the requirements of these regulations for new construction using an importance factor, I, equal to or greater than 1.0.*

***1638A.3.2.3 Major structural alterations, additions or repairs.** Major structural alterations will be permitted provided the entire building, as modified, including the structural alterations or repairs, conforms to the requirements of Sections 1640A through 1649A for no less than SPC 2. Additions shall meet the requirements of these regulations for new construction.*

It shall also be demonstrated by a written report submitted by the structural engineer, acceptable to the enforcement agency, that an investigation of the existing building structure shows it to be constructed in reasonable conformance with the submitted drawings and specifications.

1638A.3.3 An alteration which involves the removal of one or more entire stories will be permitted if the lateral-load-resisting capacity of the remaining structure is not reduced.

An alteration which involves the removal of other than one or more entire stories will be permitted in accordance with Sections 1640A through 1649A.

SECTION 1639A — RESERVED

TABLE 16A-K—OCCUPANCY CATEGORY

OCCUPANCY CATEGORY	OCCUPANCY OR FUNCTIONS OF STRUCTURE	SEISMIC IMPORTANCE FACTOR, I	SEISMIC IMPORTANCE[1] FACTOR, I_p	WIND IMPORTANCE FACTOR, I_w
1. Essential facilities[2]	*Hospitals and other medical facilities as defined in Section 1250, Health and Safety Code* Group I, Division 1 Occupancies having surgery and emergency treatment areas Fire and police stations, *sheriffs offices, California Highway Patrol offices and California State Police Offices* *Municipal, county and state government disaster operation and communication centers deemed vital in emergencies* *For wind only, building areas where the primary occupancy is for assembly use for more than 300 people and portions of connecting or adjacent structures, the collapse of which would endanger the assembly area or restrict egress from it. (For earthquake, see Category 3.)* Garages and shelters for emergency vehicles and emergency aircraft Structures and shelters in emergency-preparedness centers Aviation control towers Structures and equipment in government communication centers and other facilities required for emergency response Standby power-generating equipment for Category 1 facilities Tanks or other structures containing housing or supporting water or other fire-suppression material or equipment required for the protection of Category 1, 2 or 3 structures	*1.50*	1.50	1.15
2. Hazardous facilities	Group H, Divisions 1, 2, 6 and 7 Occupancies and structures therein housing or supporting toxic or explosive chemicals or substances Nonbuilding structures housing, supporting or containing quantities of toxic or explosive substances that, if contained within a building, would cause that building to be classified as a Group H, Division 1, 2 or 7 Occupancy	1.25	1.50	1.15
3. Special occupancy structures[3]	Group A, Divisions 1, 2 and 2.1 Occupancies *For earthquake only, covered structures whose primary occupancy is public assembly-capacity greater than 300 persons. (For wind, see Category 1.)* Buildings housing Group E, Divisions 1 and 3 Occupancies with a capacity greater than 300 students Buildings housing Group B Occupancies used for college or adult education with a capacity greater than 500 students Group I, Divisions 1 and 2 Occupancies with 50 or more resident incapacitated patients, but not included in Category 1 Group I, Division 3 Occupancies All structures with an occupancy greater than 5,000 persons Structures and equipment in power-generating stations, and other public utility facilities not included in Category 1 or Category 2 above, and required for continued operation	*1.15*	*1.15*	1.00
4. Standard occupancy structures[3]	All structures housing occupancies or having functions not listed in Category 1, 2 or 3 and Group U Occupancy towers	1.00	1.00	1.00
5. Miscellaneous structures	Group U Occupancies except for towers	1.00	1.00	1.00

[1]The limitation of I_p for panel connections in Section 1633A.2.4 shall be 1.0 for the entire connector.
[2]Structural observation requirements are given in Section 1702A.
[3]For anchorage of machinery and equipment required for life-safety systems, the value of I_p shall be taken as 1.5.

TABLE 16A-L—VERTICAL STRUCTURAL IRREGULARITIES

IRREGULARITY TYPE AND DEFINITION	REFERENCE SECTION
1*a*. **Stiffness irregularity—soft story** A soft story is one in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above.	1629A.8.4, Item 2
*1b. **Severe soft story*** *A severe soft story is one in which the lateral stiffness is less than 60 percent of that in the story above or less than 70 percent of the average stiffness of three stories above.*	*1629A.9.4*
2. **Weight (mass) irregularity** Mass irregularity shall be considered to exist where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	1629A.8.4, Item 2
3. **Vertical geometric irregularity** Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral-force-resisting system in any story is more than 130 percent of that in an adjacent story. One-story penthouses need not be considered.	1629A.8.4, Item 2
4. **In-plane discontinuity in vertical lateral-force-resisting element** An in-plane offset of the lateral-load-resisting elements greater than the length of *the* elements *below.*	1630A.8.2
5. **Discontinuity in capacity—weak story** A weak story is one in which the *ratio of the story strength to the story shear is less than 80 percent of that in the story above. The story strength is the strength of all seismic-resisting elements sharing the story shear for the direction under consideration. The load deformation characteristics of the elements shall be considered so that the strength is determined for compatible deformations.*	1629A.9.1

TABLE 16A-M—PLAN STRUCTURAL IRREGULARITIES

IRREGULARITY TYPE AND DEFINITION	REFERENCE SECTION
1*a*. **Torsional irregularity—to be considered when diaphragms are not flexible** Torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of the structure.	1633A.1, 1633A.2.9, Item 6
*1b. **Severe torsional irregularity—to be considered when diaphragms are not flexible*** *Severe torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts of the two ends of the structure.*	*1629A.9.5* *1633A.2.9, Item 6*
2. **Re-entrant corners** Plan configurations of a structure and its lateral-force-resisting system contain re-entrant corners, where both projections of the structure beyond a re-entrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.	1633A.2.9, Items 6 and 7
3. **Diaphragm discontinuity** Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50 percent from one story to the next.	1633A.2.9, Item 6
4. **Out-of-plane offsets** Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements.	1630A.8.2; 1633A.2.9, Item 6; 2213A.9.1
5. **Nonparallel systems** The vertical lateral-load-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral-force-resisting system.	1633A.1

TABLE 16A-N—STRUCTURAL SYSTEMS[1]

BASIC STRUCTURAL SYSTEM[2]	LATERAL-FORCE-RESISTING SYSTEM DESCRIPTION	R	Ω_o	HEIGHT LIMIT FOR SEISMIC ZONES 3 AND 4 (feet) × 304.8 for mm
1. Bearing wall system	1. Light-framed walls with shear panels			
	a. Wood structural panel walls for structures three stories or less	5.5	2.8	65
	b. All other light-framed walls	4.5	2.8	65
	2. Shear walls			
	a. Concrete	4.5	2.8	160
	b. Masonry	4.5	2.8	160
	3. Light steel-framed bearing walls with tension-only bracing	2.8	2.2	65
	4. Braced frames where bracing carries gravity load			
	a. Steel[10]	4.4	2.2	160
	b. *Not adopted by the State of California.*	—	—	—
	c. Heavy timber[10]	2.8	2.2	65
2. Building frame system	1. Steel eccentrically braced frame (EBF)	7.0	2.8	240
	2. Light-framed walls with shear panels			
	a. Wood structural panel walls for structures three stories or less	6.5	2.8	65
	b. All other light-framed walls	5.0	2.8	65
	3. Shear walls			
	a. Concrete	5.5	2.8	240
	b. Masonry	5.5	2.8	160
	4. Ordinary braced frames			
	a. Steel[10]	5.6	2.2	160
	b. *Not adopted by the State of California.*	—	—	—
	c. Heavy timber[10]	5.6	2.2	65
	5. Special concentrically braced frames			
	a. Steel	6.4	2.2	240
3. Moment-resisting frame system	1. Special moment-resisting frame (SMRF)			
	a. Steel	8.5	2.8	N.L.
	b. Concrete[4]	8.5	2.8	N.L.
	2. Masonry moment-resisting wall frame (MMRWF)	6.5	2.8	160
	3. *Not adopted by the State of California.*	—	—	—
	4. Ordinary moment-resisting frame (OMRF)			
	a. Steel[9]	4.5	2.8	160
	b. *Not adopted by the State of California.*	—	—	—
	5. Special truss moment frames of steel (STMF)[10]	6.5	2.8	240
4. Dual systems	1. Shear walls			
	a. Concrete with SMRF	8.5	2.8	N.L.
	b. Concrete with steel OMRF[10]	4.2	2.8	160
	c. *Not adopted by the State of California.*	—	—	—
	d. Masonry with SMRF	5.5	2.8	160
	e. Masonry with steel OMRF[10]	4.2	2.8	160
	f. *Not adopted by the State of California.*	—	—	—
	g. Masonry with masonry MMRWF	6.0	2.8	160
	2. Steel EBF			
	a. With steel SMRF	8.5	2.8	N.L.
	b. With steel OMRF	4.2	2.8	160
	3. Ordinary braced frames			
	a. Steel with steel SMRF[10]	6.5	2.8	N.L.
	b. Steel with steel OMRF[10]	4.2	2.8	160
	c. *Not adopted by the State of California.*	—	—	—
	d. *Not adopted by the State of California.*	—	—	—
	4. Special concentrically braced frames			
	a. Steel with steel SMRF	7.5	2.8	N.L.
	b. Steel with steel OMRF[10]	4.2	2.8	160
5. Cantilevered column building systems	1. Cantilevered column elements	2.2	2.0	35[7]
6. Shear wall-frame interaction systems	1. *Not adopted by the State of California.*	—	—	—
7. Undefined systems	See Sections 1629A.6.7 and 1629A.9.2	—	—	—

N.L.—no limit

[1]See Section 1630A.4 for combination of structural systems.

[2]Basic structural systems are defined in Section 1629A.6.

[3]*Not adopted by the State of California.*

[4]Includes precast concrete conforming to Section 1921A.2.7.

[5]*Not adopted by the State of California.*

[6]*Not adopted by the State of California.*

[7] Total height of the building including cantilevered columns.

[8]*Not adopted by the State of California.*

[9]*[Not adopted by OSHPD] Allowed only for certain school occupancies. See Section 2213A.6.*

[10]*Not adopted by OSHPD*

TABLE 16A-O—HORIZONTAL FORCE FACTORS, a_p AND R_p

ELEMENTS OF STRUCTURES AND NONSTRUCTURAL COMPONENTS AND EQUIPMENT[1]	a_p	R_p	FOOTNOTE
1. Elements of Structures			
A. Walls including the following:			
(1) Unbraced (cantilevered) parapets.	2.5	3.0	
(2) Exterior walls at or above the ground floor and parapets braced above their centers of gravity.	1.0	3.0	2
(3) All interior-bearing and nonbearing walls.	1.0	3.0	2, *11*
B. Penthouse (except when framed by an extension of the structural frame).	2.5	4.0	
C. Connections for prefabricated structural elements other than walls. See also Section 1632A.2.	1.0	3.0	3
2. Nonstructural Components			
A. Exterior and interior ornamentations and appendages.	2.5	3.0	
B. Chimneys, stacks and trussed towers supported on or projecting above the roof:			
(1) Laterally braced or anchored to the structural frame at a point below their centers of mass.	2.5	3.0	
(2) Laterally braced or anchored to the structural frame at or above their centers of mass.	1.0	3.0	
C. Signs and billboards.	2.5	3.0	
D. Storage racks (include contents) *with upper storage level more than 5 feet (1524 mm) in height*	2.5	4.0	4, *23*
E. Permanent floor-supported cabinets and book stacks more than 6 feet (1829 mm) in height (include contents).	1.0	3.0	5, *23, 24*
F. Anchorage and lateral bracing for suspended ceilings and light fixtures.	1.0	3.0	3, 6, 7, 8, *25, 26*
G. Access floor systems.	1.0	3.0	9, *14*
H. Masonry or concrete fences over 6 feet (1829 mm) high.	1.0	3.0	
I. Partitions.	1.0	3.0	*11*
J. Wall hung cabinets and storage shelving (plus contents)	*1.0*	*3.0*	
3. Equipment			
A. Tanks and vessels (include contents), including support systems.	1.0	3.0	
B. Electrical, mechanical and plumbing equipment and associated conduit and ductwork and piping, *and machinery. In essential services buildings, this includes all piping, electrical conduits, cable trays and air-handling ducting necessary to the continuing operation of the facility.*	1.0	3.0	5, 10, 12, 13, 14, 15, 16
C. Any flexible equipment laterally braced or anchored to the structural frame at a point below their center of mass.	2.5	3.0	5, 10, 14, 15, 16
D. Anchorage of emergency power supply systems and essential communications equipment. Anchorage and support systems for battery racks and fuel tanks necessary for operation of emergency equipment. See also Section 1632A.2.	*1.0*	*2.5*	*14,* 17, 18, *21*
E. Temporary containers with flammable or hazardous materials.	1.0	3.0	19
F. Power cable-driven elevators or hydraulic elevators with lifts over 5 feet (1524 mm): *(1) Hoistway structural framing providing the support for guide rail brackets*			*27*
(2) Guide rails and guide rail brackets			
(3) Car and counterweight auxiliary guiding members or retainer plates			
(4) Driving machinery, pump unit tanks operating devices and control equipment cabinets			
4. Other Components			
A. Rigid components with ductile material and attachments.	1.0	3.0	1, *14*
B. Rigid components with nonductile material or attachments.	1.0	1.5	1, *14*
C. Flexible components with ductile material and attachments.	2.5	3.0	1, *14*
D. Flexible components with nonductile material or attachments.	2.5	1.5	1, *14*

[1] See Section 1627A for definitions of flexible components and rigid components. *See Section 1632A for formula using a_p. Horizontal forces are to be applied in any horizontal direction. The value of a_p shall not be reduced for all walls. Welded, bolted or other intermittent connections such as inserts for anchorage of nonstructural components shall not be allowed the one-third increase in allowable stress permitted in Section 1612A.3.2.*

[2] See Sections 1633A.2.4 and 1633A.2.8 for concrete and masonry walls and Section 1632A.2 for connections for *wall* panels.

[3] Applies to Seismic Zones 2, 3 and 4 only.

[4] Ground supported steel storage racks may be designed using the provisions of Section 1634A. Chapter 22A, Division *X*, may be used for design, provided seismic design forces are equal to or greater than those specified in Section 1632A.5 or 1634A.5, as appropriate.

[5] Only attachments, anchorage or restraints need be designed. *See Section 1632A.1.*

[6] Ceiling weight shall include all light fixtures and other equipment or partitions that are laterally supported by the ceiling. For purposes of determining the seismic force, a ceiling weight of not less than 4 psf (0.19 kN/m^2) shall be used.

[7] Ceilings constructed of lath and plaster or gypsum board screw or nail attached to suspended members that support a ceiling at one level extending from wall to wall need not be analyzed, provided the walls are not over 50 feet (15 240 mm) apart.

[8] Light fixtures and mechanical services installed in metal suspension systems for acoustical tile and lay-in panel ceilings shall be independently supported from the structure above as specified in UBC Standard 25-2, Part III.

(Continued)

FOOTNOTES TO TABLE 16A-O—(Continued)

[9]W_p for access floor systems shall be the dead load of the access floor system plus 25 percent of the floor live load plus a 10-psf (0.48 kN/m^2) partition load allowance.

[10]Equipment includes, but is not limited to, boilers, chillers, heat exchangers, pumps, air-handling units, cooling towers, control panels, motors, switchgear, transformers and life-safety equipment. It shall include major conduit, ducting and piping, which services such machinery and equipment and fire sprinkler systems. See Section 1632A.2 for additional requirements for determining a_p for nonrigid or flexibly mounted equipment.

[11]*See also Section 1611A.5 for minimum load and deflection criteria for interior partitions.*

[12]Seismic restraints may be omitted from electrical raceways, such as cable trays, conduit and bus ducts, if all the following conditions are satisfied:

[12.1] Lateral motion of the raceway will not cause damaging impact with other systems.

[12.2] Lateral motion of the raceway does not cause loss of system vertical support.

[12.3] Rod-hung supports of less than 12 inches (305 mm) in length *where the hangers are detailed to avoid bending of the hangers and their connections.*

[12.4] Support members cantilevered up from the floor are checked for stability.

[13]Piping, ducts and electrical raceways, which must be functional following an earthquake, spanning between different buildings or structural systems shall be sufficiently flexible to withstand relative motion of support points assuming out-of-phase motions.

[14]Vibration isolators supporting *nonstructural components and* equipment shall be designed for lateral loads or restrained from displacing laterally by other means. Restraint shall also be provided, which limits vertical displacement, such that lateral restraints do not become disengaged. a_p and R_p for equipment supported on vibration isolators shall be taken as 2.5 and 1.5, respectively. *If* the isolation mounting frame is supported by shallow or expansion anchors, the design forces for the anchors calculated by Formula (32A-1), (32A-2) or (32A-3) shall be ✳ ✳ ✳ multiplied by a factor of *1.3*.

[15]Equipment anchorage shall not be designed such that lateral loads are resisted by friction *induced solely by the effects of gravity* (e.g., friction clips).

[16]Expansion anchors, which are required to resist seismic loads in tension, shall not be used where operational vibrating loads are present.

[17]Movement of components within electrical cabinets, rack- and skid-mounted equipment and portions of skid-mounted electromechanical equipment that may cause damage to other components by displacing, shall be restricted by attachment to anchored equipment or support frames.

[18]Batteries on racks shall be restrained against movement in all directions due to earthquake forces.

[19]Seismic restraints may include straps, chains, bolts, barriers or other mechanisms that prevent sliding, falling and breach of containment of flammable and toxic materials. Friction forces may not be used to resist lateral loads in these restraints unless positive uplift restraint is provided which ensures that the friction forces act continuously.

[20]*Not used.*

[21]*Emergency equipment should be located where there is the least likelihood of damage due to earthquake. Such equipment should be located at ground level, and where it can be easily maintained to assure its operation during an emergency.*

[22]*Not used.*

[23]*Floor-supported storage racks, cabinets or book stacks not more than 5 feet (1524 mm) in height need not be anchored if the width of the supporting base or width between the exterior legs is equal to or greater than two thirds the height. In addition to gravity loads, storage racks or cabinets shall be designed and constructed to resist the horizontal force, F_p, with the base assumed to be anchored.*

[24]*Mobile storage racks or cabinets mounted on wheels and not restrained by fixed tracks are not subject to approval by the enforcement agency when the rack or cabinet is not more than 5 feet (1524 mm) in height and the width of the supporting base or width between the exterior legs/wheels is equal to or greater than two thirds the height. All such racks or cabinets shall be restrained to prevent movement when not in use. Movable storage racks or cabinets mounted on wheels or glides restrained by fixed tracks shall be designed and constructed to resist the horizontal force, F_p, with the base of the rack or cabinet assumed to be anchored. Provisions shall be made to resist translation perpendicular to the track and overturning both perpendicular and parallel to the track.*

[25]*Suspension systems for light fixtures which have passed shaking table tests approved by the enforcement agency, or which, as installed, are free to swing a minimum of 45 degrees from the vertical in all directions without contacting obstructions, shall be assumed to comply with the lateral-force requirements of Section 1632A.2. Unless the cable-type, free-swinging suspension systems shall have a safety wire or cable attached to the fixture and structure at each support capable of supporting four times the supported load.*

[26]*For suspended and surface-mounted light fixtures, the product of I a_p need not exceed 1.5 for any value of I.*

[27]*See Section 1633A.2.13.*

TABLE 16A-P—R AND Ω_o FACTORS FOR NONBUILDING STRUCTURES

STRUCTURE TYPE	R	Ω_o
1. Vessels, including tanks and pressurized spheres, on braced or unbraced legs.	2.2	2.0
2. Cast-in-place concrete silos and chimneys having walls continuous to the foundations.	3.6	2.0
3. Distributed mass cantilever structures such as stacks, chimneys, silos and skirt-supported vertical vessels.	2.9	2.0
4. Trussed towers (freestanding or guyed), guyed stacks and chimneys.	2.9	2.0
5. Cantilevered column-type structures.	—	—
5.1 Single-column structures	*2.2*	*2.0*
5.2 Multicolumn structures with strut ties capable of developing the capacity of the column	*2.9*	*2.0*
6. Cooling towers.	3.6	2.0
7. Bins and hoppers on braced or unbraced legs.	2.9	2.0
8. Storage racks.	3.6	2.0
9. Signs and billboards.	3.6	2.0
10. Amusement structures and monuments.	2.2	2.0
11. All other self-supporting structures not otherwise covered.	2.9	2.0

TABLE 16A-Q—SEISMIC COEFFICIENT C_a

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
S_F	See Footnote 1				

[1]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

TABLE 16A-R—SEISMIC COEFFICIENT C_v

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$
S_F	See Footnote 1				

[1]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

TABLE 16A-S—NEAR-SOURCE FACTOR N_a [1]

SEISMIC SOURCE TYPE	CLOSEST DISTANCE TO KNOWN SEISMIC SOURCE[2,3]		
	≤ 2 km	5 km	≥ 10 km
A	1.5	1.2	1.0
B	1.3	1.0	1.0
C	1.0	1.0	1.0

[1]The Near-Source Factor may be based on the linear interpolation of values for distances other than those shown in the table.

[2]The location and type of seismic sources to be used for design shall be established based on approved geotechnical data (e.g., most recent mapping of active faults by the United States Geological Survey or the California Division of Mines and Geology).

[3]The closest distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of the source on the surface (i.e., surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km or greater. The largest value of the Near-Source Factor considering all sources shall be used for design.

TABLE 16A-T—NEAR-SOURCE FACTOR N_v [1]

SEISMIC SOURCE TYPE	CLOSEST DISTANCE TO KNOWN SEISMIC SOURCE[2,3]			
	≤ 2 km	5 km	10 km	≥ 15 km
A	2.0	1.6	1.2	1.0
B	1.6	1.2	1.0	1.0
C	1.0	1.0	1.0	1.0

[1]The Near-Source Factor may be based on the linear interpolation of values for distances other than those shown in the table.

[2]The location and type of seismic sources to be used for design shall be established based on approved geotechnical data (e.g., most recent mapping of active faults by the United States Geological Survey or the California Division of Mines and Geology).

[3]The closest distance to seismic source shall be taken as the minimum distance between the site and the area described by the vertical projection of the source on the surface (i.e., surface projection of fault plane). The surface projection need not include portions of the source at depths of 10 km or greater. The largest value of the Near-Source Factor considering all sources shall be used for design.

9. Propose the program for additional data collection, condition assessment and testing requirements to complete the analysis. Identify locations for the proposed material assessment and tests.

Submit with the Evaluation and Design Criteria Report:

1. Approved or "as-built" building plans, specifications and associated construction documents that accurately depict the existing construction.

2. Available material test reports, geohazard and geotechnical reports from the existing construction.

1640A.8.3 Requirements for Method B.

1. Upon selection of Method B, the design professional(s) in responsible charge of the design and the independent peer reviewer(s) shall meet with the DSA prior to development of the Evaluation and Design Criteria Report to: define the scope of the structural rehabilitation, determine appropriate evaluation and design methodologies, determine initial data collection requirements, and determine the scope of the peer review process for the project.

2. During the schematic phase, upon review of the Evaluation and Design Criteria Report, the peer reviewer shall provide a written report to the owner and DSA in accordance with Section 1649A.

3. During the design development phase of the project, upon completion of the analysis, the peer reviewer shall review the analysis results and provide a written progress report in accordance with Section 1649A. The design professional(s) shall provide responses and corrective actions in accordance with Section 1649A.6.

> **EXCEPTION:** When the DSA determines that the project scope does not require a report during the design development phase, this requirement may be waived by the DSA.

4. Upon completion of the construction documents prior to submittal of the application to the DSA, the peer reviewer shall review the plans, specifications and any final analysis results and provide a written report to the owner and DSA in accordance with Section 1649A. The design professional(s) shall provide responses and corrective actions in accordance with Section 1649A.6.

5. During construction of the rehabilitation and when determined necessary by the design professional or the DSA, the peer reviewer shall review proposed changes to the approved plans and specifications and provide a written report to the owner and DSA in accordance with Section 1649A.

1640A.9 [For DSA/SS] Where only a portion(s) of a structure is to be rehabilitated, the school portion of the structure shall:

1. Be seismically separated from the unrehabilitated portion in accordance with Section 1646A.2.11.1, or the entire structure shall be rehabilitated in accordance with this division. For structures in which the unrehabilitated portion is above or below the school portion, the entire structure shall be rehabilitated in accordance with this division.

2. Be retrofitted as necessary to protect the occupants from falling hazards of the unrehabilitated portion of the building, and;

3. Be retrofitted as necessary to protect required exitways being blocked by collapse or falling hazards of the unrehabilitated portion.

SECTION 1641A — DEFINITIONS

1641A.1 For the purposes of this division, certain terms are defined in addition to those in Section 1627A and Chapter 6, Part 1, Title 24, Building Standards Administrative Code, as follows:

ACTIVE EARTHQUAKE FAULT is one that has exhibited surface displacement within Holocene time (about 11,000 years) as determined by the California Division of Mines and Geology under the Alquist-Priolo Special Studies Zones Act or other authoritative source, Federal, State or Local Governmental Agency.

CODE-COMPLYING ELEMENT [Not adopted by DSA/SS] is an element that complies with the Seismic Zones 3 and 4 detailing requirements for elements that are part of the selected lateral-force-resisting system as given in the 1976 or later editions of the UBC [For OSHPD 1, 4, Title 17 and Title 24]. Refer to Section 1645A for specific elements and materials.

[For DSA/SS] CODE-COMPLYING ELEMENT is an element that complies with the Seismic Zones 3 and 4 detailing requirements for "ductile" elements that are part of the lateral-force-resisting system for a β equal to 1.0 as defined in Section 1645A for specific elements and materials.

CODE-COMPLYING SYSTEM [Not adopted by DSA/SS] is a system that complies with the Seismic Zones 3 and 4 requirements for lateral-force-resisting systems and materials as given in the 1976 or later editions of Title 17 and Title 24.

[For DSA/SS] CODE-COMPLYING SYSTEM is a system that complies with the Seismic Zones 3 and 4 requirements for lateral-force-resisting systems and materials consisting of code-complying elements.

DESIGN is the procedure that includes both the evaluation and retrofit design of an existing element and the design of a new element.

DESIGN BASIS EARTHQUAKE is the earthquake ground motion having a 5 percent damped acceleration response spectrum as represented by $R/1$ times the Base Shear V given by Formulas (44A-1) and (44A-2).[For OSHPD 1,4 is the earthquake ground motion defined in Section 1648A.2.2.1.]

DISTANCE FROM AN ACTIVE EARTHQUAKE FAULT is measured from the nearest point of the building to the closest edge of an Alquist-Priolo Special Study zone [For DSA/SS: Alquist-Priolo Earthquake Fault Zone] for an active fault, if such a map exists, or to the closest mapped splay of the fault.

DUCTILE ELEMENT is an element capable of sustaining large cyclic deformations beyond the attainment of its nominal strength without any significant loss in capacity. Refer to Section 1645A for specific elements and materials.

ELEMENT is a part of an architectural, electrical, mechanical or structural system.

ENFORCEMENT AGENT is that individual within the agency or organization charged with responsibility for agency or organization compliance with the requirements of Division VI-R.

ESSENTIALLY COMPLYING STRUCTURAL SYSTEM or ELEMENT [Not adopted by DSA/SS] is a lateral-force-resisting system or element that may deviate from but can provide comparable elastic and inelastic cyclic load-deformation behavior as a system or element that complies to the 1976 or later editions of the Uniform Building Code provisions for systems or elements resisting seismic forces. Refer to Section 1645A for specific elements and materials.

[For DSA/SS] ESSENTIALLY COMPLYING STRUCTURAL SYSTEM or ELEMENT is a lateral-force-resisting system or element that may deviate from but can provide comparable elastic and inelastic cyclic load-deformation behavior as a code-complying system or code-complying element.

ESSENTIAL LIFE SAFETY is the retrofit or repair of a structure to a goal of essential life safety as a level of expected structural performance taken to mean that occupants will be able to exit *the structure safely following an earthquake. It does not mean that they will be uninjured or not be in need of medical attention. A structure is presumed to achieve this level of performance where, although significant damage to the structure may have occurred, some margin against either total or partial structural collapse remains, even though damage may not be economical to repair; major structural elements have not become dislodged or fallen so as to pose a life-safety threat; and, nonstructural systems or elements, which are heavy enough to cause severe injuries either within or outside the building, have not become dislodged so as to pose a life-safety threat.*

IMMEDIATE OCCUPANCY. *The retrofit or repair of a structure to a goal of immediate occupancy as a level of expected performance is taken to mean the post-earthquake damage state in which only limited structural and nonstructural damage has occurred. The original strength and stiffness of the structure is substantially retained, with minor cracking and yielding of structural elements. Basic access and life-safety systems, including doors, stairways, elevators, emergency lighting, fire alarms and suppression systems, remain operable, provided that utilities are available. It is expected that occupants could safely remain in the building, although normal use may be impaired and some clean-up, inspection and limited structural and nonstructural repairs may be required.*

INELASTIC DEMAND RATIO (IDR) *is the ratio of the total load demand on an element to the nominal strength capacity of an element, where load demand is the combination of gravity loads and the unreduced (by R) elastic response force due to the specified earthquake ground motion.*

LATERAL LOAD CAPACITY *is the capacity as determined either by Method A or Method B of the subject element. The capacity of a system is the sum of all element capacities acting individually reduced by the β factor for the element and meeting the requirements of Section 1646A.2.4. All forms of loading are to consider both displacements in orthogonal directions and torsion.*

LIMITED-DUCTILE ELEMENT **[Not adopted by DSA/SS]** *is an element that is capable of sustaining moderate cyclic deformations beyond the attainment of nominal strength without significant loss in strength. The deformation capability is less than that of a ductile element, and these elements do not meet the ductile element criteria of the 1976 or later versions of the UBC. Refer to Section 1645A for specific elements and materials.*

[For DSA/SS] LIMITED-DUCTILE ELEMENT *is an element that is capable of sustaining moderate cyclic deformations beyond the attainment of nominal strength without significant loss in strength. The deformation capability is less than that of a "ductile" element, and these elements do not meet the "ductile" element criteria for a β-factor equal to 1.0 per Section 1645A for specific elements and materials.*

METHOD A *refers to the procedures contained in Sections 1645A-1647A.*

METHOD B *refers to the procedures contained in Section 1648A.*

NOMINAL STRENGTH *is the peak capacity of an element using specified material and assembly properties of the applicable materials chapters of Title 24. Examples are the flexural strength of a reinforced concrete beam M_n when the maximum concrete strain is at 0.003, or the plastic flexural capacity of a steel beam $M_p = ZF_y$ when all fibers in the section are at yield stress F_y and Z is the plastic section modulus. It is also the accepted peak strength from test results.*

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

PEER REVIEW *refers to the procedures contained in Section 1649A.*

PROBABLE STRENGTH *is the level of strength of an element likely in as-built or existing materials. For example, in reinforced concrete, it is common that actual steel yield is larger than the specified design value, and therefore probable strength is taken as equal to 1.25 times the nominal strength in flexure.*

[For DSA/SS] PROTECTION OF LIFE AND PROPERTY *is the rehabilitation of a structure to a goal of protection of life and property as a level of expected structural and nonstructural performance taken to mean: a) the building has substantial margin against either total or partial collapse of the gravity and lateral structural systems allowing occupants to exit safely; b) structural and nonstructural elements either within or outside the building have not fallen or been dislodged so as to pose a life-safety threat. It is expected that the structure may experience some repairable damage.*

[For DSA/SS] REHABILITATION *is the evaluation and retrofit of an existing nonconforming building or a school building conforming to earlier code requirements to bring the building, or portion thereof, into conformance with the safety standards of the currently effective regulations, Parts 2, 3, 4, 5, 6, 7, 8, 9 and 12, Title 24, C. C. R.*

REPAIR *as used in this division means all the design and construction work undertaken to restore or enhance the structural and nonstructural load-resisting system participating in the lateral response of a structure that has experienced damage from earthquakes or other destructive events.*

[For DSA/SS] RETROFIT *as used in this division means all design and construction work undertaken to construct any new or to repair or strengthen any existing structural or nonstructural elements required by the evaluation and design of the building.*

USABLE STRENGTH or FACTORED STRENGTH *is the product of under strength [for DSA/SS: strength-reduction] factor φ times the nominal strength in the appropriate material.*

SECTION 1642A — SYMBOLS AND NOTATIONS

1642A.1 *The following symbols and notations apply to this division in addition to those of Section 1628A:*

ϕC_n = *Usable strength or capacity of an element as determined in the materials chapters where φ is the strength reduction factor and C_n is the nominal strength.*

C_w = *Allowable or working stress resistance of an element.*

E = *Seismic load action on an element due to the specified total design base shear.*

H = *The seismic coefficient defined in Section 1643A.8.*

IDR = *Inelastic Demand Ratio.*

IDR_L = *Limit value of the IDR that an element can develop without failure.*

β = *Seismic Load Penalty Factor representing the limited inelastic deformation capability of nonductile and limited-ductile elements with respect to that of ductile elements in a given mode of failure (attainment of nominal strength).*

Ω_o = *Seismic Force Amplification Factor set forth in Table 16A-N.*

Δ_s = *Design Level Response Displacement, which is the total drift or total story drift that occurs when the structure is subjected to the specified seismic forces.*

Δ_M = Maximum Inelastic Response Displacement, which is the total drift or total story drift given by $0.7\,R\,\Delta_S$.

SECTION 1643A — CRITERIA SELECTION

1643A.1 [Not adopted by DSA/SS] Basis for Evaluation and Design. *This section determines what technical approach is to be used for the seismic evaluation and design for existing buildings. For those buildings or portions of buildings for which Section 1640A.2 requires action, the procedures and limitations for the evaluation of existing buildings and design of retrofit systems and/or repair thereof shall be implemented in accordance with this section. One of three alternative approaches must be used: the first, Method A (Sections 1644A-1647A), is prescriptive and comparable to the Division IV provisions for new structures; the second, Method B (Section 1648A), for complex or potentially hazardous situations is performance based and depends on the independent review of a peer reviewer (Section 1649A); the third is the use of one of the applicable Uniform Code for Building Conservation (UCBC) special procedures given in Section 1643A.1.1.*

[For DSA/SS] Basis for Evaluation and Design. *This section determines which technical approach may be used for the seismic evaluation and design for existing buildings. For those buildings or portions of buildings for which Section 1640A.2 requires retrofit, the procedures and limitations for the evaluation of existing buildings and design of retrofit systems and/or repair thereof shall be implemented in accordance with this section. One of three alternative approaches must be used: the first, Method A (Sections 1644A-1647A), as defined in Section 1640A.7, is prescriptive and comparable to the Part 2, Title 24, provisions for new buildings; the second, Method B (Section 1648A), as defined in Section 1640A.6, for complex or potentially hazardous situations is performance based and depends on the independent review of a peer reviewer (Section 1649A); the third is the use of Part 2, Title 24, as defined in Section 1640A.3.*

1643A.1.1 [Not adopted by DSA/SS] Special procedures. *Where there are special prescriptive procedures for the repair and/or retrofit of existing buildings as a part of these regulations, the UCBC, or accepted practice by the enforcement agent, these procedures may be used in lieu of the requirements of Chapter 34. The following special prescriptive procedures may be used for their respective types of construction to meet the requirements of this division.*

1. The UCBC for Seismic Strengthening Provisions for Unreinforced Masonry Bearing Wall Buildings (Appendix Chapter 1).

2. The UCBC for Cripple Walls and Anchor Bolts (Appendix Chapter 6).

3. The UCBC for Flexible Diaphragm—Rigid Wall Buildings (Appendix Chapter 5).

4. The SAC Interim Guidelines for the Evaluation, Repair, Modification, and Design of Welded Steel Moment Frame Structures, FEMA 267, August 1995. The ground motion specifications of this division shall be used when the SAC procedures are applied.

1643A.1.1.1 *The UCBC for Seismic Strengthening Provisions for Unreinforced Masonry Bearing Wall Buildings (Appendix Chapter 1).*

> ***[For OSHPD 1] EXCEPTION:*** *For hospital buildings, the use of unreinforced masonry wall elements is not allowed.*

1643A.1.1.2 *The UCBC for Cripple Walls and Anchor Bolts (Appendix Chapter 6). [For OSHPD 1] Where the requirements of these regulations for new construction are more restrictive, they shall govern. Section A604.4.2 of the UCBC is not adopted.*

> ***EXCEPTION:*** *Single-story wood light frame hospital buildings as defined in Section 2.2.3, Article 2, Chapter 6, Part 1, Title 24, which fail the check of Section 5.6.4, Article 5, Chapter 6, Part 1, Title 24, may be upgraded to SPC 2 by seismically retrofitting this deficiency in accordance with the provisions of the UCBC for Cripple Walls and Anchor Bolts (Appendix Chapter 6).]*

1643A.1.1.3 *The UCBC for Flexible Diaphragm—Rigid Wall Buildings. [For OSHPD 1]: Where the requirements of these regulations for new construction are more restrictive, they shall govern.*

1643A.1.1.4 *The SAC Interim Guidelines for the Evaluation, Repair, Modification, and Design of Welded Steel Moment Frame Structures, FEMA 267, August 1995. The ground motion specifications of this division shall be used when the SAC procedures are applied.*

1643A.2 Existing Conditions. *The existing condition and properties of the entire structure must be determined and documented by thorough inspection, review of all available related construction documents, and performance of necessary testing and investigations [for DSA/SS: in accordance with data collection provisions of Section 1650A]. Where samples from the existing structure are taken or in situ tests are performed, they shall be selected and interpreted in a statistically appropriate manner to ensure that the properties determined and used in the evaluation or design are representative of the conditions and structural circumstances likely to be encountered in the structure as a whole.*

The entire load path of the lateral-force-resisting system shall be determined, documented and evaluated. The load path includes all the horizontal and vertical elements participating in the structural response such as diaphragms, diaphragm chords, diaphragm drags, vertical lateral-force-resisting system (walls, frames, braces, etc.), foundations and the connection between the elements of the load path.

1643A.3 Site Geology and Soil Characteristics. *Soil profile shall be assigned in accordance with the requirements of Section 1629A.3 [for DSA/SS: where Method A or Part 2, Title 24, are used].*

1643A.4 Occupancy Categories. *For purposes of earthquake-resistant design, each structure shall be placed in one of the occupancy categories in accordance with the requirements of Section 1629A.2. [For OSHPD 1] For hospital buildings, 1=1.0 for category SPC-2 and 1=1.5 for SPC-3 through SPC-5, as determined in accordance with the requirements of Chapter 6, Part 1, title 24, Building Standards Administrative Code.*

1643A.5 Configuration Requirements. *Each structure shall be designated structurally regular or irregular in accordance with the requirements of Section 1629A.5.*

1643A.6 Selection of the Design Method. *The requirements of Method B (Section 1648A) [For DSA/SS: and Part 2, Title 24,] may be used for any existing building.*

1643A.7 *The requirements of Method A (Sections 1644A-1647A) may be used except under the following conditions, where Method B [for OSHPD 1 & 4] or Special Procedures as defined in Section 1643A.1.1 must be used.*

[For DSA/SS] *The requirements of Method A (Sections 1644A-1647A) may be used except under the following conditions, where Method B or Part 2, Title 24, shall be used.*

1643A.7.1 *When the building contains prestressed or posttensioned structural elements (beams, columns, walls or slabs) or contains precast structural elements (beams, columns, walls or flooring systems).*

1643A.7.2 When the building is classified as irregular in vertical or horizontal plan by application of Table 16A-L or 16A-M, unless the irregularity is demonstrated not to affect the seismic performance of the building.

> **EXCEPTION:** If the retrofit design removes the configurational attributes that caused the building to be classified as irregular, then Section 1643A.7.2 does not apply and Method A may be used.

1643A.7.3 [Not adopted by DSA/SS] For any building that has an importance factor I greater than 1.00 (Table 16A-K).

> **EXCEPTIONS:** 1. For hospital buildings, Method A may be used for retrofitting SPC-1 structures to SPC-2 structures where: a) the building has four or fewer stories, but with continuous diaphragms; or b) where the building is of Type V construction; or c) located in Zone 3.
>
> 2. For hospital buildings, Method A may be used for retrofit or repair of nonstructural components and systems.

1643A.7.4 For any building using undefined or hybrid structural systems. [For DSA/SS] Method B shall be used for these structural systems.

1643A.7.5 When passive or active energy absorption systems are used in the retrofit or repair, either as part of the existing structure or as part of the modifications. [For DSA/SS] Method B shall be used for these structural systems.

1643A.7.6 [Not adopted by DSA/SS] When the height of the structure exceeds 240 feet (73 152 mm).

1643A.7.6.1 [For DSA/SS] When the building exceeds three stories.

> **EXCEPTION:** 1. Any school building for which the structural system is retrofitted using Method B, Method A may be used for retrofit or repair of nonstructural components and systems.

1643A.7.7 [For DSA/SS] When the building contains unreinforced masonry.

1643A.8 Seismic Hazard Factor. The Seismic Hazard Factor, H, shall be determined according to the following procedure.

1643A.8.1 [Not adopted by DSA/SS] When the Importance Factor, I, is equal to 1, then H is equal to:

> **[For OSHPD 1] EXCEPTION:** For hospitals this value of H may be used where I is equal to 1.0 if the assigned performance category is SPC-2.

1643A.8.1.1 Three-quarters (0.75), when the seismic coefficients C_a and C_v are determined from Tables 16A-Q and 16A-R.

1643A.8.1.2 Unity (1.0) when the seismic coefficients C_a and C_v are determined from a 5 percent damped acceleration response spectrum with a 20-percent probability of exceedance in 50 years determined from a probabilistic seismic hazard analysis for the specific site. The smoothed response spectrum value at the period of 0.3 second provides the value of 2.5 C_a g, and the spectrum at 1.0 second provides the vaue of C_v g, where g is the gravity constant.

> **EXCEPTIONS:** 1. When there has been a Section 1643A.8.1.2 analysis performed, the Enforcement Agency may accept the results of this prior study on a case by case basis.
>
> 2. The results of a community-wide probabilistic seismic analysis (Section 1643A.8.1.2) may be used when the responsible enforcement agent has accepted a probabilistic seismic hazard study for the jurisdiction to determine the value required by Section 1643A.8.1.2 for sites within the jurisdiction, provided that the study on which it is based was accepted by reviewers, who were selected and charged consistent with the professional requirements of Section 1649A.

1643A.8.2 [Not adopted by DSA/SS] Otherwise, the H value is equal to unity (1.0), and the seismic coefficients C_a and C_v may be determined either from Tables 16A-Q and 16A-R or from a 5 percent damped acceleration response spectrum with a 10-percent probability of exceedance in 50 years determined from a probabilistic seismic hazard analysis for the specific site.

> **EXCEPTIONS:** 1. Exception 1 of Section 1643A.8.1.2 applies.
>
> 2. For Section 1643A.8.2, when the importance factor, I, is greater than 1 and less than or equal to 1.25, then I may be set equal to 1 for subsequent load determinations if the seismic coefficients C_a and C_v are determined from a 5 percent damped response spectrum with 10-percent probability of exceedance in 100 years determined from a probabilistic anaysis for the specific site.

1643A.8.3 [For DSA/SS] The Seismic Hazard Factor, H, is equal to 1.2.

1643A.9 Capacity Requirements. All elements of the lateral-force-resisting system must have the capacity to resist the seismic demand. Any element not having this capacity shall have its capacity increased by modifying or supplementing its capacity so that it exceeds the demand, or the demand reduced to less than the existing capacity by making other modifications to the structural system. [For DSA/SS: Any element that has experienced damage or deterioration and no longer retains the capacity to resist the seismic demand and/or gravity load shall be retrofitted in accordance with Section 1643A.10.1.]

> **EXCEPTIONS:** 1. An element's usable strength capacity may be less than that required by the specified seismic load combinations if it can be demonstrated that the associated reduction in seismic performance of the element or its removal due to the failure does not result in a structural system in which there is a life-safety hazard due to the loss of support of gravity loads; a laterally unstable structure; or falling structural or nonstructural elements or parts thereof. If this exception is taken for an element, then it cannot be considered part of the primary lateral-load-resisting system.
>
> 2. The load transferred from an adjoining element to a given element need not exceed the probable strength 1.25 C_n of the adjoining element, given that the assembly remains stable [for DSA/SS:, where Method A or Part 2, Title 24, are used]. For elements where the resistance is expressed in terms of the allowable or working stress method, the usable strength ϕC_n may be determined using an allowable stress increase of 1.70, or may be established by acceptable published factors for a given material or element, or by the use of appropriate available test data and the applicable principles of mechanics.
>
> 3. [Not adopted by DSA/SS] This requirement does not apply to a mechanical penthouse when its floor area is less than one third of that of the immediately lower floor.

1643A.10 [Not adopted by DSA] New Elements. All new elements shall either be "code-complying or ductile" or "limited-ductile," and shall be selected and designed to have compatible force-deformation performance with existing elements and nonstructural components.

> **EXCEPTION:** The use of "nonductile" elements is allowed if the particular material provides the only means of ensuring compatible performance without detrimental interaction effects on the existing element material. Code-complying or essentially code-complying details shall be used where possible.

1643A.10.1 [For DSA/SS] New or Retrofitted Elements. Any new or retrofitted structural or nonstructural element(s) shall comply with the detailing requirements for new construction of Part 2, Title 24, currently effective edition, and shall meet the capacity requirements of Section 1643A.9.

> **EXCEPTION:** 1. Where approved by the DSA, other nationally recognized standards or guidelines may be used in lieu of Part 2, Title 24, provisions.

the incremental changes of the design overturning moment shall be distributed to the various resisting elements in the manner prescribed in Section 1630A.6. Overturning effects on every element, wherever possible, shall be carried down directly in a linear path to the foundation. See load combinations in Sections 1644A.4.1.1 and 1644A.4.1.2 for combining gravity and seismic forces.

1644A.9.2 Seismic Zones 3 and 4. *In Seismic Zones 3 and 4, where a lateral-load-resisting element is discontinuous, such as for vertical irregularity Type 4 in Table 16A-L or plan irregularity Type 4 in Table 16A-M, columns supporting such elements shall have the strength to resist the axial force resulting from the following load combinations, in addition to all other applicable load combinations:*

$$\phi C_n = D + 0.8L + \Omega_o \beta E \qquad (44A\text{-}9)$$

$$\phi C_n = \Omega_o \beta E - 0.9D \qquad (44A\text{-}10)$$

$\Omega_o \beta E$ in Formulas (44A-9) and (44A-10) need not exceed RE.

1644A.9.2.1 *The axial forces in such columns need not exceed the resultant of the probable strengths of the other elements of the structure that transfer such loads to the column.*

1644A.9.2.2 *Such columns shall be capable of carrying the above-described axial forces without exceeding the usable axial load capacity (ϕC_n) of the column. For designs using working stress methods, this capacity may be determined using an allowable stress increase of 1.7 or acceptable published factors for a given material or element.*

EXCEPTION: *See Exceptions 1 and 2 in Section 1643A.9.*

1644A.9.2.3 Columns.

1644A.9.2.3.1 *Such columns shall either resist the above-described axial forces without exceeding the usable axial capacity (ϕC_n), or shall meet the following detailing and member limitations:*

1. Chapter 19, Section 1921.4, for concrete, and Chapter 22, Section 2210, 2211.4 and 2211.5, for steel in structures in Seismic Zones 3 and 4, except for welded steel moment connections where the current SAC Guidelines for columns apply.

2. Chapter 19, Section 1921.8, for concrete, and Chapter 22A, Divisions I and IX, special provisions for developing plastic hinges at ultimate loading, for steel in structures in Seismic Zone 2.

1644A.9.2.3.2 [For OSHPD 1 & 4] *In order to qualify for a β value equal to 1.0, such columns shall meet the following detailing and member limitations:*

1. Chapter 19A, Section 1921A.4, for concrete, and Chapter 22A, Section 2210A, 2211A and 2213A.5 for steel in structures in Seismic Zones 3 and 4, except for welded steel moment connections where the SAC Interim Guidelines for the Evaluation, Repair, Modification, and Design of Welded Steel Moment Frame Structures, FEMA 267, August, 1995, provisions for columns apply.

1644A.9.2.3.3 [For DSA/SS] *In order to qualify for a β value equal to 1.0, such columns shall meet the following detailing and member limitations:*

1. Chapter 19A, Section 1921A.4, for concrete, and Chapter 22A, Section 2210A, 2211A and 2213A.5, for steel in structures in Seismic Zones 3 and 4, except for welded steel moment connections where the current SAC Guidelines for the evaluation, repair, modification and design of welded steel moment frame buildings, FEMA 350, 351, 352, July 2000, provisions for columns apply.

1644A.9.2.4 *Transfer girders that support such columns or that provide support for the discontinuous lateral-load-resisting element shall resist the above-described axial forces or support reactions without exceeding the capacity ϕC_n for each mode of failure. For this case, the β factor shall correspond to the properties of the girder.*

1644A.9.3 At foundation. *See Section 1809A.4 for overturning moments to be resisted at the foundation soil interface. The foundation soil interface shall be capable of resisting the following load combinations on the allowable stress basis of Section 1809A.2 and Table 18A-I-A, and other load combinations need not apply:*

$$D + L + \frac{E}{1.4} \qquad (44A\text{-}11)$$

$$\frac{E}{1.4} - 0.9D \qquad (44A\text{-}12)$$

In order to determine the strength design basis loads for the elements of the foundation structure, the soil pressures and pile or caisson reactions due to these load combinations shall be load factored by 1.4. The resulting bending moments, shears and axial loads on the sections of the foundation structure are to be factored by the appropriate β value and shall be resisted by the corresponding usable strength ϕC_n of the section. If piles or caissons are required for overturning moment tension resistance due to the load combination (44A-12), then the minimum tensile load-carrying resistance ϕC_n shall be E/14.

1644A.10 Drift and Story Drift Limitations. *Drift or horizontal displacements of the structure shall be computed whre required by this code. For both Allowable Stress Design and Strength Design, the Maximum Inelastic Response Displacement, Δ_M, of the structure caused by the Design Basis Ground Motion shall be determined in accordance with this section. The drifts corresponding to the design seismic forces of Section 1644A.4.1, Δ_s, shall be determined in accordance with Section 1644A.10.1. To determine Δ_M, these drifts shall be amplified in accordance with Section 1644A.10.2.*

1644A.10.1 Determination of Δ_s. *A static, elastic analysis of the lateral force-resisting system shall be prepared using the design seismic forces from Section 1644A.4.1 and 1644A.6. The mathematical model shall comply with Section 1644A.2.3. The resulting deformations, denoted as Δ_s, shall be determined at all critical locations in the structure. Calculated drift shall include translational and torsional deflections.*

1644A.10.2 Determination of Δ_M. *The Maximum Inelastic Response Displacement, Δ_M, shall be computed as follows:*

$$\Delta_M = 0.7\,R\,\Delta_s \qquad (44A\text{-}13)$$

1644A.10.3 Story drift defined. *Story drift is the displacement of one level relative to the level above or below using the Maximum Inelastic Displacement, Δ_M, at each level.*

1644A.10.4 Story drift limits. *Calculated story drift using Δ_M shall not exceed 0.025 times the story height for structures having a fundamental period of less than 0.7 second. For structures having a fundamental period of 0.7 second or greater, the calculated story drift shall not exceed 0.020 times the story height.*

EXCEPTION: [Not adopted by DSA/SS] *These story drift limits may be exceeded when it is demonstrated that greater drift can be tolerated by both structural elements and nonstructural elements that could affect life safety [for OSHPD 1 & 4] for buildings in seismic performance categories SPC-1 and SPC-2, and life safety and continued operation in SPC-3 through SPC-5 buildings.*

1644A.11 PΔ Effects. *The resulting member forces and moments and the story drifts induced by PΔ effects shall be considered in the evaluation of overall structural frame stability and* *shall be evaluated using the specified design forces and their corresponding displacements Δ_s. $P\Delta$ need not be considered when the ratio of secondary moment to primary moment does not exceed 0.10; the ratio may be evaluated for any story as the product of the unfactored total dead, floor live load and snow load above the story times the seismic drift Δ_s in that story divided by the product of the seismic shear in that story times the height of that story. In Seismic Zones 3 and 4, $P\Delta$ need not be considered where the story drift ratio does not exceed 0.02/R.*

1644A.12 Vertical Component. *The following requirements apply in Seismic Zones 3 and 4 only.*

Horizontal cantilever components shall have the usable strength capacity ϕC_n to resist (0.7) $H\ C_a\ W_p$, or have an allowable or working stress capacity C_w to resist (0.5) $H\ C_a\ W_p$. The value of the seismic hazard factor H shall be as prescribed by Section 1643A.8 according to the occupancy and conditions of the building.

1644A.13 Lateral Force on Elements of Structures, Nonstructural Components and Equipment Supported by Structures. *Elements of structures and their attachments, permanent nonstructural components and their attachments, and the attachments for permanent equipment supported by a structure shall be designed to resist the total design seismic forces prescribed in Section 1644A.13.1. Attachments for floor- or roof-mounted, but not suspended, equipment weighing less than 400 pounds (181 kg), and furniture need not be designed.*

Attachments shall include anchorages and required bracing. Friction resulting from gravity loads shall not be considered to provide resistance to seismic forces.

When the failure of the lateral-force-resisting anchorage, bracing or connection of nonrigid equipment would cause a life hazard, such elements shall be designed to resist the seismic forces prescribed in Section 1644A.13.1.

When allowable design stresses and other acceptance criteria are not contained in or referenced by this code, such criteria shall be obtained from approved national standards.

1644A.13.1 Design for total lateral force.

1644A.13.1.1 [Not adopted for DSA/SS] *The total design lateral seismic force, F_p, shall be determined from the following formula:*

$$F_p = 4.0\, H\, C_a\, I_p\, W_p \qquad (44A\text{-}14)$$

Alternatively, F_p may be calculated using the following formula:

$$F_p = a_p\, H\, C_a / R_p\, (1 + 3h_x/h_r)\, W_p \qquad (44A\text{-}15)$$

Except that:

$$F_g \text{ shall not be less than } 0.7\, H\, C_a\, I_p\, W_p \text{ and Need not be more than } 4\, H\, C_a\, I_g\, W_g \qquad (44A\text{-}16)$$

WHERE:

h_x = *the element or component attachment elevation with respect to grade, h_x shall not be taken less than 0.0.*

h_r = *the structure roof elevation with respect to grade.*

a_p = *the in-structure Component Amplification Factor that varies from 1.0 to 2.5.*

A value for a_p shall be selected from Table 16A-O.

R_p is the Component Response Modification Factor that shall be taken from Table 16A-O, except that R_p for anchorages shall equal 1.5 for shallow expansion bolts, shallow chemical anchors or shallow cast-in-place anchors. Shallow anchors are those with an embedment length-to-diameter ratio of less than 8. Where anchorage is constructed of nonductile materials, or has nonductile behavior, or the component is attached with an adhesive surface joint, R_p shall equal 1.0. The β factor may be taken as 1.0 for anchorages requiring R_p equal to 1.0, 1.5 or 3.0.

The design lateral forces determined using Formula (44A-14) or (44A-15) shall be distributed in proportion to the mass distribution of the element or component.

Forces determined using Formula (44A-14) or (44A-15) shall be used to design members and connections that transfer these forces to the seismic-resisting systems. Members and connections shall use the load combinations and factors specified in Section 1644A.4.1.1 or 1644A.4.1.2. The member or connection actions due to F_p are the earthquake load E to be used in the load combinations. [For DSA/SS] The appropriate β factor shall be assigned for the elements and connections.

> ***EXCEPTION:*** *Where a probabilistic hazard analysis has been performed, the Exception 2 of Section 1643A.8.2 may be applied for the term HI_p in Formula 44-11.*

To determine the out-of-plane loading for elements such as walls or wall panels that have points of attachment at two or more different elevations, the following procedure may be used. For the vertical span of the element having a unit weight W_p between two successive attachment elevations h_x and h_{x+1} evaluate the force coefficients F_a/W_a at each of the two points, observing the minimum and maximum limits, and compute the average of the two values. The resulting average coefficient times the unit weight W_p provides the distributed seismic load for the span between the attachment points,and this load may be extended to the top of any wall parapet above the roof attachment point at h_r.

1644A.13.1.2 [For OSHPD 1 & 4] *Critical nonstructural components and systems, as defined in Table 11.1, Chapter 6, California Building Standards Administrative Code, and all components and systems in buildings in seismic performance categories SPC-3 through SPC-5 shall meet the requirements for new buildings, Section 1632A. All other elements of structures, nonstructural components and equipment supported by structures shall comply with provisions of Section 1645A.7 and this section.*

> ***EXCEPTIONS:*** *1. Anchorage and bracing of nonstructural components in buildings in seismic performance categories SPC-1 and SPC-2 with a performance level of NPC-3R may comply with the provisions of Section 1630A of the 1995 California Building Code using an importance factor I_p = 1.0. The capacity of welds, anchors and fasteners shall be determined in accordance with requirements of the 2001 California Building Code.*
>
> *2. Anchorage and bracing of nonstructural components in buildings in seismic performance categories SPC-1 and SPC-2 with a performance level of NPC-3 or higher, and SPC-3 and SPC-4, may comply with the provisions of Section 1630B of the 1998 California Building Code using an importance factor of I_p = 1.5. The capacity of welds, anchors and fasteners shall be determined in accordance with requirements of the 2001 California Building Code.*

A continuous load path of sufficient strength and stiffness between the component and the supporting structure shall be verified. Local elements of the supporting structure shall be verified for the component loads where they control the design of the elements of their connections. Increases in F_p, due to anchorage conditions (for example, shallow anchors) need not be considered. For NPC-3R, the adequacy of load path for nonstructural elements need only be verified when the total reaction at the point of support (including the application of F_p) exceeds the following limits:

1. 250 pounds for components or equipment attached to light frame walls. For the purposes of this requirement, the sum of the absolute value of all reactions due to component loads on a single stud shall not exceed 250 pounds.

2. 1,000 pounds for components or equipment attached to roofs, or walls of reinforced concrete or masonry construction.

3. 2,000 pounds for components or equipment attached to floors or slabs-on-grade.

EXCEPTION: *If the anchorage or bracing is configured in a manner that results in significant torsion on a supporting structural element, the effects of the nonstructural reaction force on the structural element shall be considered in the anchorage design.*

[For OSHPD] *The total design lateral force , F_p, shall be determined from the following formula:*

$$F_p = 4.0\,HC_aI_pW_p \qquad (44A\text{-}14)$$

Alternatively, F_p may be calculated using the following formula:

$$F_p = \frac{a_pHC_aI_p}{R_p}\left(1 + 2\frac{h_x}{h_r}\right)W_p \qquad (44A\text{-}15)$$

Except that:

F_p shall not be less than $0.7HC_aI_pW_p$ and need not be more than $4HC_aI_pW_p$. (44A-16)

WHERE:

I_p = *the value used for the structure selected from Table 16A-K.*

h_x = *the elemement or component attachment elevation with respect to grade. h_x shall not be taken less than 0.0.*

h_r = *the structure roof elevation with respect to grade. The value of h_x/h_r need not exceed 1.0.*

a_p = *the in-structure Component Amplification Factor that varies from 1.0 to 2.5. A value for a_p shall be selected from Table 16A-O.*

R_p = *the Component Response Modification Factor that shall be taken from Table 16A-0, except that R_p for anchorages shall equal 1.5 for shallow expansion bolts, shallow chemical anchors or shallow cast-in-place anchors.*

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Shallow anchors are those with an embedment length-to-diameter ratio of less than 8. Where anchorage is constructed of nonductile materials, or has nonductile behavior, or the component is attached with an adhesive surface joint, R_p shall equal 1.0.

The design lateral forces determined using Formula (44A-14) or (44A-15) shall be distributed in proportion to the mass distribution of the element or component.

Forces determined using Formula (44A-14) or (44A-15) shall be used to design members and connections that transfer these forces to the seismic-resisting sytems. Members and connections shall use the load combinations and factors specified in Section 1644A.4.1.1 or 1644A.4.1.2. The member or connection actions due to F_p are the earthquake load E to be used in the load combinations. The vertical earthquake effect E_v, shall be applied simultaneously with the horizontal earthquake effect E_h. E_v shall be taken as $0.5HC_aIW_p$ for Strength Design and $0.35HC_aIW_p$ for Allowable Stress Design. E_v shall be applied to the dead load effect D, in a manner that produces the most severe member and connection demands.

***EXCEPTION:** Where a probabilistic hazard analysis has been performed, exception 2 of Section 1643A.8.2 may be applied for the term HI_p in formula 44A-11.*

To determine the out-of-plane loading for elements such as walls or wall panels that have points of attachment at two or more different elevations, the following procedure may be used. For the vertical span of the element having a unit weight w_p between two succesive attachment elevations h_x and h_{x+i} evaluate the force coefficients F_p and W_p at each of the two points, observing the minimum and maximum limits, and compute the average of the two values. The resulting average coefficient times the unit weight w_p provides the distributed seismic load for the span between the attachment points, and this load may be extended to the top of any wall parapet above the roof attachment point at h_r.

***1644A.13.1.2.1 [For DSA/SS]** Nonstructural components and systems shall meet the requirements for new buildings, Section 1632A, or comply with provisions of Section 1645A.8 and this section.*

The total design lateral force, F_p, shall be determined from the following formula:

$$F_p = 4.0\, HC_a I_p W_p \qquad \text{(44A-14)}$$

Alternatively, Fp may be calculated using the following formula:

$$F_p = \frac{a_p\, HC_a I_p}{R_p}\left(1 + 2\frac{h_x}{h_r}\right)W_p \qquad \text{(44A-15)}$$

Except that:

F_p shall not be less than $0.7HC_aI_pW_p$ and need not be more than $4HC_aI_pW_p$. (44A-16)

WHERE:

I_p = the value used for the structure selected from Table 16A-K.

h_x = the element or component attachment elevation with respect to grade. h_x shall not be taken less than 0.0.

h_r = the structure roof elevation with respect to grade. The value of h_x / h_r need not exceed 1.0.

a_p = the in-structure Component Amplification Factor that varies from 1.0 to 2.5. A value for a_p shall be selected from Table 16A-O.

R_p = the Component Response Modification Factor that shall be taken from Table 16A-0, except that R_p for anchorages shall equal 1.5 for shallow expansion bolts, shallow chemical anchors or shallow cast-in-place anchors. Shallow anchors are those with an embedment length-to-diameter ratio of less than 8. Where anchorage is constructed of nonductile materials, or has nonductile behavior, or the component is attached with an adhesive surface joint, R_p shall equal 1.0.

The design lateral forces determined using Formula (44A-14) or (44A-15) shall be distributed in proportion to the mass distribution of the element or component.

Forces determined using Formula (44A-14) or (44A-15) shall be used to design members and connections that transfer these forces to the seismic-resisting systems. Members and connections shall use the load combinations and factors specified in Section 1644A.4.1.1 or 1644A.4.1.2. The member or connection actions due to F_p are the earthquake load E to be used in the load combinations. The vertical earthquake effect E_v, shall be applied simultaneously with the horizontal earthquake effect E_h. E_v shall be taken as $0.5HC_aIW_p$ for Strength Design and 0.35HCaIWp for Allowable Stress Design. E_v shall be applied to the dead load effect D, in a manner that produces the most severe member and connection demands.

To determine the out-of-plane loading for elements such as walls or wall panels that have points of attachment at two or more different elevations, the following procedure may be used. For the vertical span of the element having a unit weight w_p between two successive attachment elevations h_x and h_{x+i} evaluate the force coefficients F_p/W_p at each of the two points, observing the minimum and maximum limits, and compute the average of the two values. The resulting average coefficient times the unit weight w_p provides the distributed seismic load for the span between the attachment points, and this load may be extended to the top of any wall parapet above the roof attachment point at h_r.

SECTION 1645A — PROCEDURES FOR THE CLASSIFICATION OF ELEMENTS INTO THE DUCTILE, LIMITED-DUCTILE AND NONDUCTILE CATEGORIES

***1645A.1 General.** All elements will be classified as either being "ductile, limited-ductile, or nonductile." The purpose of this section is to provide the procedures and guidelines necessary for this classification and assignment of β values. The general requirements for all materials are listed below and will be followed by the specific requirements for each material. [For DSA/SS: Information from data collection (Sections 1643A.2 and 1650A) shall be used to establish the β-values.]*

***1645A.1.1 Ductile category.** A ductile element is one that complies with the definition of ductile. Code-complying elements shall be classified as ductile, except as noted in Section 1644A.9.2.3. Otherwise, a rational analysis, as described in the nonductile category below, may be used to justify the use of the ductile classification.*

***1645A.1.2 Nonductile category.** Any element that does not comply with the code-compliant definition shall be classified as nonductile; except for the case where it either complies with the specific provisions of Section 1645A required for the limited-ductile category, or a rational analysis based on the principles of mechanics, related research and test results can demonstrate that it has the cyclic inelastic deformation behavior required for the limited-ductile or ductile categories.*

***1645A.1.3 Limited-ductile category.** An element that does not qualify as ductile, but does comply or essentially complies with the specific material limited-ductile provisions of Section 1645A, may* *be classified as limited-ductile. Otherwise, a rational analysis as described in the nonductile category above may be used to justify the use of the limited-ductile classification.*

1645A.2 *For each element and loading condition, a β value is assigned that represents the expected load-deflection behavior of the element during the full earthquake loading of the element, including repeated, reversing loads. β values that are significantly different from those given in Section 1645A must receive the acceptance of the enforcement agency when they are used in the analysis and design.*

[For DSA/SS] *For each element and loading condition, a β value is assigned that represents the expected load-deflection behavior of the element during the full earthquake loading of the element, including repeated, reversing loads. β values that are different from those given in Section 1645A must receive the acceptance of the DSA when they are used in the analysis and design.*

1645A.2.1 *Sections 1645A.3 through 1645A.6.2 provide reference values for selected elements and loading conditions; these β values are to be used as guidance for the assignment of values for conditions and elements not listed by comparison of expected performance to that expected for listed elements.*

1645A.2.2 *Alternative β values to those listed may be used where experimental results, coupled with rational analysis, lead to the conclusion that a different β value better represents the behavior of a given element and its conditions. Such interpretation and analysis shall be subject to the review and approval of the enforcement agent and shall consider the following items:*

1. The effects of cyclic load reversals representative of seismic loading beyond the strength level of the element, considering the specific nature of the loading used in the test, especially whether essentially static or dynamic.

2. The size or scale effect of the test data, along with the compatibility of the test specimen details with those of the existing element.

3. The sample size of the test program and range of related test variables necessary to reasonably define behavior.

1645A.3 Reinforced Concrete. *Reinforced concrete is considered to be any combination of concrete with steel reinforcing that can develop the compressive and tensile properties of the respective materials. The procedures and provisions for the classification of ductile, limited-ductile and nonductile elements are given in Sections 1645A.3.1 through 1645A.3.1.4. The corresponding β values are given in Table 16A-R-1.*

1645A.3.1 Reinforced concrete frame elements.

1645A.3.1.1 [Not adopted by DSA/SS] *Any frame element in conformance with the requirements of 1976 UBC Section 2626 or later editions (Sections 1921A.1 through 1921A.5 for Seismic Zones 3 and 4) may be classified as ductile and the β value taken as 1.0.*

> ***EXCEPTIONS:*** *1. Hooked bar development length shall comply with Section 1921A.5.4 to qualify the bar anchorage as ductile.*
>
> *2. For a column to be classified as ductile, no more than one-third of the columns in a story level of its frame-line may have the weak column-strong beam condition; otherwise, each column in the story level frame-line shall be classified as no more than limited ductile.*

1645A.3.1.1.1 [For DSA/SS] *Any frame element in conformance with the requirements of 1985 UBC Section 2625 or later editions may be classified as ductile and the β value taken as 1.0.*

> ***EXCEPTION:*** *For a column to be classified as ductile, no more than one-third of the columns in a story level of its frame-line may have the weak column-strong beam condition; otherwise, each column in the story level frame-line shall be classified as no more than limited ductile.*

1645A.3.1.2 *Any frame element in essential conformance with the requirements of Section 1921A.8 [for DSA/SS: UBC Section 1921.8] or equivalent requirements of earlier editions, shall be classified as limited ductile and assigned a β value equal to or greater than that given in Table 16A-R-1.*

1645A.3.1.3 *Any column members in essential compliance with the requirements of Sections 1921A.7.2 and 1921A.7.3 shall be classified as limited-ductile and assigned a β value equal to or greater than that given in Table 16A-R-1.*

1645A.3.1.4 *Any element not meeting the requirements of Section 1645A.3.1.1, 1645A.3.1.2 or 1645A.3.1.3 shall be classified as nonductile, with corresponding β value equal to or greater than that given in Table 16A-R-1, except where Section 1645A.2 allows use of another value. The Section 1645A.2.2 analysis shall consider at a minimum:*

1. Reinforcing bar lap splice length, cover and ties.

2. Pile-to-footing connection resistance to tension due to overturning moment (Section 1644A.9.3).

3. Footing flexural and shear capacity.

4. Column ties for both shear resistance and concrete confinement.

5. Positive moment tension bar pullout or slab flexural failure. (Section 1646A.1.3.2)

6. Negative moment hook pullout.

7. Stirrups for both shear resistance and concrete confinement.

8. Noncontinuous longitudinal steel leaving sections with weakness in flexural and shear resistance (Section 1921.8.4.1).

9. Joint shear reinforcing and confinement.

10. Weak column-strong beam condition (Sections 1645A.3.1.1, Exception 2, and 1921A.4.2.2).

11. Slab punching shear.

12. Short or captive column.

13. The shear capacity of columns.

1645A.3.2 Shear walls and diaphragms.

1645A.3.2.1 [Not adopted by DSA/SS] *Any shear wall or diaphragm in conformance with the requirements of the 1976 UBC Section 2626 or later editions (Section 1921.6) may be classified as ductile and the β value taken as 1.0.*

> ***EXCEPTION:*** *A shear wall shall essentially meets the boundary zone requirements of Section 1921.6.6 to be classified as ductile.*

1645A.3.2.1.1 [For DSA/SS] *Any shear wall or diaphragm in conformance with the requirements of the 1985 UBC Section 2625 or later editions may be classified as ductile and the β value taken as 1.0.*

> ***EXCEPTION:*** *A shear wall that essentially meets the boundary zone requirements of Section 1921.6.6 may be classified as ductile.*

1645A.3.2.2 *Any shear wall or diaphragm in conformance with 1976 UBC Section 2614 may be classified as a limited-ductile element and assigned a β value equal to or greater than that given in Table 16A-R-1.*

1645A.3.2.3 *Any wall element not meeting the requirements of Section 1645A.3.2.1 or 1645A.3.2.2 shall be classified as nonductile, with corresponding β value equal to or greater than that given in Table 16A-R-1, except where Section 1645A.2 allows use of another value. The Section 1645A.2.2 analysis shall consider at a minimum:*

1. Dowel and reinforcing bar lap splice length, cover and ties.

meets the protection of life and property for a seismic event based on ground shaking having a 10 percent probability of exceedance in 50 years and the maximum considered earthquake at the performance level for collapse prevention.

1648A.1.2 [For DSA/SS] *The evaluation and retrofit design provisions of FEMA 356, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings," November 2000, shall be used for evaluation and retrofit of the existing building; except that the ground motion characterization shall be in accordance with Section 1648A.2.2. Any of the methodologies contained in FEMA 356, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings," hereafter referred to as the "approach," may be used subject to the approval of the peer reviewer (Section 1649A) and the DSA in accordance with the procedures of Section 1640A.8 and the provisions of this division.*

For application of the procedures of FEMA 356 to structural elements, the acceptance criteria factor (e.g. m, ε, rotation) for the protection of life and property shall be at a performance level between the life safety (LS) and immediate occupancy (IO) performance levels and shall be interpolated as follows:

Acceptance criteria factor (e.g. m, ε, rotation) = LS - 0.33 (IO – LS), where the factors for systems and components are defined in the material chapters of FEMA 356.

> ***EXCEPTION:*** *An alternative evaluation and retrofit methodology that will yield a structure of equal or greater reliability than a structure evaluated and retrofitted to FEMA 356 may be used subject to the approval of the peer reviewer(s) and the DSA in accordance with the procedures of Section 1640A.8.*

1648A.2 [Not adopted by DSA/SS] *The approach, models, analysis procedures, assumptions on material and system behavior, and conclusions shall be peer reviewed in accordance with the requirements of Section 1649A and accepted by the peer reviewer(s) [OSHPD 1: and/or Enforcement Agent].*

> ***EXCEPTIONS:*** *1. The enforcement agency may perform the work of peer review when qualified staff is available within the jurisdiction.*
>
> *2. The enforcement agency may modify or waive the requirements for peer review when appropriate.*

1648A.2.1 [For DSA/SS] *The approach, models, analysis procedures, assumptions on material and system behavior, and conclusions shall be peer reviewed in accordance with the requirements of Section 1649A and accepted by DSA.*

> ***EXCEPTION:*** *When determined appropriate by DSA, DSA may perform the work of peer review.*

1648A.2.2 [For DSA/SS] *The following provisions apply to wood and light-gage metal frame buildings:*

1. The linear procedures of FEMA 356 may be used for evaluation and retrofit of wood and light-gage metal frame buildings. Nonlinear procedures shall not apply.

2. Lateral force resisting diaphragms and shear wall systems shall be in accordance with Section 1643A.15.

1648A.2.3 [Not adopted by DSA/SS] *The approach used in the development of the design shall be acceptable to the peer reviewer. Approaches that are specifically tailored to the type of building, construction materials and specific building characteristics may be used, if they are acceptable to the independent peer reviewer. Section 1648A.3 provides several approaches that may be considered. The following conditions apply to whatever approach is selected.*

1648A.2.3.1 *If load (e.g., R, β) factors, capacity reduction factors (e.g., ϕ), or measures of inelastic deformation capability (e.g., IDR_L, μ_L, ε_L, rotation, θ_L) are used, the basis for their use and the specific values assigned shall be assessed and supported in a consistent manner.*

1648A.2.3.2 *Where dynamic time history analysis is used, at least three distinct representative records with simultaneous loadings in different directions, as appropriate, shall be used in the analysis. The maximum response parameter of interest shall be used for design.*

1648A.2.3.3 *When an elastic analysis approach is adopted, the stiffness characteristics for the elements of the elastic model should be representative of the inelastic behavior at the maximum response for the strength degrading materials and the nominal strength deformation for nondegrading materials. The following items are given for consideration:*

1. For reinforced concrete frame elements and reinforced concrete and masonry shear wall elements, this stiffness may be taken as one-half of that of the gross section or that of the cracked section. A more appropriate value may be used if justified by analysis.

2. Steel framing and bracing elements are to have their elastic section stiffness.

3. Steel-framing elements encased in reinforced concrete are to have the composite section stiffness which may be taken as 1.3 times the concrete gross-section stiffness, and beam-column joints may be assumed to be rigid.

4. Framing elements shall have model lengths equal to the clear span length, or have a suitable rigid element representation of the joint configuration.

5. If framing element connections and/or supports are not fully rigid, then these shall be modeled as springs.

6. The representation of foundation flexibility shall be included when it results in more than a 25-percent reduction in the assumed full fixity of supported elements. This includes the effects of both rotational and horizontal deformations and sliding.

1648A.2.3.4 *Reliable capacities shall be used for all elements, consistent with the fundamental behavior of the element and/or system under reversing loads at the design level of earthquake loads.*

1648A.2.3.5 *The value of the earthquake loading of an element need not exceed the force action induced in the element when the inelastic structure is displaced due to the prescribed ground motions, and the elements are assigned their probable strength values.*

1648A.2.3.6 *All nonstructural elements that can affect life safety shall be shown to have acceptable behavior in the design loadings. For structural elements not considered as part of the lateral-load-resisting system, the requirements of Section 1644A.13 are sufficient to meet this requirement.*

1648A.2.4 *The ground motion characterization used for Method B shall be consistent with this section.*

1648A.2.4.1 [For OSHPD 1] *The ground motion characterization used for Method B shall be based on ground shaking having a 10 percent probability of exceedance in 50 years for category SPC 2 at the essential life-safety performance level. For SPC 3 through SPC 5, the ground motion characterization used for Method B shall be based on ground shaking having a 10 percent probability of exceedance in 50 years at the immediate occupancy performance level and the maximum considered earthquake at the collapse prevention performance level.*

Ground shaking having a 10 percent probability of exceedance in 50 years need not exceed $^2/_3$ of the maximum considered earthquake.

Ground shaking response spectra for use in Method B shall be determined in accordance with either the General Procedure of Section 1648A.2.4.2 or the Site-Specific Procedure of Section 1648A.2.4.3.

In the General Procedure, ground shaking hazard is determined from the response spectrum acceleration contour maps. Maps showing 5-percent-damped response spectrum ordinates for short-period (0.2 second) and long-period (1 second) response distributed by FEMA for use with the "NEHRP Guidelines for the Seismic Rehabilitation of Buildings" (FEMA 273) shall be used directly with the General Procedure of Section 1648A.2.4.2 for developing design response spectra for either or both the 10 percent probability of exceedance in 50 years and the maximum considered earthquake. In the Site-Specific Procedure, ground shaking hazard is determined using a specific study of the faults and seismic source zones that may affect the site, as well as evaluation of the regional and geologic conditions that affect the character of the site ground motion caused by events occurring on these faults and sources.

The General Procedure may be used for any building except as specified below. The Site-Specific Procedure may also be used for any building and shall be required where any of the following apply:

1. The building is category SPC 5.

2. The building site is located within 10 kilometers of an active fault.

3. The building is located on Type E soils (as defined in Section 1648A.2.4.2) and the mapped maximum considered earthquake spectral response acceleration at short periods (S_s) exceeds 2.0g.

4. The building is located on Type F soils as defined in Section 1648A.2.4.2.

> ***EXCEPTION:*** *Where S_s determined in accordance with Section 1648A.2.4.2, < 0.20g. In these cases, a Type E soil profile may be assumed.*

5. A time-history response analysis of the building is performed as part of the design.

1648A.2.4.1.1 [For DSA/SS] *The ground motion characterization used for Method B shall be based on ground shaking having a 10 percent probability of exceedance in 50 years at a performance level for the protection of life and property and the maximum considered earthquake at the performance level for collapse prevention.*

Ground shaking having a 10 percent probability of exceedance in 50 years need not exceed 2/3 of the maximum considered earthquake. Ground shaking response spectra for use in Method B shall be determined in accordance with either the General Procedure of Section 1648A.2.4.2.1 or the Site-Specific Procedure of Section 1648A.2.4.3.1.

In the General Procedure, ground shaking hazard is determined from the response spectrum acceleration contour maps. Maps showing 5-percent-damped response spectrum ordinates for short-period (0.2 second) and long-period (1 second) response distributed by FEMA for use with FEMA 356, "Prestandard and Commentary for the Seismic Rehabilitation of Existing Buildings" shall be used directly with the General Procedure of Section 1648A.2.2.2.1 for developing design response spectra for either or both the 10 percent probability of exceedance in 50 years and the maximum considered earthquake. In the Site-Specific Procedure, ground shaking hazard is determined using a specific study of the faults and seismic source zones that may affect the site, as well as evaluation of the regional and geologic conditions that affect the character of the site ground motion caused by events occurring on these faults and sources.

The General Procedure may be used for any building except as specified below. The Site-Specific Procedure may also be used for any building and shall be required where any of the following apply:

1. The building site is located within 10 kilometers of an active fault.

2. The building is located on Type E soils (as defined in Section 1648A.4.2.2.1) and the mapped maximum considered earthquake spectral response acceleration at short periods (S_s) exceeds 2.0g.

3. The building is located on Type F soils as defined in Section 1648A.4.2.2.1.

> ***EXCEPTION:*** *Where S_s determined in accordance with Section 1648A.4.2.2.1, < 0.20g. In these cases, a Type E soil profile may be assumed.*

4. A time-history response analysis of the building is performed as part of the design.

1648A.2.4.2 [For OSHPD 1] General procedure to determine the acceleration response spectra. *The general procedures of this section shall be used to determine the acceleration response spectra.*

Deterministic estimates of earthquake hazard, in which an acceleration response spectrum is obtained for a specific magnitude earthquake occurring on a defined fault, shall be made using the Site-Specific Procedures of Section 1648A.2.4.3.

The mapped short-period response acceleration parameter, S_s, and mapped response acceleration parameter at a 1-second period, S_1, for 10 percent probability of exceedance in 50 years ground motion shall be obtained directly from the maps distributed by FEMA for use with the "NEHRP Guidelines for the Seismic Rehabilitation of Buildings" (FEMA 273). The mapped short-period response acceleration parameter, S_s, and mapped response acceleration parameter at a 1-second period, S_1, for the maximum considered earthquake shall also be obtained directly from the maps.

Parameters S_s and S_1 shall be obtained by interpolating between the values shown on the response acceleration contour lines on either side of the site, on the appropriate map, or by using the value shown on the map for the higher contour adjacent to the site.

The mapped short-period response acceleration parameter, S_s, and mapped response acceleration parameter at a 1-second period, S_1, for 10 percent probability of exceedance in 50 years ground shaking hazards shall be taken as the smaller of the following:

1. The values of the parameters S_s and S_1, respectively, determined for 10 percent probability of exceedance in 50 years ground motion.

2. Two-thirds of the values of the parameters S_s and S_1, respectively, determined from the maximum considered earthquake ground motion map.

The design short-period spectral response acceleration parameter, S_{xs}, and the design spectral response acceleration parameter at 1 second, S_{x1}, shall be obtained, respectively, from Equations (48A-1) and (48A-2) as follows:

$$S_{xs} = F_a S_s \qquad (48A\text{-}1)$$

$$S_{x1} = F_v S_1 \qquad (48A\text{-}2)$$

where F_a and F_v are site coefficients determined respectively from Tables 16A-R-3 and 16A-R-4, based on the site class and the values of the response acceleration parameters S_s and S_1.

Site classes shall be defined as follows:

Class A: *Hard rock with measured shear wave velocity, $\bar{v}_s$ > 5,000 ft/sec (1524 m/s).*

Class B: *Rock with 2,500 ft/sec (762 m/s) <$\bar{v}_s$ < 5,000 ft/sec (1524 m/s).*

ation of substantial portions of the design and/or analysis work that is to be reviewed, and review shall start as soon as practical after Method B is adopted and sufficient information defining the project is available.

1649A.2.1 [For DSA/SS] Timing of Independent Review. *The peer reviewer(s) shall be retained in a timely manner to provide services in accordance with the procedures specified in Section 1640A.8.3.*

1649A.3 Qualifications and Terms of Employment. *The reviewer shall be independent from the design and construction team.*

1649A.3.1 *The reviewer(s) shall have no other involvement in the project before, during or after the review, except in a review capacity.*

1649A.3.2 *The reviewer shall be selected and paid by the owner and shall have technical expertise in repair of buildings similar to the one being reviewed, as determined by the responsible enforcement agent.*

1649A.3.3 *The reviewer (or in the case of review teams, the chair) shall be a California-licensed structural engineer who is familiar with the technical issues and regulations governing the work to be reviewed.*

1649A.3.4 *The reviewer shall serve through completion of the project and shall not be terminated except for failure to perform the duties specified herein. Such termination shall be in writing with copies to the enforcement agent, owner, and the engineer of record. When a reviewer is terminated or resigns, a qualified replacement shall be appointed within 10 working days. [For DSA/SS: If the reviewer resigns or is terminated by the owner prior to completion of the project, then the reviewer shall submit copies of all reports, notes and correspondence to the design professional in responsible charge, the owner and DSA within 10 working days of such termination.]*

1649A.4 [Not adopted by DSA/SS] Scope of Review. *Review activities shall include, where appropriate, available construction documents, observations of the condition of the structure, all inspection and testing reports, including methods of sampling, analyses prepared by the engineer of record and consultants, and the retrofit or repair design. Review shall include consideration of the proposed design approach, methods, materials and details.*

1649A.4.1 [For DSA/SS] Scope of Review. *Review activities shall include, where appropriate, available new and original construction documents, observations of the condition of the structure, all new and original inspection and testing reports, including methods of sampling, and analyses prepared by the project structural engineer and consultants. Review shall consider the proposed design approach, retrofit or repair methods, materials and details for appropriateness to the performance objectives. Where required by DSA, changes observed during construction that affect the seismic-resisting system or the approved retrofit shall be reported to the peer reviewer by the design professional for review and recommendations.*

1649A.5 [Not adopted by DSA/SS] Reports. *The reviewer(s) shall prepare a written report to the owner and responsible enforcement agent that covers all aspects of the review performed, including conclusions reached by the reviewer. Reports shall be issued after the schematic phase, during design development, and at the completion of construction documents, but prior to their issuance for permit. Such reports should include, at the minimum, statements of the following.*

1. Scope of engineering design peer review with limitations defined.

2. The status of the project documents at each review stage.

3. Ability of selected materials and framing systems to meet performance criteria with given loads and configuration.

4. Degree of structural system redundancy and the deformation compatibility among structural and nonstructural elements.

5. Basic constructibility of the retrofit or repair system.

6. Other recommendations that would be appropriate to the specific project.

7. Presentation of the conclusions of the reviewer identifying any areas that need further review, investigation and/or clarification.

8. Recommendations.

1649A.5.1 [For DSA/SS] Reports. *The reviewer(s) shall prepare a written report to the owner and DSA that covers all aspects of the review performed, including conclusions reached by the reviewer, in accordance with Section 1640A.8.3. Such reports shall address the following.*

1. Scope of engineering design peer review performed during phase of work.

2. The status of the project documents and/or analyses at each review stage.

3. Ability of structural and nonstructural materials and framing systems to meet the performance objective.

4. Basic constructability of the retrofit or repair system.

5. Recommendations that would be appropriate to the specific project.

6. Presentation of the conclusions of the reviewer identifying any areas that need further review, investigation and/or clarification.

7. Compliance with the evaluation and retrofit report criteria per Section 1640A.8.

1649A.6 [Not adopted by DSA/SS] Responses and Corrective Actions. *The engineer of record shall review the report from the reviewer(s) and shall develop corrective actions and other responses as appropriate. Changes observed during construction that affect the seismic-resisting system shall be reported to the reviewer in writing for review and recommendations. All reports, responses and corrective actions prepared pursuant to this section shall be submitted to the responsible enforcement agent and the owner along with other plans, specifications and calculations required. If the reviewer resigns or is terminated by the owner prior to completion of the project, then the reviewer shall submit copies of all reports, notes and correspondence to the responsible enforcement agent, the owner, and the engineer of record within 10 working days of such termination.*

1649A.6.1 [For DSA/SS] Responses and Corrective Actions. *The project structural engineer shall review the report from the peer reviewer(s) and shall develop corrective actions and other responses as appropriate. During the design development and construction document phases, all reports, responses and corrective actions prepared pursuant to this section shall be submitted to the project design professional, the owner and DSA.*

1649A.7 [For DSA/SS] Resolution of Conflicts. *When the conclusions and recommendations of the peer reviewer conflict with the design professional's proposed design, the DSA shall make the final determination of the requirements for the design.*

SECTION 1650A — [FOR DSA] DATA COLLECTION

1650A.1 Data Collection*. Data collection shall be performed to determine the as-built conditions and material properties and assess the condition of the structural and non-structural components of the existing building. Knowledge of construction and material properties shall be determined for all components and connections of the lateral load resisting system and those components of the gravity load resisting system, exterior elements and the nonstructural elements that may affect the strength and stiffness of the lateral system, and/or create falling hazards to the building occupants.*

Data collection shall be directed and observed by the project structural engineer or design professional in general responsible charge of the design.

1650A.2 Data collection requirements. *Data collection of the as-built conditions shall address the following:*

1. Information shall be obtained, to the extent possible, from original construction documents including design drawings, specifications, material test records and quality assurance reports covering original construction and subsequent modifications to the structure. The information shall be verified, or when missing, determined, by visual and comprehensive condition assessment and by comprehensive material testing in accordance with Sections 1650A.3 and 1650A.4, respectively. Qualified test data from the original construction may be accepted, in part or in whole, by DSA to fulfill the requirements of the comprehensive material testing. Comprehensive condition assessment and material testing shall meet the "comprehensive" requirements of FEMA 356, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings," November 2000.

> ***EXCEPTIONS:*** *1. Alternative recognized processes for condition assessment and/or material testing may be used when approved by DSA.*
>
> *2. Welded steel moment frame connections of buildings that may have experienced potentially damaging ground motions shall be inspected in accordance with Chapters 3 and 4, FEMA 352, Recommended Post Earthquake Evaluation and Repair Criteria for Welded Moment-Frame Construction for Seismic Applications (July 2000). If any damage is observed, then each connection in the lateral force-resisting frame shall be inspected.*

2. In absence of qualified material test records and quality assurance reports, material properties shall be determined by comprehensive materials testing. The coefficient of variation in material test results shall be less than 20%, unless accepted by the peer reviewer and the DSA.

3. Information on adjacent buildings may be obtained through field surveys and available as-built information.

4. Information on the existing foundations and site-related items may be obtained from existing documents, geohazard and geotechnical reports, field surveys or a program of subsurface investigation. Where geohazard and geotechnical reports are not available, these reports may be required by the DSA for existing sites in accordance with Section 4-317(e), Article 1, Group 1, Part 1, Title 24.

5. Repaired or retrofitted elements shall be identified, and the standards under which the work was constructed shall be determined by review of construction documents and visual or comprehensive condition assessment.

1650A.3 Condition assessment. *Condition assessment is the determination of both physical configuration and physical condition of structural and nonstructural elements and components. Condition assessment shall be based on visual and comprehensive (destructive and nondestructive) examination.*

Visual assessment shall include direct visual inspection of assessable components and connections and indirect visual inspection where coverings or other obstructions exist. Direct visual inspection of accessible components and connections shall be performed to identify configurational issues, dimensions and degradation of material. Direct visual inspection may require removal of finish materials or fireproofing to obtain access. Indirect visual assessment may require the use of scoping equipment to inspect through an obstruction.

Comprehensive assessment may require destructive and nondestructive investigation processes or removal of localized structural material to expose the component of the element under consideration. Destructive and/or non-destructive testing may be used to determine material components and any damage or deterioration of the element.

The minimum number of samples for condition assessment to be performed shall meet the "comprehensive" requirements of FEMA 356, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings" as specified in the respective material chapters.

> ***EXCEPTIONS:*** *1. When damage or deterioration is observed, the DSA may require inspection of additional elements that may be similarly affected.*
>
> *2. Deviation in the number of samples for condition assessment may be approved by DSA when it has been determined that adequate information regarding the condition of the element has been obtained.*
>
> *3. Welded steel moment frame connections of buildings that may have experienced potentially damaging ground motions shall be inspected in accordance with Section 1650A.1, Item 1, Exception 2.*

Condition assessment shall include:

1. Characterization of the site soil.

2. Examination of the physical condition of components that will receive seismic-induced forces or deformations to identify the presence of degradation due to environmental or loading effects.

3. Verification of the presence and configuration of structural elements and components and their connections; and the continuity of load paths between components, elements and systems.

4. Identification of conditions that may influence building performance including: a) the presence of neighboring party walls and buildings, b) the presence of nonstructural components, c) prior modifications or additions, d) the presence and attachment of veneer.

1650A.4 Material Testing. *Properties of materials and/or assemblies shall be determined and/or quantified by testing of specific components and/or mock-up assemblies.*

The minimum number of samples for any material tests to be performed shall meet the "comprehensive" requirements of FEMA 356, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings" as specified in the respective material chapters. The location and number of the material or assembly samples to be tested shall be indicated in the Evaluation and Design Criteria Report per Section 1640A.8.

> ***EXCEPTION:*** *Deviation in the number of samples for material testing may be approved by DSA when it has been determined by DSA that adequate information identifying the material properties has been obtained.*

EXCEPTION: When justified in accordance with Section 1807.11, the allowable axial stress may be increased above 12,600 psi (86.88 MPa) and $0.35F_y$, but shall not exceed $0.5F_y$.

1808.6.3 Minimum dimensions. Sections of driven H-piles shall comply with the following:

1. The flange projection shall not exceed 14 times the minimum thickness of metal in either the flange or the web, and the flange widths shall not be less than 80 percent of the depth of the section.

2. The nominal depth in the direction of the web shall not be less than 8 inches (203 mm).

3. Flanges and webs shall have a minimum nominal thickness of $^3/_8$ inch (9.5 mm).

Sections of driven pipe piles shall have an outside diameter of not less than 10 inches (254 mm) and a minimum thickness of not less than $^1/_4$ inch (6.4 mm).

1808.7 Concrete-filled Steel Pipe Piles.

1808.7.1 Material. The concrete-filled steel pipe piles shall conform to UBC Standard 22-1 and shall be identified in accordance with Section 2202.2. The concrete-filled steel pipe piles shall have a specified compressive strength f'_c of not less than 2,500 psi (17.24 MPa).

1808.7.2 Allowable stresses. The allowable axial stresses shall not exceed 0.35 of the minimum specified yield strength F_y of the steel plus 0.33 of the specified compressive strength f'_c of concrete, provided F_y shall not be assumed greater than 36,000 psi (248.22 MPa) for computational purposes.

EXCEPTION: When justified in accordance with Section 2807.11, the allowable stresses may be increased to 0.50 F_y.

1808.7.3 Minimum dimensions. Driven piles of uniform section shall have a nominal outside diameter of not less than 8 inches (203 mm).

SECTION 1809 — FOUNDATION CONSTRUCTION—SEISMIC ZONES 3 AND 4

1809.1 General. In Seismic Zones 3 and 4 the further requirements of this section shall apply to the design and construction of foundations, foundation components and the connection of superstructure elements thereto.

1809.2 Soil Capacity. The foundation shall be capable of transmitting the design base shear and overturning forces prescribed in Section 1630 from the structure into the supporting soil. The short-term dynamic nature of the loads may be taken into account in establishing the soil properties.

1809.3 Superstructure-to-Foundation Connection. The connection of superstructure elements to the foundation shall be adequate to transmit to the foundation the forces for which the elements were required to be designed.

1809.4 Foundation-Soil Interface. For regular buildings, the force F_t as provided in Section 1630.5 may be omitted when determining the overturning moment to be resisted at the foundation-soil interface.

1809.5 Special Requirements for Piles and Caissons.

1809.5.1 General.

1809.5.1.1 Piles, caissons and caps shall be designed according to the provisions of Section 1603, including the effects of lateral displacements. Special detailing requirements as described in Section 1809.5.2 shall apply for a length of piles equal to 120 percent of the flexural length. Flexural length shall be considered as a length of pile from the first point of zero lateral deflection to the underside of the pile cap or grade beam. CA

1809.5.1.2 [For BSC] Piles, caissons and caps shall be designed according to the provisions of Section *1605*, including the effects of lateral displacements. Special detailing requirements as described in Section 1809.5.2 shall apply for a length of piles equal to 120 percent of the flexural length. Flexural length shall be considered as a length of pile from the first point of zero lateral deflection to the underside of the pile cap or grade beam. CAC

1809.5.2 Steel piles, nonprestressed concrete piles and prestressed concrete piles.

1809.5.2.1 Steel piles. Piles shall conform to width-thickness ratios of stiffened, unstiffened and tubular compression elements as shown in Chapter 22, Division VIII.

1809.5.2.2 Nonprestressed concrete piles. Piles shall have transverse reinforcement meeting the requirements of Section 1921.4.

EXCEPTION: Transverse reinforcement need not exceed the amount determined by Formula (21-2) in Section 1921.4.4.1 for spiral or circular hoop reinforcement or by Formula (21-4) in Section 1921.4.4.1 for rectangular hoop reinforcement.

1809.5.2.3 Prestressed concrete piles. Piles shall have a minimum volumetric ratio of spiral reinforcement no less than 0.021 for 14-inch (356 mm) square and smaller piles, and 0.012 for 24-inch (610 mm) square and larger piles unless a smaller value can be justified by rational analysis. Interpolation may be used between the specified ratios for intermediate sizes.

TABLE 18-I-A—ALLOWABLE FOUNDATION AND LATERAL PRESSURE

CLASS OF MATERIALS[1]	ALLOWABLE FOUNDATION PRESSURE (psf)[2] × 0.0479 for kPa	LATERAL BEARING LBS./SQ./FT./FT. OF DEPTH BELOW NATURAL GRADE[3] × 0.157 for kPa per meter	LATERAL SLIDING[4] Coefficient[5]	LATERAL SLIDING[4] Resistance (psf)[6] × 0.0479 for kPa
1. Massive crystalline bedrock	4,000	1,200	0.70	
2. Sedimentary and foliated rock	2,000	400	0.35	
3. Sandy gravel and/or gravel (GW and GP)	2,000	200	0.35	
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	1,500	150	0.25	
5. Clay, sandy clay, silty clay and clayey silt (CL, ML, MH and CH)	1,000[7]	100		130

[1]For soil classifications OL, OH and PT (i.e., organic clays and peat), a foundation investigation shall be required.
[2]All values of allowable foundation pressure are for footings having a minimum width of 12 inches (305 mm) and a minimum depth of 12 inches (305 mm) into natural grade. Except as in Footnote 7, an increase of 20 percent shall be allowed for each additional foot (305 mm) of width or depth to a maximum value of three times the designated value. Additionally, an increase of one third shall be permitted when considering load combinations, including wind or earthquake loads, as permitted by Section 1612.3.2.
[3]May be increased the amount of the designated value for each additional foot (305 mm) of depth to a maximum of 15 times the designated value. Isolated poles for uses such as flagpoles or signs and poles used to support buildings that are not adversely affected by a $^1/_2$-inch (12.7 mm) motion at ground surface due to short-term lateral loads may be designed using lateral bearing values equal to two times the tabulated values.
[4]Lateral bearing and lateral sliding resistance may be combined.
[5]Coefficient to be multiplied by the dead load.
[6]Lateral sliding resistance value to be multiplied by the contact area. In no case shall the lateral sliding resistance exceed one half the dead load.
[7]No increase for width is allowed.

TABLE 18-I-B—CLASSIFICATION OF EXPANSIVE SOIL

EXPANSION INDEX	POTENTIAL EXPANSION
0-20	Very low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very high

TABLE 18-I-C—FOUNDATIONS FOR STUD BEARING WALLS—MINIMUM REQUIREMENTS[1,2,3]

NUMBER OF FLOORS SUPPORTED BY THE FOUNDATION[4]	THICKNESS OF FOUNDATION WALL (inches) × 25.4 for mm: Concrete	THICKNESS OF FOUNDATION WALL (inches) × 25.4 for mm: Unit Masonry	WIDTH OF FOOTING (inches) × 25.4 for mm	THICKNESS OF FOOTING (inches) × 25.4 for mm	DEPTH BELOW UNDISTURBED GROUND SURFACE (inches) × 25.4 for mm
1	6	6	12	6	12
2	8	8	15	7	18
3	10	10	18	8	24

[1]Where unusual conditions or frost conditions are found, footings and foundations shall be as required in Section 1806.1.
[2]The ground under the floor may be excavated to the elevation of the top of the footing.
[3]Interior stud bearing walls may be supported by isolated footings. The footing width and length shall be twice the width shown in this table and the footings shall be spaced not more than 6 feet (1829 mm) on center.
[4]Foundations may support a roof in addition to the stipulated number of floors. Foundations supporting roofs only shall be as required for supporting one floor.

Chapter 18A *[For DSA/SS, OSHPD]*
FOUNDATIONS AND RETAINING WALLS

NOTES: 1. This chapter is applicable to public schools and state-owned or state-leased essential services buildings regulated by the Division of the State Architect, Structural Safety Section.

2. This chapter is applicable to hospitals, skilled nursing facilities, intermediate-care facilities and correctional treatment centers regulated by the Office of Statewide Health Planning and Development.

[For OSHPD 2] EXCEPTION: Single-story Type V skilled nursing or intermediate care facilities utilizing wood-frame or light-steel-frame construction as defined in Health and Safety Code Section 129725, which shall comply with UBC Chapter 18 and any applicable amendments therein.

Division I—GENERAL

SECTION 1801A — SCOPE

1801A.1 General.

1801A.1.1 This chapter sets forth requirements for excavation and fills for any building or structure and for foundations and retaining structures.

Refer to Appendix Chapter 33 for requirements governing excavation, grading and earthwork construction, including fills and embankments.

1801A.2 Standards of Quality. The standards listed below labeled a "UBC Standard" are also listed in Chapter 35, Part II, and are part of this code.

1. **Testing.**

   1.1 UBC Standard 18-1, Soils Classification

   1.2 UBC Standard 18-2, Expansion Index Test

SECTION 1802A — QUALITY AND DESIGN

The quality and design of materials used structurally in excavations, footings and foundations shall conform to the requirements specified in Chapters 16A, 19A, 21A, 22A and 23A.

Excavations and fills shall comply with Chapter 33 *and Appendix Chapter 33*.

Allowable bearing pressures, allowable stresses and design formulas provided in this chapter shall be used with the allowable stress design load combinations specified in Section 1612A.3.

NOTE: 1. See Section 1611A.6 for retaining walls.

2. See Sections 1701A.5, Item 11, and 1809A.6 for pile inspection requirements. See Section 1701A.5, Item 11, and 1809A.7 for caisson inspection requirements.

SECTION 1803A — SOIL CLASSIFICATION—EXPANSIVE SOIL

1803A.1 General. For the purposes of this chapter, the definition and classification of soil materials for use in Table 18A-I-A shall be according to UBC Standard 18-1.

1803A.2 Expansive Soil. When the expansive characteristics of a soil are to be determined, the procedures shall be in accordance with UBC Standard 18-2 and the soil shall be classified according to Table 18A-I-B. Foundations for structures resting on soils with an expansion index greater than 20, as determined by UBC Standard 18-2, shall require special design consideration. If the soil expansion index varies with depth, the variation is to be included in the engineering analysis of the expansive soil effect upon the structure.

SECTION 1804A — FOUNDATION INVESTIGATION

1804A.1 General. *Soil investigation reports which include foundation or pile capacity recommendations, recommendations regarding installation, and, in the case of engineered fills, directions as to materials and construction procedures shall be prepared by a geotechnical engineer qualified to undertake investigations for foundation and earthwork design. Investigations involving test borings, exploration shafts or load tests shall be made under the engineering control of such a geotechnical engineer.*

Site investigations and reports pertaining to geologic hazards shall be made where required by Sections 17212 and 17212.5 of the Education Code for public school buildings, Sections 129775 and 129780 of the Health and Safety Code for hospital buildings, and Section 16014 of the Health and Safety Code for essential services buildings. See also Sections 1629A.3 and 1637A.

1804A.2 Investigation. *Whenever it is necessary to make special investigations, sufficient borings or exploration shafts shall be made as deemed necessary by the geotechnical engineer to evaluate the character of the soil under the entire building or structure, except that there shall not be less than one boring or exploration shaft for each 5,000 square feet (465 m^2) of building area at the foundation level with a minimum of two provided for any one building. The possibility of liquefaction under seismic disturbance shall be considered in the investigation. If there is a potential for liquefaction, the geotechnical engineer shall report the estimated amount of displacement.*

Borings shall be of sufficient size to permit visual examination of the soil in place or, in lieu thereof, cores shall be taken.

1804A.3 Reports. The soil classification and design-bearing capacity *of the soil* shall be shown on the plans * * *. The *enforcement agency* may require submission of a written report of the investigation, which shall include, but need not be limited to, the following information:

1. A plot showing the location of all test borings and/or excavations.

2. *Logs of all explorations, including* descriptions and classifications of the materials encountered.

3. Elevation of the water table, if encountered.

4. Recommendations for foundation type and design criteria, including bearing capacity, provisions to mitigate the effects of expansive soils, provisions to mitigate the effects of liquefaction and soil strength, the effects of adjacent loads*, and additional earth pressure caused by seismic ground shaking where required by Section 1611.A.6.*

5. Expected total and differential settlement.

6. The report shall consider the effects of stepped footings addressed in Section 1806A.4.

7. The geotechnical engineer shall assess the potential for liquefaction during the geotechnical investigation for the project. Any significant liquefaction hazard shall be considered in arriving at an appropriate foundation system and design parameters and in evaluating foundation performance. The geotechnical report should include estimated amounts of foundation settlement and differential settlement and recommended means for mitigating settlements associated with potential liquefaction.

8. In areas subject to high sulphate soils, an evaluation of the impact on the durability of concrete foundations shall be considered.

1804A.4 Expansive Soils. When expansive soils are present, the *enforcing agency* may require that special provisions be made in the foundation design and construction to safeguard against damage due to this expansiveness. The *enforcement agency* may require a special investigation and report to provide these design and construction criteria.

1804A.5 Liquefaction Potential and Soil Strength Loss. When required by Section 1804A.2, the potential for soil liquefaction and soil strength loss during earthquakes shall be evaluated during the geotechnical investigation. The geotechnical report shall assess potential consequences of any liquefaction and soil strength loss, including estimation of differential settlement, lateral movement or reduction in foundation soil-bearing capacity, and discuss mitigating measures. Such measures shall be given consideration in the design of the building and may include, but are not limited to, ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements, or any combination of these measures.

The potential for liquefaction and soil strength loss shall be evaluated for a site peak ground acceleration that, as a minimum, conforms to the probability of exceedance specified in Section 1631A.2. Peak ground acceleration may be determined based on a site-specific study taking into account soil amplification effects. In the absence of such a study, peak ground acceleration may be assumed equal to the seismic zone factor in Table 16A-I.

1804A.6 Adjacent Loads. Where footings are placed at varying elevations, the effect of adjacent loads shall be included in the foundation design.

1804A.7 Drainage. Provisions shall be made for the control and drainage of surface water around buildings. (See also Section 1806A.5.5.)

SECTION 1805A — ALLOWABLE FOUNDATION AND LATERAL PRESSURES

The allowable foundation and lateral pressures shall not exceed the values set forth in Table 18A-I-A unless data to substantiate the use of higher values are submitted. Table 18A-I-A may be used for design of foundations on rock or nonexpansive soil for Type II One-hour, Type II-N and Type V buildings that do not exceed three stories in height or for structures that have continuous footings having a load of less than 2,000 pounds per lineal foot (29.2 kN/m) and isolated footings with loads of less than 50,000 pounds (222.4 kN).

Allowable bearing pressures provided in Table 18A-I-A shall be used with the allowable stress design load combinations specified in Section 1612A.3.

SECTION 1806A — FOOTINGS

1806A.1 General. Footings and foundations shall be constructed of masonry *or* concrete *unless prior approval has been obtained from the enforcement agency* and shall extend below the frost line. *The horizontal dimensions of unformed concrete footings shall be increased 1 inch (25 mm) at every vertical surface at which concrete is placed directly against the soil.*

All exterior footings shall extend into firm bearing in undisturbed soil or controlled compacted fill, or if in expansive soil, to a depth sufficient to minimize movements in the underlying soil due to moisture changes in the soil. Footings shall have a minimum depth below finished grade as indicated in Table 18A-I-C unless another depth is recommended by a soils report. Foundation walls supporting wood shall extend above grade as required by Section 2306A.4.

The provisions of this section do not apply to building and foundation systems in those areas subject to scour and water pressure by wind and wave action. Buildings and foundations subject to such loads shall be designed in accordance with approved national standards. See Section 3302 for subsoil preparation and wood form removal.

1806A.2 Footing Design. Except for special provisions of Section 1808A covering the design of piles, all portions of footings shall be designed in accordance with the structural provisions of this code and shall be designed to minimize differential settlement when necessary and the effects of expansive soils when present.

The enforcing agency may require an elastic analysis at footing and grade beam elements to determine subgrade deformations in order to evaluate their effect on the superstructure drift values in Chapter 16A.

Slab-on-grade and mat-type footings for buildings located on expansive soils may be designed in accordance with the provisions of Division III or such other engineering design based on geotechnical recommendation as approved by the *enforcement agency.*

1806A.3 Bearing Walls. Bearing walls shall be supported on masonry or concrete foundations or piles or other approved foundation system that shall be of sufficient size to support all loads. Where a design is not provided, the minimum foundation requirements for stud bearing walls shall be as set forth in Table 18A-I-C.

All shear walls shall be supported on footings or foundations as required for bearing walls.

1806A.4 Stepped Foundations. Foundations for all buildings where the surface of the ground slopes more than 1 unit vertical in 10 units horizontal (10% slope) shall be level or shall be stepped so that both top and bottom of such foundation are level.

Individual steps in continuous footings shall not exceed 18 inches (457 mm) in height and the slope of a series of such steps shall not exceed 1 unit vertical to 2 units horizontal (50% slope) unless otherwise recommended by a soils report. The steps shall be detailed on the drawings. The local effects due to the discontinuity of the steps shall be considered in the design of the foundation.

1806A.5 Footings on or Adjacent to Slopes.

1806A.5.1 Scope. The placement of buildings and structures on or adjacent to slopes steeper than 1 unit vertical in 3 units horizontal (33.3% slope) shall be in accordance with this section.

1806A.5.2 Building clearance from ascending slopes. In general, buildings below slopes shall be set a sufficient distance from the slope to provide protection from slope drainage, erosion and shallow failures. Except as provided for in Section 1806A.5.6 and Figure 18A-I-1, the following criteria will be assumed to provide ting shall be carried out in such a manner that the carrying capacity of existing piles and structures shall not be impaired. After withdrawal of the jet, piles shall be driven down until the required resistance is obtained.

1807A.9 Protection of Pile Materials. Where the boring records of site conditions indicate possible deleterious action on pile materials because of soil constituents, changing water levels or other factors, such materials shall be adequately protected by methods or processes approved by the *enforcement agency*. The effectiveness of such methods or processes for the particular purpose shall have been thoroughly established by satisfactory service records or other evidence which demonstrates the effectiveness of such protective measures.

1807A.10 Allowable Loads. The allowable loads based on soil conditions shall be established in accordance with Section 1807A.

> **EXCEPTION:** Any uncased cast-in-place pile may be assumed to develop a frictional resistance equal to one sixth of the bearing value of the soil material at minimum depth as set forth in Table 18A-I-A but not to exceed 500 pounds per square foot (24 kPa) unless a greater value is allowed by the *enforcement agency* after a soil investigation as specified in Section 1804A is submitted. Frictional resistance and bearing resistance shall not be assumed to act simultaneously unless recommended after a foundation investigation as specified in Section 1804A.

1807A.11 Use of Higher Allowable Pile Stresses. Allowable compressive stresses greater than those specified in Section 1808A shall be permitted when substantiating data justifying such higher stresses are submitted to and approved by the *enforcement agency*. Such substantiating data shall include a foundation investigation including a report in accordance with Section 1807A.1 by a soils engineer defined as a civil engineer experienced and knowledgeable in the practice of soils engineering.

SECTION 1808A — SPECIFIC PILE REQUIREMENTS

1808A.1 Round Wood Piles.

1808A.1.1 Material. Except where untreated piles are permitted, wood piles shall be pressure treated. Untreated piles may be used only when it has been established that the cutoff will be below lowest groundwater level assumed to exist during the life of the structure.

1808A.1.2 Allowable stresses. The allowable unit stresses for round wood piles shall not exceed those set forth in Chapter 23A, Division III, Part I.

The allowable values listed in Chapter 23A, Division III, Part I, for compression parallel to the grain at extreme fiber in bending are based on load sharing as occurs in a pile cluster. For piles which support their own specific load, a safety factor of 1.25 shall be applied to compression parallel to the grain values and 1.30 to extreme fiber in bending values.

1808A.2 Uncased Cast-in-place Concrete Piles.

1808A.2.1 Material. Concrete piles cast in place against earth in drilled or bored holes shall be made in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. The length of such pile shall be limited to not more than 30 times the average diameter. Concrete shall have a specified compressive strength f'_c of not less than 2,500 psi (17.24 MPa).

> **EXCEPTION:** The length of pile may exceed 30 times the diameter provided the design and installation of the pile foundation is in accordance with an approved investigation report.

1808A.2.2 Allowable stresses. The allowable compressive stress in the concrete shall not exceed $0.33f'_c$. The allowable compressive stress of reinforcement shall not exceed 34 percent of the yield strength of the steel or 25,500 psi (175.7 MPa).

1808A.3 Metal-cased Concrete Piles.

1808A.3.1 Material. Concrete used in metal-cased concrete piles shall have a specified compressive strength f'_c of not less than 2,500 psi (17.24 MPa).

1808A.3.2 Installation. Every metal casing for a concrete pile shall have a sealed tip with a diameter of not less than 8 inches (203 mm).

Concrete piles cast in place in metal shells shall have shells driven for their full length in contact with the surrounding soil and left permanently in place. The shells shall be sufficiently strong to resist collapse and sufficiently watertight to exclude water and foreign material during the placing of concrete.

Piles shall be driven in such order and with such spacing as to ensure against distortion of or injury to piles already in place. No pile shall be driven within four and one-half average pile diameters of a pile filled with concrete less than 24 hours old unless approved by the *enforcement agency.*

1808A.3.3 Allowable stresses. Allowable stresses shall not exceed the values specified in Section 1808A.2.2, except that the allowable concrete stress may be increased to a maximum value of $0.40f'_c$ for that portion of the pile meeting the following conditions:

1. The thickness of the metal casing is not less than 0.068 inch (1.73 mm) (No. 14 carbon sheet steel gage).

2. The casing is seamless or is provided with seams of equal strength and is of a configuration that will provide confinement to the cast-in-place concrete.

3. The specified compressive strength f'_c shall not exceed 5,000 psi (34.47 MPa) and the ratio of steel minimum specified yield strength f_y to concrete specified compressive strength f'_c shall not be less than 6.

4. The pile diameter is not greater than 16 inches (406 mm).

1808A.4 Precast Concrete Piles.

1808A.4.1 Materials. Precast concrete piles shall have a specified compressive strength f'_c of not less than 3,000 psi (20.68 MPa), and shall develop a compressive strength of not less than 3,000 psi (20.68 MPa) before driving.

1808A.4.2 Reinforcement ties. The longitudinal reinforcement in driven precast concrete piles shall be laterally tied with steel ties or wire spirals. Ties and spirals shall not be spaced more than 3 inches (76 mm) apart, center to center, for a distance of 2 feet (610 mm) from the ends and not more than 8 inches (203 mm) elsewhere. The gage of ties and spirals shall be as follows:

For piles having a diameter of 16 inches (406 mm) or less, wire shall not be smaller than 0.22 inch (5.6 mm) (No. 5 B.W. gage).

For piles having a diameter of more than 16 inches (406 mm) and less than 20 inches (508 mm), wire shall not be smaller than 0.238 inch (6.0 mm) (No. 4 B.W. gage).

For piles having a diameter of 20 inches (508 mm) and larger, wire shall not be smaller than $^1/_4$ inch (6.4 mm) round or 0.259 inch (6.6 mm) (No. 3 B.W. gage).

1808A.4.3 Allowable stresses. Precast concrete piling shall be designed to resist stresses induced by handling and driving as well as by loads. The allowable stresses shall not exceed the values specified in Section 1808A.2.2.

1808A.5 Precast Prestressed Concrete Piles (Pretensioned).

1808A.5.1 Materials. Precast prestressed concrete piles shall have a specified compressive strength f'_c of not less than 5,000 psi (34.48 MPa) and shall develop a compressive strength of not less than 4,000 psi (27.58 MPa) before driving.

1808A.5.2 Reinforcement. The longitudinal reinforcement shall be high-tensile seven-wire strand. Longitudinal reinforcement shall be laterally tied with steel ties or wire spirals.

Ties or spiral reinforcement shall not be spaced more than 3 inches (76 mm) apart, center to center, for a distance of 2 feet (610 mm) from the ends and not more than 8 inches (203 mm) elsewhere.

At each end of the pile, the first five ties or spirals shall be spaced 1 inch (25 mm) center to center.

For piles having a diameter of 24 inches (610 mm) or less, wire shall not be smaller than 0.22 inch (5.6 mm) (No. 5 B.W. gage). For piles having a diameter greater than 24 inches (610 mm) but less than 36 inches (914 mm), wire shall not be smaller than 0.238 inch (6.0 mm) (No. 4 B.W. gage). For piles having a diameter greater than 36 inches (914 mm), wire shall not be smaller than $^1/_4$ inch (6.4 mm) round or 0.259 inch (6.6 mm) (No. 3 B.W. gage).

1808A.5.3 Allowable stresses. Precast prestressed piling shall be designed to resist stresses induced by handling and driving as well as by loads. The effective prestress in the pile shall not be less than 400 psi (2.76 MPa) for piles up to 30 feet (9144 mm) in length, 550 psi (3.79 MPa) for piles up to 50 feet (15 240 mm) in length, and 700 psi (4.83 MPa) for piles greater than 50 feet (15 240 mm) in length.

The compressive stress in the concrete due to externally applied load shall not exceed:

$$f_c = 0.33f'_c - 0.27fp_c$$

WHERE:

fp_c = effective prestress stress on the gross section.

Effective prestress shall be based on an assumed loss of 30,000 psi (206.85 MPa) in the prestressing steel. The allowable stress in the prestressing steel shall not exceed the values specified in Section 1918A.

1808A.6 Structural Steel Piles.

1808A.6.1 Material. Structural steel piles, steel pipe piles and fully welded steel piles fabricated from plates shall conform to UBC Standard 22-1 and be identified in accordance with Section 2202A.2.

1808A.6.2 Allowable stresses. The allowable axial stresses shall not exceed 0.35 of the minimum specified yield strength F_y or 12,600 psi (86.88 MPa), whichever is less.

> **EXCEPTION:** When justified in accordance with Section 1807A.11, the allowable axial stress may be increased above 12,600 psi (86.88 MPa) and $0.35F_y$, but shall not exceed $0.5F_y$.

1808A.6.3 Minimum dimensions. Sections of driven H-piles shall comply with the following:

1. The flange projection shall not exceed 14 times the minimum thickness of metal in either the flange or the web, and the flange widths shall not be less than 80 percent of the depth of the section.

2. The nominal depth in the direction of the web shall not be less than 8 inches (203 mm).

3. Flanges and webs shall have a minimum nominal thickness of $^3/_8$ inch (9.5 mm).

Sections of driven pipe piles shall have an outside diameter of not less than 10 inches (254 mm) and a minimum thickness of not less than $^1/_4$ inch (6.4 mm).

1808A.7 Concrete-filled Steel Pipe Piles.

1808A.7.1 Material. The concrete-filled steel pipe piles shall conform to UBC Standard 22-1 and shall be identified in accordance with Section 2202A.2. The concrete-filled steel pipe piles shall have a specified compressive strength f'_c of not less than 2,500 psi (17.24 MPa).

1808A.7.2 Allowable stresses. The allowable axial stresses shall not exceed 0.35 of the minimum specified yield strength F_y of the steel plus 0.33 of the specified compressive strength f'_c of concrete, provided F_y shall not be assumed greater than 36,000 psi (248.22 MPa) for computational purposes.

> **EXCEPTION:** When justified in accordance with Section 2807.11, the allowable stresses may be increased to 0.50 F_y.

1808A.7.3 Minimum dimensions. Driven piles of uniform section shall have a nominal outside diameter of not less than 8 inches (203 mm).

SECTION 1809A — FOUNDATION CONSTRUCTION—SEISMIC ZONES 3 AND 4

1809A.1 General. In Seismic Zones 3 and 4 the further requirements of this section shall apply to the design and construction of foundations, foundation components and the connection of superstructure elements thereto.

1809A.2 Soil Capacity. The foundation shall be capable of transmitting the design base shear and overturning forces prescribed in Section 1630A from the structure into the supporting soil. The short-term dynamic nature of the loads may be taken into account in establishing the soil properties.

1809A.3 Superstructure-to-Foundation Connection. The connection of superstructure elements to the foundation shall be adequate to transmit to the foundation the forces for which the elements were required to be designed.

1809A.4 Foundation-Soil Interface. For regular buildings, the force F_t as provided in Section 1630A.5 may be omitted when determining the overturning moment to be resisted at the foundation-soil interface.

1809A.5 Special Requirements for Piles and Caissons.

1809A.5.1 General. Piles, caissons and caps shall be designed according to the provisions of Section *1605A*, including the effects of lateral displacements. *Whenever such members are founded in Type S_D, S_E, or S_F soils,* special detailing requirements as described in Section 1809A.5.2 shall apply for a length of *such members* equal to 120 percent of the flexural length. Flexural length shall be considered as a length of pile from the first point of zero lateral deflection to the underside of the pile cap or grade beam.

1809A.5.2 Steel piles, nonprestressed concrete piles and prestressed concrete piles.

1809A.5.2.1 Steel piles. Piles shall conform to width-thickness ratios of stiffened, unstiffened and tubular compression elements as shown in Chapter 22A, Division *IX.*

1809A.5.2.2 Nonprestressed concrete piles. Piles shall have transverse reinforcement meeting the requirements of Section 1921A.4.

> **EXCEPTION:** Transverse reinforcement need not exceed the amount determined by Formula (21-2) in Section 1921A.4.4.1 for spi-

1903.5.5 Prestressing tendons.

1903.5.5.1 1. ASTM A 416, Uncoated Seven-wire Stress-relieved Steel Strand for Prestressed Concrete

2. ASTM A 421, Uncoated Stress-relieved Wire for Prestressed Concrete

3. ASTM A 722, Uncoated High-strength Steel Bar for Prestressing Concrete

1903.5.5.2 Wire, strands and bars not specifically listed in *ASTM A 416, A 421 and A 722* may be used, provided they conform to minimum requirements of these specifications and do not have properties that make them less satisfactory than those listed.

1903.5.6 Structural steel, steel pipe or tubing.

1903.5.6.1 For structural steel used with reinforcing bars in composite compression members meeting requirements of Section 1910.16.7 or 1910.16.8, see *ASTM A 36, A 242, A 572 and A 588.*

1903.5.6.2 For steel pipe or tubing for composite compression members composed of a steel-encased concrete core meeting requirements of Section 1910.16.4, see *ASTM A 53, A 500 and A 501.*

1903.5.7 *UBC Standard 19-1, Welding Reinforcing Steel, Metal Inserts and Connections in Reinforced Concrete Construction*

1903.6 Admixtures.

1903.6.1 Admixtures to be used in concrete shall be subject to prior approval by the *building official.*

1903.6.2 An admixture shall be shown capable of maintaining essentially the same composition and performance throughout the work as the product used in establishing concrete proportions in accordance with Section 1905.2.

1903.6.3 Calcium chloride or admixtures containing chloride from other than impurities from admixture ingredients shall not be used in prestressed concrete, in concrete containing embedded aluminum, or in concrete cast against stay-in-place galvanized steel forms. See Sections 1904.3.2 and 1904.4.1.

1903.6.4 ASTM C 260, Air-entraining Admixtures for Concrete

1903.6.5 ASTM C 494 and C 1017, Chemical Admixtures for Concrete

1903.6.6 ASTM C 618, Fly Ash and Raw or Calcined Natural Pozzolans for Use as Admixtures in Portland Cement Concrete

1903.6.7 *ASTM C 989,* Ground-iron Blast-furnace Slag for Use in Concrete and Mortars

1903.6.8 Admixtures used in concrete containing ASTM C 845 expansive cements shall be compatible with the cement and produce no deleterious effects.

1903.6.9 Silica fume used as an admixture shall conform to ASTM C 1240 (*Silica Fume for Use in Hydraulic Cement Concrete and Mortar*).

1903.7 Storage of Materials.

1903.7.1 Cementitious materials and aggregate shall be stored in such manner as to prevent deterioration or intrusion of foreign matter.

1903.7.2 Any material that has deteriorated or has been contaminated shall not be used for concrete.

1903.8 Concrete Testing.

1. ASTM C 192, Making and Curing Concrete Test Specimens in the Laboratory

2. ASTM C 31, Making and Curing Concrete Test Specimens in the Field

3. ASTM C 42, Obtaining and Testing Drilled Cores and Sawed Beams of Concrete

4. ASTM C 39, Compressive Strength of Cylindrical Concrete Specimens

5. ASTM C 172, Sampling Freshly Mixed Concrete

6. ASTM C 496, Splitting Tensile Strength of Cylindrical Concrete Specimens

7. ASTM C 1218, Water-Soluble Chloride in Mortar and Concrete

1903.9 Concrete Mix.

1. *ASTM C 94, Ready-mixed Concrete*

2. ASTM C 685, Concrete Made by Volumetric Batching and Continuous Mixing

3. *UBC Standard 19-2, Mill-mixed Gypsum Concrete and Poured Gypsum Roof Diaphragms*

4. ASTM C 109, Compressive Strength of Hydraulic Cement Mortars

5. ASTM C 567, Unit Weight of Structural Lightweight Concrete

1903.10 Welding. *The welding of reinforcing steel, metal inserts and connections in reinforced concrete construction shall conform to UBC Standard 19-1.*

1903.11 Glass Fiber Reinforced Concrete.

1903.11.1 Recommended Practice for Glass Fiber Reinforced Concrete Panels, Manual 128. CA

1903.11.2 [For BSC] Recommended Practice for Glass Fiber Reinforced Concrete Panels, *PCI* Manual 128. CAC

SECTION 1904 — DURABILITY REQUIREMENTS

1904.0 Notation.

f'_c = specified compressive strength of concrete, psi (MPa).

1904.1 Water-Cementitious Materials Ratio.

1904.1.1 The water-cementitious materials ratios specified in Tables 19-A-2 and 19-A-4 shall be calculated using the weight of cement meeting ASTM C 150, C 595 or C 845 plus the weight of fly ash and other pozzolans meeting ASTM C 618, slag meeting ASTM C 989, and silica fume meeting ASTM C 1240, if any, except that when concrete is exposed to deicing chemicals, Section 1904.2.3 further limits the amount of fly ash, pozzolans, silica fume, slag or the combination of these materials.

1904.2 Freezing and Thawing Exposures.

1904.2.1 Normal-weight and lightweight concrete exposed to freezing and thawing or deicing chemicals shall be air entrained with air content indicated in Table 19-A-1. Tolerance on air content as delivered shall be ± 1.5 percent. For specified compressive strength f'_c greater than 5,000 psi (34.47 MPa), reduction of air content indicated in Table 19-A-1 by 1.0 percent shall be permitted.

1904.2.2 Concrete that will be subjected to the exposures given in Table 19-A-2 shall conform to the corresponding maximum water-cementitious materials ratios and minimum specified concrete compressive strength requirements of that table. In addition, concrete that will be exposed to deicing chemicals shall conform to the limitations of Section 1904.2.3.

1904.2.3 For concrete exposed to deicing chemicals, the maximum weight of fly ash, other pozzolans, silica fume or slag that is included in the concrete shall not exceed the percentages of the total weight of cementitious materials given in Table 19-A-3.

1904.3 Sulfate Exposure.

1904.3.1 Concrete to be exposed to sulfate-containing solutions or soils shall conform to the requirements of Table 19-A-4 or shall be concrete made with a cement that provides sulfate resistance and that has a maximum water-cementitious materials ratio and minimum compressive strength set forth in Table 19-A-4.

1904.3.2 Calcium chloride as an admixture shall not be used in concrete to be exposed to severe or very severe sulfate-containing solutions, as defined in Table 19-A-4.

1904.4 Corrosion Protection of Reinforcement.

1904.4.1 For corrosion protection of reinforcement in concrete, maximum water soluble chloride ion concentrations in hardened concrete at ages from 28 to 42 days contributed from the ingredients, including water, aggregates, cementitious materials and admixtures shall not exceed the limits of Table 19-A-5. When testing is performed to determine water soluble chloride ion content, test procedures shall conform to ASTM C 1218.

1904.4.2 If concrete with reinforcement will be exposed to chlorides from deicing chemicals, salt, salt water, brackish water, sea water or spray from these sources, requirements of Table 19-A-2 for water-cementitious materials ratio and concrete strength and the minimum concrete cover requirements of Section 1907.7 shall be satisfied. In addition, see Section 1918.14 for unbonded prestressed tendons.

SECTION 1905 — CONCRETE QUALITY, MIXING AND PLACING

1905.0 Notations.

f'_c = specified compressive strength of concrete, psi (MPa).

f'_{cr} = required average compressive strength of concrete used as the basis for selection of concrete proportions, psi (MPa).

f_{ct} = average splitting tensile strength of lightweight aggregate concrete, psi (MPa).

s = standard deviation, psi (MPa).

1905.1 General.

1905.1.1 Concrete shall be proportioned to provide an average compressive strength as prescribed in Section 1905.3.2, as well as satisfy the durability criteria of Section 1904. Concrete shall be produced to minimize frequency of strengths below f'_c as prescribed in Section 1905.6.2.3.

1905.1.2 Requirements for f'_c shall be based on tests of cylinders made and tested as prescribed in Section 1905.6.2.

1905.1.3 Unless otherwise specified, f'_c shall be based on 28-day tests. If other than 28 days, test age for f'_c shall be as indicated in design drawings or specifications.

Design drawings shall show specified compressive strength of concrete f'_c for which each part of structure is designed.

1905.1.4 Where design criteria in Sections 1909.5.2.3, 1911.2; and 1912.2.4, provide for use of a splitting tensile strength value of concrete, laboratory tests shall be made to establish value of f_{ct} corresponding to specified values of f'_c.

1905.1.5 Splitting tensile strength tests shall not be used as a basis for field acceptance of concrete.

1905.2 Selection of Concrete Proportions.

1905.2.1 Proportions of materials for concrete shall be established to provide:

1. Workability and consistency to permit concrete to be worked readily into forms and around reinforcement under conditions of placement to be employed without segregation or excessive bleeding.

2. Resistance to special exposures as required by Section 1904.

3. Conformance with strength test requirements of Section 1905.6.

1905.2.2 Where different materials are to be used for different portions of proposed work, each combination shall be evaluated.

1905.2.3 Concrete proportions, including water-cementitious materials ratio, shall be established on the basis of field experience and/or trial mixtures with materials to be employed (see Section 1905.3), except as permitted in Section 1905.4 or required by Section 1904.

1905.3 Proportioning on the Basis of Field Experience and Trial Mixtures.

1905.3.1 Standard deviation.

1905.3.1.1 Where a concrete production facility has test records, a standard deviation shall be established. Test records from which a standard deviation is calculated:

1. Must represent materials, quality control procedures and conditions similar to those expected, and changes in materials and proportions within the test records shall not have been more restricted than those for proposed work.

2. Must represent concrete produced to meet a specified strength or strengths f'_c within 1,000 psi (6.89 MPa) of that specified for proposed work.

3. Must consist of at least 30 consecutive tests or two groups of consecutive tests totaling at least 30 tests as defined in Section 1905.6.1.4, except as provided in Section 1905.3.1.2.

1905.3.1.2 Where a concrete production facility does not have test records meeting requirements of Section 1905.3.1.1, but does have a record based on 15 to 29 consecutive tests, a standard deviation may be established as the product of the calculated standard deviation and the modification factor of Table 19-A-6. To be acceptable, the test record must meet the requirements of Section 1905.3.1.1, Items 1 and 2, and represent only a single record of consecutive tests that span a period of not less than 45 calendar days.

1905.3.2 Required average strength.

1905.3.2.1 Required average compressive strength f'_{cr} used as the basis for selection of concrete proportions shall be the larger of Formula (5-1) or (5-2) using a standard deviation calculated in accordance with Section 1905.3.1.1 or 1905.3.1.2.

$$f'_{cr} = f'_c + 1.34s \tag{5-1}$$

or

$$f'_{cr} = f'_c + 2.33s - 500 \tag{5-2}$$

For **SI:** $f'_{cr} = f'_c + 2.33s - 3.45$

1905.3.2.2 When a concrete production facility does not have field strength test records for calculation of standard deviation meeting requirements of Section 1905.3.1.1 or 1905.3.1.2, required average strength f'_{cr} shall be determined from Table 19-B permitted to reduce the resulting computed moments in such proportion that the absolute sum of the positive and average negative moments used in the design need not exceed the value obtained from Formula (13-3).

1913.7.7.5 Distribution of moments at critical sections across the slab-beam strip of each frame to column strips, beams and middle strips as provided in Sections 1913.6.4, 1913.6.5 and 1913.6.6 shall be permitted if the requirement of Section 1913.6.1.6 is satisfied.

SECTION 1914 — WALLS

1914.0 Notations.

A_g = gross area of section, square inches (mm^2).

f'_c = specified compressive strength of concrete, pounds per square inch (MPa).

h = overall thickness of member, inches (mm).

k = effective length factor.

l_c = vertical distance between supports, inches (mm).

M_{cr} = *cracking moment* $5\sqrt{f'_c}Ig/y_t$ (For **SI:** $0.42\sqrt{f'_c}Ig/y_t$) *for regular concrete.*

M_n = *nominal moment strength at section, inch-pound (N·m).*

M_u = *factored moment at section, inch-pound (N·m). See Section 1914.8.3.*

P_{nw} = nominal axial load strength of wall designed by Section 1914.4.

P_u = *factored axial load at midheight of wall, including tributary wall weight.*

ρ = *ratio of nonprestressed tension reinforcement.*

ρ_b = *reinforcement ratio producing balanced strain conditions. See Formula (8-1).*

ϕ = strength-reduction factor. See Section 1909.3.

1914.1 Scope.

1914.1.1 Provisions of Section 1914 shall apply for design of walls subjected to axial load, with or without flexure.

1914.1.2 Cantilever retaining walls are designed according to flexural design provisions of Section 1910 with minimum horizontal reinforcement according to Section 1914.3.3.

1914.2 General.

1914.2.1 Walls shall be designed for eccentric loads and any lateral or other loads to which they are subjected.

1914.2.2 Walls subject to axial loads shall be designed in accordance with Sections 1914.2, 1914.3 and either Section 1914.4 or 1914.5.

1914.2.3 Design for shear shall be in accordance with Section 1911.10.

1914.2.4 Unless demonstrated by a detailed analysis, horizontal length of wall to be considered as effective for each concentrated load shall not exceed center-to-center distance between loads, or width of bearing plus four times the wall thickness.

1914.2.5 Compression members built integrally with walls shall conform to Section 1910.8.2.

1914.2.6 Walls shall be anchored to intersecting elements such as floors or roofs or to columns, pilasters, buttresses, and intersecting walls and footings.

1914.2.7 Quantity of reinforcement and limits of thickness required by Sections 1914.3 and 1914.5 shall be permitted to be waived where structural analysis shows adequate strength and stability.

1914.2.8 Transfer of force to footing at base of wall shall be in accordance with Section 1915.8.

1914.3 Minimum Reinforcement.

1914.3.1 Minimum vertical and horizontal reinforcement shall be in accordance with Sections 1914.3.2 and 1914.3.3 unless a greater amount is required for shear by Sections 1911.10.8 and 1911.10.9.

1914.3.2 Minimum ratio of vertical reinforcement area to gross concrete area shall be:

1. 0.0012 for deformed bars not larger than No. 5 with a specified yield strength not less than 60,000 psi (413.7 MPa), or

2. 0.0015 for other deformed bars, or

3. 0.0012 for welded wire fabric (plain or deformed) not larger than W31 or D31.

1914.3.3 Minimum ratio of horizontal reinforcement area to gross concrete area shall be:

1. 0.0020 for deformed bars not larger than No. 5 with a specified yield strength not less than 60,000 psi (413.7 MPa), or

2. 0.0025 for other deformed bars, or

3. 0.0020 for welded wire fabric (plain or deformed) not larger than W31 or D31.

1914.3.4 Walls more than 10 inches (254 mm) thick, except basement walls, shall have reinforcement for each direction placed in two layers parallel with faces of wall in accordance with the following:

1. One layer consisting of not less than one half and not more than two thirds of total reinforcement required for each direction shall be placed not less than 2 inches (51 mm) or more than one third the thickness of wall from exterior surface.

2. The other layer, consisting of the balance of required reinforcement in that direction, shall be placed not less than $^3/_4$ inch (19 mm) or more than one third the thickness of wall from interior surface.

1914.3.5 Vertical and horizontal reinforcement shall not be spaced farther apart than three times the wall thickness, nor 18 inches (457 mm). *Unless otherwise required by the engineer, the upper- and lowermost horizontal reinforcement shall be placed within one half of the specified spacing at the top and bottom of the wall.*

1914.3.6 Vertical reinforcement need not be enclosed by lateral ties if vertical reinforcement area is not greater than 0.01 times gross concrete area, or where vertical reinforcement is not required as compression reinforcement.

1914.3.7 In addition to the minimum reinforcement required by Section 1914.3.1, not less than two No. 5 bars shall be provided around all window and door openings. Such bars shall be extended to develop the bar beyond the corners of the openings but not less than 24 inches (610 mm).

1914.3.8 The minimum requirements for horizontal and vertical steel of Sections 1914.3.2 and 1914.3.3 may be interchanged for precast panels which are not restrained along vertical edges to inhibit temperature expansion or contraction.

1914.4 Walls Designed as Compression Members. Except as provided in Section 1914.5, walls subject to axial load or combined flexure and axial load shall be designed as compression members in accordance with provisions of Sections 1910.2, 1910.3, 1910.10, 1910.11, 1910.12, 1910.13, 1910.14, 1910.17, 1914.2 and 1914.3.

1914.5 Empirical Design Method.

1914.5.1 Walls of solid rectangular cross section shall be permitted to be designed by the empirical provisions of Section 1914.5 if resultant of all factored loads is located within the middle third of the overall thickness of wall and all limits of Sections 1914.2, 1914.3 and 1914.5 are satisfied.

1914.5.2 Design axial load strength ϕP_{nw} of a wall satisfying limitations of Section 1914.5.1 shall be computed by Formula (14-1) unless designed in accordance with Section 1914.4.

$$\phi P_{nw} = 0.55\,\phi f'_c A_g \left[1 - \left(\frac{kl_c}{32h}\right)^2\right] \qquad (14\text{-}1)$$

where $\phi = 0.70$ and effective length factor k shall be:

For walls braced top and bottom against lateral translation and

1. Restrained against rotation at one or both ends (top and/or bottom) 0.8

2. Unrestrained against rotation at both ends 1.0

For walls not braced against lateral translation 2.0

1914.5.3 Minimum thickness of walls designed by empirical design method.

1914.5.3.1 Thickness of bearing walls shall not be less than $^1/_{25}$ the supported height or length, whichever is shorter, or not less than 4 inches (102 mm).

1914.5.3.2 Thickness of exterior basement walls and foundation walls shall not be less than $7^1/_2$ inches (191 mm).

1914.6 Nonbearing Walls.

1914.6.1 Thickness of nonbearing walls shall not be less than 4 inches (102 mm), or not less than $^1/_{30}$ the least distance between members that provide lateral support.

1914.7 Walls as Grade Beams.

1914.7.1 Walls designed as grade beams shall have top and bottom reinforcement as required for moment in accordance with provisions of Sections 1910.2 through 1910.7. Design for shear shall be in accordance with provisions of Section 1911.

1914.7.2 Portions of grade beam walls exposed above grade shall also meet requirements of Section 1914.3.

1914.8 Alternate Design Slender Walls.

1914.8.1 *When flexural tension controls design of walls, the requirements of Section 1910.10 may be satisfied by complying with the limitations and procedures set forth in this section.*

1914.8.2 *The following limitations apply when this section is employed.*

1. Vertical service load stress at the location of maximum moment does not exceed 0.04 f'_c.

2. The reinforcement ratio ρ does not exceed 0.6 ρ_b.

3. Sufficient reinforcement is provided so that the nominal moment capacity times the ϕ factor is greater than M_{cr}.

4. Distribution of concentrated load does not exceed the width of bearing plus a width increasing at a slope of 2 vertical to 1 horizontal down to the design flexural section.

1914.8.3 *The required factored moment, M_u at the midheight cross section for combined axial and lateral factored loads, including the $P\,\Delta$ moments, shall be as set forth in Formula (14-2).*

$$M_u \le \phi M_n \qquad (14\text{-}2)$$

Unless a more comprehensive analysis is used, the $P\,\Delta$ moment shall be calculated using the maximum potential deflection, Δ_n, as defined in Section 1914.8.4.

1914.8.4 *The midheight deflection Δ_s, under service lateral and vertical loads (without load factors), shall be limited by the relation*

$$\Delta_s = \frac{l_c}{150} \qquad (14\text{-}3)$$

Unless a more comprehensive analysis is used, the midheight deflection shall be computed with the following formulas:

$$\Delta_s = \Delta_{cr} + \left(\frac{M_s - M_{cr}}{M_n - M_{cr}}\right)(\Delta_n - \Delta_{cr});\ \text{for } M_s > M_{cr} \qquad (14\text{-}4)$$

$$\Delta_s = \frac{5M_s l_c^{\,2}}{48E_c I_g};\ \text{for } M_s < M_{cr} \qquad (14\text{-}5)$$

WHERE:

$A_{se} = \dfrac{P_u + A_s f_y}{f_y}$

$I_{cr} = nA_{se}(d - c)^2 + \dfrac{bc^3}{3}$

M_s = *the maximum moment in the wall resulting from the application of the unfactored load combinations.*

$\Delta_{cr} = \dfrac{5M_{cr} l_c^{\,2}}{48E_c I_g}$

$\Delta_n = \dfrac{5M_n l_c^{\,2}}{48E_c I_{cr}}$

SECTION 1915 — FOOTINGS

1915.0 Notations.

A_g = gross area of section, square inches (mm^2).
d_p = diameter of pile at footing base.
β = ratio of long side to short side of footing.

1915.1 Scope.

1915.1.1 Provisions of this section shall apply for design of isolated footings and, where applicable, to combined footings and mats.

1915.1.2 Additional requirements for design of combined footings and mats are given in Section 1915.10.

1915.2 Loads and Reactions.

1915.2.1 Footings shall be proportioned to resist the factored loads and induced reactions, in accordance with the appropriate design requirements of this code and as provided in this section.

1915.2.2 Base area of footing or number and arrangement of piles.

1915.2.2.1 Base area of footing or number and arrangement of piles shall be determined from the external forces and moments (transmitted by footing to soil or piles) and permissible soil pressure or permissible pile capacity selected through principles of soil mechanics. External forces and moments are those resulting from unfactored loads (D, L, W and E) specified in Chapter 16. CA

1915.2.2.2 [For BSC, HCD 1 & HCD 2] Base area of footing or number and arrangement of piles shall be determined from the ex- CA ternal forces and moments (transmitted by footing to soil or piles) and permissible soil pressure or permissible pile capacity selected through principles of soil mechanics. *External forces and moments are those resulting from* ∗ ∗ ∗ *the load combinations of Section 1612.3.*

1915.2.3 For footings on piles, computations for moments and shears may be based on the assumption that the reaction from any pile is concentrated at pile center.

1915.2.4 *External forces and moments applied to footings shall be transferred to supporting soil without exceeding permissible soil pressures.*

1915.3 Footings Supporting Circular or Regular Polygon-shaped Columns or Pedestals. For location of critical sections for moment, shear and development of reinforcement in footings, it shall be permitted to treat circular or regular polygon-shaped concrete columns or pedestals as square members with the same area.

1915.4 Moment in Footings.

1915.4.1 External moment on any section of a footing shall be determined by passing a vertical plane through the footing and computing the moment of the forces acting over entire area of footing on one side of that vertical plane.

1915.4.2 Maximum factored moment for an isolated footing shall be computed as prescribed in Section 1915.4.1 at critical sections located as follows:

1. At face of column, pedestal or wall, for footings supporting a concrete column, pedestal or wall.
2. Halfway between middle and edge of wall, for footings supporting a masonry wall.
3. Halfway between face of column and edge of steel base, for footings supporting a column with steel base plates.

1915.4.3 In one-way footings, and two-way square footings, reinforcement shall be distributed uniformly across entire width of footing.

1915.4.4 In two-way rectangular footings, reinforcement shall be distributed as follows:

1915.4.4.1 Reinforcement in long direction shall be distributed uniformly across entire width of footing.

1915.4.4.2 For reinforcement in short direction, a portion of the total reinforcement given by Formula (15-1) shall be distributed uniformly over a band width (centered on center line of column or pedestal) equal to the length of short side of footing. Remainder of reinforcement required in short direction shall be distributed uniformly outside center band width of footing.

$$\frac{\text{Reinforcement in band width}}{\text{Total reinforcement in short direction}} = \frac{2}{(\beta + 1)} \qquad (15\text{-}1)$$

1915.5 Shear in Footings.

1915.5.1 Shear strength in footings shall be in accordance with Section 1911.12.

1915.5.2 Location of critical section for shear in accordance with Section 1911 shall be measured from face of column, pedestal or wall, for footings supporting a column, pedestal or wall. For footings supporting a column or pedestal with steel base plates, the critical section shall be measured from location defined in Section 1915.4.2, Item 3.

1915.5.3 Computation of shear on any section through a footing supported on piles shall be in accordance with the following:

1915.5.3.1 Entire reaction from any pile whose center is located $d_p/2$ or more outside the section shall be considered as producing shear on that section.

1915.5.3.2 Reaction from any pile whose center is located $d_p/2$ or more inside the section shall be considered as producing no shear in that section.

1915.5.3.3 For intermediate positions of pile center, the portion of the pile reaction to be considered as producing shear on the section shall be based on straight-line interpolation between full value at $d_p/2$ outside the section and zero value at $d_p/2$ inside the section.

1915.6 Development of Reinforcement in Footings.

1915.6.1 Development of reinforcement in footings shall be in accordance with Section 1912.

1915.6.2 Calculated tension or compression in reinforcement at each section shall be developed on each side of that section by embedment length, hooks (tension only), mechanical device or combinations thereof.

1915.6.3 Critical sections for development of reinforcement shall be assumed at the same locations as defined in Section 1915.4.2 for maximum factored moment, and at all other vertical planes where changes of section or reinforcement occur. See also Section 1912.10.6.

1915.7 Minimum Footing Depth. Depth of footing above bottom reinforcement shall not be less than 6 inches (152 mm) for footings on soil, or not less than 12 inches (305 mm) for footings on piles.

1915.8 Transfer of Force at Base of Column, Wall or Reinforced Pedestal.

1915.8.1 Forces and moments at base of column, wall, or pedestal shall be transferred to supporting pedestal or footing by bearing on concrete and by reinforcement, dowels and mechanical connectors.

1915.8.1.1 Bearing on concrete at contact surface between supported and supporting member shall not exceed concrete bearing strength for either surface as given by Section 1910.17.

1915.8.1.2 Reinforcement, dowels or mechanical connectors between supported and supporting members shall be adequate to transfer:

1. All compressive force that exceeds concrete bearing strength of either member.
2. Any computed tensile force across interface.

In addition, reinforcement, dowels or mechanical connectors shall satisfy Section 1915.8.2 or 1915.8.3.

1915.8.1.3 If calculated moments are transferred to supporting pedestal or footing, reinforcement, dowels or mechanical connectors shall be adequate to satisfy Section 1912.17.

1915.8.1.4 Lateral forces shall be transferred to supporting pedestal or footing in accordance with shear-friction provisions of Section 1911.7 or by other appropriate means.

1915.8.2 In cast-in-place construction, reinforcement required to satisfy Section 1915.8.1 shall be provided either by extending longitudinal bars into supporting pedestal or footing, or by dowels.

1915.8.2.1 For cast-in-place columns and pedestals, area of reinforcement across interface shall not be less than 0.005 times gross area of supported member.

1915.8.2.2 For cast-in-place walls, area of reinforcement across interface shall not be less than minimum vertical reinforcement given in Section 1914.3.2.

1915.8.2.3 At footings, No. 14 and No. 18 longitudinal bars, in compression only, may be lap spliced with dowels to provide reinforcement required to satisfy Section 1915.8.1. Dowels shall not be larger than No. 11 bar and shall extend into supported member a distance not less than the development length of No. 14 or No. 18 bars or the splice length of the dowels, whichever is greater, and into the footing a distance not less than the development length of the dowels.

1915.8.2.4 If a pinned or rocker connection is provided in cast-in-place construction, connection shall conform to Sections 1915.8.1 and 1915.8.3.

1915.8.3 In precast construction, reinforcement required to satisfy Section 1915.8.1 may be provided by anchor bolts or suitable mechanical connectors.

1915.8.3.1 Connection between precast columns or pedestals and supporting members shall meet the requirements of Section 1916.5.1.3, Item 1.

1915.8.3.2 Connection between precast walls and supporting members shall meet the requirements of Section 1916.5.1.3, Items 2 and 3.

> ***EXCEPTION:*** *In tilt-up construction, this connection may be to an adjacent floor slab. In no case shall the connection provided be less than that required by Section 1611.*

1915.8.3.3 Anchor bolts and mechanical connectors shall be designed to reach their design strength prior to anchorage failure or failure of surrounding concrete.

1915.9 Sloped or Stepped Footings.

1915.9.1 In sloped or stepped footings, angle of slope or depth and location of steps shall be such that design requirements are satisfied at every section.

1915.9.2 Sloped or stepped footings designed as a unit shall be constructed to assure action as a unit.

1915.10 Combined Footings and Mats.

1915.10.1 Footings supporting more than one column, pedestal or wall (combined footings or mats) shall be proportioned to resist the factored loads and induced reactions in accordance with appropriate design requirements of this code.

1915.10.2 The direct design method of Section 1913 shall not be used for design of combined footings and mats.

1915.10.3 Distribution of soil pressure under combined footings and mats shall be consistent with properties of the soil and the structure and with established principles of soil mechanics.

1915.11 Plain Concrete Pedestals and Footings. *See Section 1922.*

SECTION 1916 — PRECAST CONCRETE

1916.0 Notations.

A_g = gross area of column, inches squared (mm^2).

l = clear span, inches (mm).

1916.1 Scope.

1916.1.1 All provisions of this code, not specifically excluded and not in conflict with the provisions of Section 1916, shall apply to structures incorporating precast concrete structural members.

1916.2 General.

1916.2.1 Design of precast members and connections shall include loading and restraint conditions from initial fabrication to end use in the structure, including form removal, storage, transportation and erection.

1916.2.2 When precast members are incorporated into a structural system, the forces and deformations occurring in and adjacent to connections shall be included in the design.

1916.2.3 Tolerances for both precast members and interfacing members shall be specified. Design of precast members and connections shall include the effects of these tolerances.

1916.2.4 In addition to the requirements for drawings and specifications in Section *106.3.2*, the following shall be included in either the contract documents or shop drawings:

1. Details of reinforcement, inserts and lifting devices required to resist temporary loads from handling, storage, transportation and erection.

2. Required concrete strength at stated ages or stages of construction.

1916.3 Distribution of Forces among Members.

1916.3.1 Distribution of forces that are perpendicular to the plane of members shall be established by analysis or by test.

1916.3.2 Where the system behavior requires in-plane forces to be transferred between the members of a precast floor or wall system, the following shall apply:

1916.3.2.1 In-plane force paths shall be continuous through both connections and members.

1916.3.2.2 Where tension forces occur, a continuous path of steel or steel reinforcement shall be provided.

1916.4 Member Design.

1916.4.1 In one-way precast floor and roof slabs and in one-way precast, prestressed wall panels, all not wider than 12 feet (4 m), and where members are not mechanically connected to cause restraint in the transverse direction, the shrinkage and temperature reinforcement requirements of Section 1907.12 in the direction normal to the flexural reinforcement shall be permitted to be waived. This waiver shall not apply to members which require reinforcement to resist transverse flexural stresses.

1916.4.2 For precast, nonprestressed walls the reinforcement shall be designed in accordance with the provisions of Section 1910 or 1914 except that the area of horizontal and vertical reinforcement shall each be not less than 0.001 times the gross cross-sectional area of the wall panel. Spacing of reinforcement shall not exceed five times the wall thickness or 30 inches (762 mm) for interior walls or 18 inches (457 mm) for exterior walls.

1916.5 Structural Integrity.

1916.5.1 Except where the provisions of Section 1916.5.2 govern, the following minimum provisions for structural integrity shall apply to all precast concrete structures:

1916.5.1.1 Longitudinal and transverse ties required by Section 1907.13.3 shall connect members to a lateral-load-resisting system.

1916.5.1.2 Where precast elements form floor or roof diaphragms, the connections between diaphragm and those members being laterally supported shall have a nominal tensile strength capable of resisting not less than 300 pounds per linear foot (630 N/mm).

longitudinal axis of the beam to the column side. See Section 1921.5.3.1.

A_{sh} = total cross-sectional area of transverse reinforcement (including crossties) within spacing, s, and perpendicular to dimension, h_c.

b = effective compressive flange width of a structural member, inches (mm).

b_w = web width, or diameter of circular section, inches (mm).

d = effective depth of section.

d_b = bar diameter.

E = load effects of earthquake, or related internal moments and forces.

f'_c = specified compressive strength of concrete, psi (MPa).

f_y = specified yield strength of reinforcement, psi (MPa).

f_{yh} = specified yield strength of transverse reinforcement, psi (MPa).

h = overall dimension of member in the direction of action considered.

h_c = cross-sectional dimension of a column core or shear wall boundary zone measured center-to-center of confining reinforcement.

h_w = height of entire wall (diaphragm) or of the segment of wall (diaphragm) considered.

l_d = development length for a straight bar.

l_{dh} = development length for a bar with a standard hook as defined in Formula (21-5).

l_o = minimum length, measured from joint face along axis of structural member, over which transverse reinforcement must be provided, inches (mm).

l_u = unsupported length of compression member (see Section 1910.11.3.1).

l_w = length of entire wall (diaphragm) or of segment of wall (diaphragm) considered in direction of shear force.

M_{pr} = probable flexural strength of members, with or without axial load, determined using the properties of the member at the joint faces assuming a tensile strength in the longitudinal bars of at least 1.25 f_y and a strength-reduction factor ϕ of 1.0.

M_s = portion of slab moment balanced by support moment.

s = spacing of transverse reinforcement measured along the longitudinal axis of the structural member, inches (mm).

$S_{e\ CONNECTION}$
= *moment, shear or axial force at connection cross section other than the nonlinear action location corresponding to probable strength at the nonlinear action location, taking gravity load effects into consideration, per Section 1921.2.7.3.*

$S_{n\ CONNECTION}$
= *nominal strength of connection cross section in flexural, shear or axial action, per Section 1921.2.7.3.*

s_o = maximum spacing of transverse reinforcement, inches (mm).

V_c = nominal shear strength provided by concrete.

V_e = design shear force determined from Section 1921.3.4.1 or 1921.4.5.1.

V_n = nominal shear strength.

V_u = factored shear force at section.

α_c = coefficient defining the relative contribution of concrete strength to wall strength.

ρ = ratio of nonprestressed tension reinforcement
= A_s/bd.

ρ_g = ratio of total reinforcement area to cross-sectional area of column.

ρ_n = ratio of distributed shear reinforcement on a plane perpendicular to plane of A_{cv}.

ρ_s = ratio of volume of spiral reinforcement to the core volume confined by the spiral reinforcement (measured out-to-out).

ρ_v = A_{sv}/A_{cv}; where A_{sv} is the projection on A_{cv} of area of distributed shear reinforcement crossing the plane of A_{cv}.

ϕ = strength-reduction factor.

Δ_M = 0.7 $R\Delta_s$.

Δ_s = *Design Level Response Displacement, which is the total drift or total story drift that occurs when the structure is subjected to the design seismic forces.*

ψ = *Dynamic Amplification Factor from Sections 1921.2.7.3 and 1921.2.7.4.*

1921.1 Definitions. For the purposes of this section, certain terms are defined as follows:

BASE OF STRUCTURE is the level at which earthquake motions are assumed to be imparted to a building. This level does not necessarily coincide with the ground level.

BOUNDARY ELEMENTS ***(or ZONES)*** are portions along wall and diaphragm edges strengthened by longitudinal and transeverse reinforcement. Boundary elements do not necessarily require an increase in the thickness of the wall or diaphragm. Edges of openings within walls and diaphragms shall be provided with boundary elements if required by Sections 1921.6.6.1 and 1921.6.7.1.

COLLECTOR ELEMENTS are elements that serve to transmit the inertial forces with the diaphragms to members of the lateral-force-resisting systems.

CONFINED CORE is the area within the core defined by h_c.

CONNECTION is an element that joins two precast members or a precast member and a cast-in-place member.

COUPLING BEAMS are a horizontal element in plane with and connecting two shear walls.

CROSSTIE is a continuous reinforcing bar having a seismic hook at one end and a hook of not less than 90 degrees with at least six diameters at the other end. The hooks shall engage peripheral longitudinal bars. The 90-degree hooks of two successive crossties engaging the same longitudinal bar shall be alternated end for end.

DESIGN LOAD COMBINATIONS are combinations of factored loads and forces specified in Sections *1612.2.1 and* 1909.2.

DEVELOPMENT LENGTH FOR A BAR WITH A STANDARD HOOK is the shortest distance between the critical section (where the strength of the bar is to be developed) and a tangent to the outer edge of the 90-degree hook.

DRY CONNECTION *is a connection used between precast members, which does not qualify as a wet connection.*

FACTORED LOADS AND FORCES are the specified loads and forces modified by the factors in Sections *1612.2.1 and 1909.2.*

HOOP is a closed tie or continuously wound tie. A closed tie can be made up of several reinforcing elements, each having seismic hooks at both ends. A continuously wound tie shall have a seismic hook at both ends.

JOINT *is the geometric volume common to intersecting members.*

LATERAL-FORCE-RESISTING SYSTEM is that portion of the structure composed of members proportioned to resist forces related to earthquake effects.

LIGHTWEIGHT-AGGREGATE CONCRETE is all lightweight or sanded lightweight aggregate concrete made with lightweight aggregates conforming to Section 1903.3.

NONLINEAR ACTION LOCATION *is the center of the region of yielding in flexure, shear or axial action.*

NONLINEAR ACTION REGION *is the member length over which nonlinear action takes place. It shall be taken as extending a distance of no less than* h/2 *on either side of the nonlinear action location.*

SEISMIC HOOK is a hook on a stirrup, hoop or crosstie having a bend not less than 135 degrees with a six-bar-diameter [but not less than 3 inches (76 mm)], extension that engages the longitudinal reinforcement and projects into the interior of the stirrup or hoop.

SHELL CONCRETE is concrete outside the transverse reinforcement confining the concrete.

SPECIFIED LATERAL FORCES are lateral forces corresponding to the appropriate distribution of the design base shear force prescribed by the governing code for earthquake-resistant design.

STRONG CONNECTION *is a connection that remains elastic, while the designated nonlinear action regions undergo inelastic response under the Design Basis Ground Motion.*

STRUCTURAL DIAPHRAGMS are structural members, such as floor and roof slabs, which transmit inertial forces to lateral-force-resisting members.

STRUCTURAL TRUSSES are assemblages of reinforced concrete members subjected primarily to axial forces.

STRUT is an element of a structural diaphragm used to provide continuity around an opening in the diaphragm.

TIE ELEMENTS are elements which serve to transmit inertia forces and prevent separation of such building components as footings and walls.

WALL PIER *is a wall segment with a horizontal length-to-thickness ratio between 2.5 and 6, and whose clear height is at least two times its horizontal length.*

WET CONNECTION *uses any of the splicing methods, per Section 1921.2.6.1 or 1921.3.2.3, to connect precast members and uses cast-in-place concrete or grout to fill the splicing closure.*

1921.2 General Requirements.

1921.2.1 Scope.

1921.2.1.1 *Section 1921* contains special requirements for design and construction of reinforced concrete members of a structure for which the design forces, related to earthquake motions, have been determined on the basis of energy dissipation in the nonlinear range of response.

1921.2.1.2 The provisions of Sections 1901 through 1918 shall apply except as modified by the provisions of Section 1921.

1921.2.1.3 *In Seismic Zones 0 and 1, the provisions of Section 1921 shall not apply.*

In Seismic Zone 2, reinforced concrete frames resisting forces induced by earthquake motions shall be intermediate moment-resisting frames proportioned to satisfy only Section 1921.8 in addition to the requirements of Sections 1901 through 1918. In Seismic Zone 2, frame members which are not designated to be part of the lateral-force-resisting system shall conform to Section 1921.7.

1921.2.1.4 *In Seismic Zones 3 and 4,* all reinforced concrete structural members that are part of the lateral-force-resisting system shall satisfy the requirements of Sections 1921.2 through 1921.7, in addition to the requirements of Sections 1901 through 1917.

1921.2.1.5 A reinforced concrete structural system not satisfying the requirements of this section may be used if it is demonstrated by experimental evidence and analysis that the proposed system will have strength and toughness equal to or exceeding those provided by a comparable monolithic reinforced concrete structure satisfying this section.

1921.2.1.6 *Precast lateral-force-resisting systems shall satisfy either of the following criteria:*

1. Emulate the behavior of monolithic reinforced concrete construction and satisfy Section 1921.2.2.5, or

2. Rely on the unique properties of a structural system composed of interconnected precast elements and conform to Section 1629.9.2.

1921.2.1.7 *In structures having precast gravity systems, the lateral-force-resisting system shall be one of the systems listed in Table 16-N [for BSC, HCD 1 & HCD 2] Table 16-N.1 and shall be well distributed using one of the following methods:* CA

1. The lateral-force-resisting systems shall be spaced such that the span of the diaphragm or diaphragm segment between lateral-force-resisting systems shall be no more than three times the width of the diaphragm or diaphragm segment.

Where the lateral-force-resisting system consists of moment-resisting frames, at least $[(N_b/4) + 1]$ of the bays (rounded up to the nearest integer) along any frame line at any story shall be part of the lateral-force-resisting system, where N_b is the total number of bays along that line at that story. This requirement applies to only the lower two thirds of the stories of buildings three stories or taller.

2. All beam-to-column connections that are not part of the lateral-force-resisting system shall be designed in accordance with the following:

Connection design force. *The connection shall be designed to develop strength M. M is the moment developed at the connection when the frame is displaced by Δ_s assuming fixity at the connection and a beam flexural stiffness of no less than one-half of the gross section stiffness. M shall be sustained through a deformation of Δ_m [for OSHPD 2] Δ_M.* CA

Connection characteristics. *The connection shall be permitted to resist moment in one direction only, positive or negative. The connection at the opposite end of the member shall resist moment with same positive or negative sign. The connection shall be permitted to have zero flexural stiffness up to a frame displacement of Δ_s.*

In addition, complete calculations for the deformation compatibility of the gravity load carrying system shall be made in accordance with Section 1633.2.4 using cracked section stiffnesses in the lateral-force-resisting system and the diaphragm.

Where gravity columns are not provided with lateral support on all sides, a positive connection shall be provided along each unsupported direction parallel to a principal plan axis of the structure. The connection shall be designed for a horizontal force equal to 4 percent of the axial load strength (P_0) of the column.

sections, it shall be permitted to use the same compression-controlled strain limit as that for reinforcement with a design yield strength f_y of 60,000 psi (413.7 MPa).

B.1910.3.3 Sections are compression-controlled when the net tensile strain in the extreme tension steel is equal to or less than the compression-controlled strain limit at the time the concrete in compression reaches its assumed strain limit of 0.003. Sections are tension-controlled when the net tensile strain in the extreme tension steel is equal to or greater than 0.005 just as the concrete in compression reaches its assumed strain limit of 0.003. Sections with net tensile strain in the extreme tension steel between the compression-controlled strain limit and 0.005 constitute a transition region between compression-controlled and tension-controlled sections.

B.1918.1.3 The following provisions of this code shall not apply to prestressed concrete, except as specifically noted: Sections 1907.6.5, 1908.10.2, 1908.10.3, 1908.10.4, 1908.11, 1910.5, 1910.6, 1910.9.1 and 1910.9.2; Section 1913; and Sections 1914.3, 1914.5 and 1914.6.

B.1918.8 Limits for Reinforcement of Flexural Members.

B.1918.8.1 Prestressed concrete sections shall be classified as tension-controlled and compression-controlled sections in accordance with Section B.1910.3.3. The appropriate ϕ-factors from Section B.1909.3.2 shall apply.

B.1918.8.2 Total amount of prestressed and nonprestressed reinforcement shall be adequate to develop a factored load at least 1.2 times the cracking load computed on the basis of the modulus of rupture specified in Section 1909.5.2.3, except for flexural members with shear and flexural strength at least twice that required by Section 1909.2.

B.1918.8.3 Part or all of the bonded reinforcement consisting of bars or tendons shall be provided as close as practicable to the extreme tension fiber in all prestressed flexural members, except that in members prestressed with unbonded tendons, the minimum bonded reinforcement consisting of bars or tendons shall be as required by Section 1918.9.

B.1918.10.4 Redistribution of negative moments in continuous prestressed flexural members.

B.1918.10.4.1 Where bonded reinforcement is provided at supports in accordance with Section 1918.9.2, it shall be permitted to increase or decrease negative moments calculated by elastic theory for any assumed loading, in accordance with Section B.1908.4.

B.1918.10.4.2 The modified negative moments shall be used for calculating moments at sections within spans for the same loading arrangement.

Division VIII—ALTERNATIVE LOAD-FACTOR COMBINATION AND STRENGTH REDUCTION FACTORS

NOTE: This is a new division.

SECTION 1928 — ALTERNATIVE LOAD-FACTOR COMBINATION AND STRENGTH REDUCTION FACTORS

1928.1 General. It shall be permitted to proportion concrete structural elements using the alternate load-factor combinations in Section 1928.1.2 in conjunction with the alternate strength reduction factors in Section 1928.1.1 if the structural framing includes primary members of other materials proportioned to satisfy the alternate load-factor combinations in Section 1928.1.2. Loads shall be determined in accordance with Chapter 16 of this code.

1928.1.1 Alternate strength reduction factors.

1928.1.1.1 Flexure, without axial load 0.80

1928.1.1.2 Axial tension and axial tension with flexure . . 0.80

1928.1.1.3 Axial compression and axial compression with flexure

1. Members with spiral reinforcement conforming to Section 1910.9.3 . . . . 0.70
2. Other reinforced members 0.65

except that for low values of axial compression, it shall be permitted to increase ϕ toward the value for flexure, 0.80, using the linear interpolation provided in either Section 1909.3.2.2 or B.1909.3.2.2.

3. In Seismic Zones 3 and 4, members resisting earthquake forces without transverse reinforcement conforming to 21.4.4 . . . . 0.50

1928.1.1.4 Shear and torsion 0.75

except that in Seismic Zones 3 and 4:

1. Shear in members resisting earthquake forces if the nominal shear strength of the member is less than the nominal shear corresponding to the development of the nominal flexural strength of the member 0.55
2. Shear in joints of building structures 0.80

1928.1.1.5 Bearing 0.65

1928.1.1.6 Plain concrete 0.55

1928.1.2 Alternate load-factor combinations.

1928.1.2.1 Symbols and notations.

D = dead load consisting of: (1) weight of the member, (2) weight of all materials of construction incorporated into the building to be permanently supported by the member, including built-in partitions, and (3) weight of permanent equipment.

E = earthquake load.

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

H = loads due to the weight and lateral pressure of soil and water in soil.

L = live loads due to intended use and occupancy, including loads due to movable objects and movable partitions and loads temporarily supported by the structure during maintenance. L includes any permissible reduction. If resistance to impact loads is taken into account in design, such effects shall be included with the live load L.

L_r = roof live loads.

P = loads, forces and effects due to ponding.

R = rain loads, except ponding.

S = snow loads.

T = self-straining forces and effects arising from contraction or expansion resulting from temperature changes, shrinkage, moisture changes, creep in component materials, movement due to differential settlement or combinations thereof.

W = wind load.

1928.1.2.2 Combining loads using strength design.

1928.1.2.3 Basic combinations.

1928.1.2.3.1 When permitted by Section 1928.1, structures, components and foundations shall be designed so that their design strength exceeds the effects of the factored loads in the following combinations: CA ||

1. $1.4D$
2. $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W)$
4. $1.2D + 1.3W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.5E + (0.5L \text{ or } 0.2S)$
6. $0.9D - (1.3W \text{ or } 1.5E)$

EXCEPTIONS: 1. The load factor on L in combinations 3, 4 and 5 shall equal 1.0 for garages, areas occupied and places of public assembly, and all areas where the live load is greater than 100 lb./ft.^{2} (pounds-force per square foot) (4.79 kPa).

2. Each relevant strength limit state shall be considered. The most unfavorable effect may occur when one or more of the contributing loads are not acting.

1928.1.2.3.2 [For BSC, HCD 1 & HCD 2] When permitted by Section 1928.1, structures, components and foundations shall be designed so that their design strength exceeds the effects of the factored loads in the following combinations: CA ||

1. 1.4D
2. $1.2D + 1.6L + 0.5(L_r \text{ or S or R})$
3. $1.2D + 1.6(L_r \text{ or S or R}) + (0.5L \text{ or } 0.8W)$
4. $1.2D + 1.3W + 0.5L + 0.5(L_r \text{ or S or R})$
5. 1.2D ± *1.0E* + (0.5L or 0.2S) CA
6. 0.9D ± (1.3W or *1.0E*) CA

EXCEPTIONS: 1. The load factor on L in combinations 3, 4 and 5 shall equal 1.0 for garages, areas occupied and places of public assembly, and all areas where the live load is greater than 100 lb/ft^2 (pounds-force per square foot) (4.79 kPa).

2. Each relevant strength limit state shall be considered. The most unfavorable effect may occur when one or more of the contributing loads are not acting.

1928.1.2.4 Other combinations. The structural effects of F, H, P or T shall be considered in design as the following factored loads: $1.3F$, $1.6H$, $1.2P$ and $1.2T$.

LOAD, DEAD, is the dead weight supported by a member, as defined by *Section 1602A* (without load factors).

LOAD, FACTORED, is the load, multiplied by appropriate load factors, used to proportion members by the strength design method of this code. See Sections 1908A.1.1 and 1909A.2.

LOAD, LIVE, *is the live load specified by Section 1602A* (without load factors).

LOAD, SERVICE, *is the live and dead loads* (without load factors).

MODULUS OF ELASTICITY is the ratio of normal stress to corresponding strain for tensile or compressive stresses below proportional limit of material. See Section 1908A.5.

NET TENSILE STRAIN is the tensile strain at nominal strength exclusive of strains due to effective prestress, creep, shrinkage and temperature.

PEDESTAL is an upright compression member with a ratio of unsupported height to average least lateral dimension of 3 or less.

PLAIN CONCRETE is structural concrete with no reinforcement or with less reinforcement than the minimum amount specified for reinforced concrete.

PLAIN REINFORCEMENT is reinforcement that does not conform to definition of deformed reinforcement.

POSTTENSIONING is a method of prestressing in which tendons are tensioned after concrete has hardened.

PRECAST CONCRETE is a structural concrete element cast in other than its final position in the structure.

PRESTRESSED CONCRETE is structural concrete in which internal stresses have been introduced to reduce potential tensile stresses in concrete resulting from loads.

PRETENSIONING is a method of prestressing in which tendons are tensioned before concrete is placed.

REINFORCED CONCRETE is structural concrete reinforced with no less than the minimum amounts of prestressing tendons or nonprestressed reinforcement specified in this code.

REINFORCEMENT is material that conforms to Section 1903A.5.1, excluding prestressing tendons unless specifically included.

RESHORES are shores placed snugly under a concrete slab or other structural member after the original forms and shores have been removed from a larger area, thus requiring the new slab or structural member to deflect and support its own weight and existing construction loads applied prior to the installation of the reshores.

SHORES are vertical or inclined support members designed to carry the weight of the formwork, concrete and construction loads above.

SPAN LENGTH. See Section 1908A.7.

SPIRAL REINFORCEMENT is continuously wound reinforcement in the form of a cylindrical helix.

SPLITTING TENSILE STRENGTH (f_{ct}) is the tensile strength of concrete. See Section 1905A.1.4.

STIRRUP is reinforcement used to resist shear and torsion stresses in a structural member; typically bars, wires, or welded wire fabric (smooth or deformed) bent into L, U or rectangular shapes and located perpendicular to or at an angle to longitudinal reinforcement. (The term "stirrups" is usually applied to lateral reinforcement in flexural members and the term "ties" to those in compression members.) See "tie."

STRENGTH, DESIGN, is the nominal strength multiplied by a strength-reduction factor ϕ. See Section 1909A.3.

STRENGTH, NOMINAL, is the strength of a member or cross section calculated in accordance with provisions and assumptions of the strength design method of this code before application of any strength-reduction factors. See Section 1909A.3.1.

STRENGTH, REQUIRED, is the strength of a member or cross section required to resist factored loads or related internal moments and forces in such combinations as are stipulated in this code. See Section 1909A.1.1.

STRESS is the intensity of force per unit area.

STRUCTURAL CONCRETE is all concrete used for structural purposes, including plain and reinforced concrete.

TENDON is a steel element such as wire, cable, bar, rod or strand, or a bundle of such elements, used to impart prestress to concrete.

TENSION-CONTROLLED SECTION is a cross section in which the net tensile strain in the extreme tension steel at nominal strength is greater than or equal to 0.005.

TIE is a loop of reinforcing bar or wire enclosing longitudinal reinforcement. A continuously wound bar or wire in the form of a circle, rectangle or other polygon shape without re-entrant corners is acceptable. See "stirrup."

TRANSFER is the act of transferring stress in prestressing tendons from jacks or pretensioning bed to concrete member.

WALL is a member, usually vertical, used to enclose or separate spaces.

WALL PIER is a wall segment with a horizontal length-to-thickness ratio between $2^1/_2$ and 6 and a clear height of at least two times its horizontal length. CACAC

WOBBLE FRICTION in prestressed concrete, is friction caused by unintended deviation of prestressing sheath or duct from its specified profile.

YIELD STRENGTH is the specified minimum yield strength or yield point of reinforcement in psi.

SECTION 1903A — SPECIFICATIONS FOR TESTS AND MATERIALS

1903A.0 Notation.

f_y = specified yield strength of nonprestressed reinforcement, psi (MPa).

1903A.1 Tests of Materials.

1903A.1.1 The enforcement agency may require the testing of any materials used in concrete construction to determine if materials are of quality specified. CA

1903A.1.2 Tests of materials and of concrete shall be made *by an approved agency and at no expense to the jurisdiction. Such tests shall be made in accordance with the standards listed in Section 1903A.*

1903A.1.3 A complete record of tests of materials and of concrete shall be available for inspection during progress of work and for two years after completion of the project, and shall be preserved by the inspecting engineer or architect for that purpose.

1903A.1.4 Material and test standards. *The standards listed in this chapter labeled a "UBC Standard" are also listed below in American Society for Testing and Materials (ASTM) recognized standards and are also part of this code. (See Sections 3503 and 3504.)* CACAC

1903A.2 Cement.

1. ASTM C 845, Expansive Hydraulic Cement
2. ASTM C 150, Portland Cement
3. ASTM C 595 or ASTM C 1157, Blended Hydraulic Cements

1903A.3 Aggregates.

1903A.3.1 Recognized standards.

1. ASTM C 33, Concrete Aggregates
2. ASTM C 330, Lightweight Aggregates for Structural Concrete
3. ASTM C 332, Lightweight Aggregates for Insulating Concrete
4. ASTM C 144, Aggregate for Masonry Mortar
5. Aggregates failing to meet the above specifications but which have been shown by special test or actual service to produce concrete of adequate strength and durability may be used where authorized by the *enforcement agency.*

1903A.3.2 ***Aggregate Size.***

1903A.3.2.1 [For DSA/SS] The nominal maximum size of coarse aggregate shall not be larger than:

1. One fifth the narrowest dimension between sides of forms, or
2. One third the depth of slabs, or
3. Three fourths the minimum clear spacing between individual reinforcing bars or wires, bundles of bars, or prestressing tendons or ducts.

These limitations may be waived if, in the judgment of the *structural engineer and the enforcement agency,* workability and methods of consolidation are such that concrete can be placed without honeycomb or voids *and the mixes are designed and tested in accordance with Method B or C of Section 1905A.*

Evidence that the aggregate used is not reactive in the presence of cement alkalies may be required by the enforcement agency. If new aggregate sources are to be used or if past experience indicates problems with existing aggregate sources, test the aggregate for potential reactivity according to ASTM C 289. If a result other than innocuous is obtained, test the cement-aggregate combination according to ASTM C 227 using the cement corresponding to that on which the selection of concrete proportions was based (see Section 1905A.2). If the results of this test indicate an expansion greater than 0.10 percent at six months, the aggregate shall be deemed to contain reactive substances in amounts deleterious to concrete, and shall be used with a cementitious material system suitable for preventing alkali-aggregate reaction as follows:

1. Low-alkali portland cement containing not more than 0.6 percent total alkali when calculated as sodium oxide, as determined by the method given in Methods of Chemical Analysis of Hydraulic Cement, ASTM C 114.

2. Blended hydraulic cement, Type 1S or 1P, conforming to UBC Standard 19-1, Part III, except that Type 1S cement shall not contain less than 40 percent slag constituent.

3. Replacement of not less than 15 percent by weight of the portland cement used by a mineral admixture conforming to ASTM C 618.

4. Replacement of not less than 40 percent by weight of the portland cement used by a ground granulated blast-furnace slag conforming to ASTM C 989.

1903A.3.2.2 [For OSHPD 1 & 4] The nominal maximum size of coarse aggregate shall not be larger than:

1. One fifth the narrowest dimension between sides of forms, or
2. One third the depth of slabs, or
3. Three fourths the minimum clear spacing between individual reinforcing bars or wires, bundles of bars, or prestressing tendons or ducts.

These limitations may be waived if, in the judgment of the *structural engineer and the enforcement agency,* workability and methods of consolidation are such that concrete can be placed without honeycomb or voids *and the mixes are designed and tested in accordance with Method B or C of Section 1905A.*

Evidence that the aggregate used is not reactive in the presence of cement alkalies may be required by the enforcement agency. If new aggregate sources are to be used or if past experience indicates problems with existing aggregate sources, the enforcement agency may require that the aggregate be tested in accordance with ASTM C 289 to determine potential reactivity in the presence of cement.

If the results of the test are other than innocuous selected concrete proportions (see Section 1905A.2), using the aggregate shall be tested in accordance with ASTM C 227. If the results of this test indicate an expansion greater than 0.10 percent at six months, the aggregate shall be used only when in combination with one of the following cementitious materials:

1. Low-alkali portland cement containing not more than 0.6 percent total alkali when calculated as sodium oxide, as determined by the method given in ASTM C 114.

2. Blended hydraulic cement, Type 1S or 1P, conforming to ASTM C 595, except that Type 1S cement shall not contain less than 40 percent slag constituent.

3. Replacement of not less than 15 percent by weight of the portland cement used by a mineral admixture conforming to ASTM C 618 for Class N or F materials (Class C is not permitted).

4. Replacement of not less than 40 percent by weight of the portland cement used by a ground granulated blast-furnace slag conforming to ASTM C 989.

1903A.4 Water.

1903A.4.1 Water used in mixing concrete shall be clean and free from injurious amounts of oils, acids, alkalis, salts, organic materials or other substances deleterious to concrete or reinforcement.

1903A.4.2 Mixing water for prestressed concrete or for concrete that will contain aluminum embedments, including that portion of mixing water contributed in the form of free moisture on aggregates, shall not contain deleterious amounts of chloride ions. See Section 1904A.4.1.

1903A.4.3 Nonpotable water shall not be used in concrete unless the following are satisfied:

1903A.4.3.1 Selection of concrete proportions shall be based on concrete mixes using water from the same source.

1903A.4.3.2 Mortar test cubes made with nonpotable mixing water shall have seven-day and 28-day strengths equal to at least 90 percent of strengths of similar specimens made with potable water. Strength test comparison shall be made on mortars, identical except for the mixing water, prepared and tested in accordance with ASTM C 109 (Compressive Strength of Hydraulic Cement Mortars).

1903A.5 Steel Reinforcement.

1903A.5.1 Reinforcement shall be deformed reinforcement, except that plain reinforcement may be used for spirals or tendons, and reinforcement consisting of structural steel, steel pipe or steel tubing may be used as specified in this chapter.

1903A.5.2 Welding of reinforcing bars shall conform to *approved nationally recognized standards.* Type and location of welded splices and other required welding of reinforcing bars shall be indicated on the design drawings or in the project specifications. ASTM reinforcing bar specifications, except for A 706, shall be supplemented to require a report of material properties necessary to conform to requirements in UBC Standard 19-1.

If mill test reports are not available, chemical analysis shall be made of bars representative of the bars to be welded. Bars with a carbon equivalent (C.E.) above 0.75 shall not be welded. Welding shall not be done on or within two bar diameters of any bent portion of a bar that has been bent cold. Welding of crossing bars shall not be permitted for assembly of reinforcement unless authorized by the structural engineer and approved by the enforcement agency per approved procedures.

1903A.5.3 Deformed reinforcements.

1903A.5.3.1 ASTM A 615, A 616, A 617, A 706, A 767 and A 775, Reinforcing Bars for Concrete.

1903A.5.3.2 Deformed reinforcing bars with a specified yield strength f_y exceeding 60,000 psi (413.7 MPa) may be used, provided f_y shall be the stress corresponding to a strain of 0.35 percent and the bars otherwise conform to approved national standards, see *ASTM A 615, A 616, A 617, A 706, A 767 and A 775.* See Section 1909A.4.

1903A.5.3.3 ASTM A 184, Fabricated Deformed Steel Bar Mats. For reinforced bars used in bar mats, see ASTM A 615, A 616, A 617, A 706, A 767 or A 775.

1903A.5.3.4 ASTM A 496, Steel Wire, Deformed, for Concrete Reinforcement.

For deformed wire for concrete reinforcement, see *ASTM A 496,* except that wire shall not be smaller than size D4, and for wire with a specified yield strength f_y exceeding 60,000 psi (413.7 MPa), f_y shall be the stress corresponding to a strain of 0.35 percent, if the yield strength specified in design exceeds 60,000 psi (413.7 MPa).

1903A.5.3.5 ASTM A 185, Steel Welded Wire, Fabric, Plain for Concrete Reinforcement.

For welded plain wire fabric for concrete reinforcement, see *ASTM 185*, except that for wire with a specified yield strength f_y exceeding 60,000 psi (413.7 MPa), f_y shall be the stress corresponding to a strain of 0.35 percent, if the yield strength specified in design exceeds 60,000 psi (413.7 MPa). Welded intersections shall not be spaced farther apart than 12 inches (305 mm) in direction of calculated stress, except for wire fabric used as stirrups in accordance with Section 1912A.14.

1903A.5.3.6 ASTM A 497, Welded Deformed Steel Wire Fabric for Concrete Reinforcement.

For welded deformed wire fabric for concrete reinforcement, see *ASTM A 497*, except that for wire with a specified yield strength f_y exceeding 60,000 psi (413.7 MPa), f_y shall be the stress corresponding to a strain of 0.35 percent, if the yield strength specified in design exceeds 60,000 psi (413.7 MPa). Welded intersections shall not be spaced farther apart than 16 inches (406 mm) in direction of calculated stress, except for wire fabric used as stirrups in accordance with Section 1912A.13.2.

1903A.5.3.7 Deformed reinforcing bars may be galvanized or epoxy coated. For zinc or epoxy-coated reinforcement, see ASTM A 615, A 616, A 617, A 706, A 767 and A 775 and ASTM A 934 *(Epoxy-Coated Steel Reinforcing Bars).*

1903A.5.3.8 Epoxy-coated wires and welded wire fabric shall comply with ASTM A 884 (*Standard Specification for Epoxy-Coated Steel Wire and Welded Wire Fabric for Reinforcement*). Epoxy-coated wires shall conform to Section 1903A.5.3.4 and epoxy-coated welded wire fabric shall conform to Section 1903A.5.3.5 or 1903A.5.3.6.

1903A.5.4 Plain reinforcement.

1903A.5.4.1 Plain bars for spiral reinforcement shall conform to approved national standards, see ASTM *A 615, A 616 and A 617.*

1903A.5.4.2 For plain wire for spiral reinforcement, see *ASTM A 82* except that for wire with a specified yield strength f_y exceeding 60,000 psi (413.7 MPa), f_y shall be the stress corresponding to a strain of 0.35 percent, if the yield strength specified in design exceeds 60,000 psi (413.7 MPa).

1903A.5.5 Prestressing tendons.

1903A.5.5.1 1. ASTM A 416, Uncoated Seven-wire Stress-relieved Steel Strand for Prestressed Concrete

2. ASTM A 421, Uncoated Stress-relieved Wire for Prestressed Concrete

3. ASTM A 722, Uncoated High-strength Steel Bar for Prestressing Concrete

1903A.5.5.2 Wire, strands and bars not specifically listed in *ASTM A 416, A 421 and A 722* may be used, provided they conform to minimum requirements of these specifications and do not have properties that make them less satisfactory than those listed.

1903A.5.6 Structural steel, steel pipe or tubing.

1903A.5.6.1 For structural steel used with reinforcing bars in composite compression members meeting requirements of Section 1910A.16.7 or 1910A.16.8, see *ASTM A 36, A 242, A 572 and A 588.*

1903A.5.6.2 For steel pipe or tubing for composite compression members composed of a steel-encased concrete core meeting requirements of Section 1910A.16.4, see *ASTM A 53, A 500 and A 501.*

1903A.5.7 *UBC Standard 19-1, Welding Reinforcing Steel, Metal Inserts and Connections in Reinforced Concrete Construction*

1903A.6 Admixtures.

1903A.6.1 Admixtures to be used in concrete shall be subject to prior approval by the *enforcement agency.*

1903A.6.2 An admixture shall be shown capable of maintaining essentially the same composition and performance throughout the work as the product used in establishing concrete proportions in accordance with Section 1905A.2.

1903A.6.3 Calcium chloride or admixtures containing chloride from other than impurities from admixture ingredients shall not be used in prestressed concrete, in concrete containing embedded aluminum, or in concrete cast against stay-in-place galvanized steel forms. See Sections 1904A.3.2 and 1904A.4.1.

1903A.6.4 ASTM C 260, Air-entraining Admixtures for Concrete

1903A.6.5 ASTM C 494 and C 1017, Chemical Admixtures for Concrete

1903A.6.6 *Fly ash or other pozzolan can be used as a partial substitute for ASTM 150 portland cement, as follows:*

1. Fly ash or other pozzolan shall conform to ASTM C 618 for Class N or Class F materials (Class C is not permitted), and

2. More than 15 percent by weight of fly ash or other pozzolans shall be permitted to be substituted for ASTM C 150 portland cement if the mix design is proportioned by Method B or C. See Section 1904A for durability requirements.

3. More than 40 percent by weight of ground-granulated blast-furnace slag conforming to ASTM C 989 shall be permitted to be substituted for ASTM C 150 portland cement if the mix design is proportioned by Method B or C. See Section 1904A for durability requirements.

1903A.6.7 *ASTM C 989,* Ground-iron Blast-furnace Slag for Use in Concrete and Mortars

1903A.6.8 Admixtures used in concrete containing ASTM C 845 expansive cements shall be compatible with the cement and produce no deleterious effects.

1903A.6.9 Silica fume used as an admixture shall conform to ASTM C 1240 (*Silica Fume for Use in Hydraulic Cement Concrete and Mortar*)*, and the concrete shall be proportioned by Method B or C.*

1903A.7 Storage of Materials.

1903A.7.1 Cementitious materials and aggregate shall be stored in such manner as to prevent deterioration or intrusion of foreign matter.

1903A.7.2 Any material that has deteriorated or has been contaminated shall not be used for concrete.

1903A.8 Concrete Testing.

1. ASTM C 192, Making and Curing Concrete Test Specimens in the Laboratory

2. ASTM C 31, Making and Curing Concrete Test Specimens in the Field

3. ASTM C 42, Obtaining and Testing Drilled Cores and Sawed Beams of Concrete

4. ASTM C 39, Compressive Strength of Cylindrical Concrete Specimens

5. ASTM C 172, Sampling Freshly Mixed Concrete

6. ASTM C 496, Splitting Tensile Strength of Cylindrical Concrete Specimens

7. ASTM C *1218, Water-Soluble Chloride in Mortar and Concrete.*

1903A.9 Concrete Mix.

1. ASTM C 94, Ready-mixed Concrete

2. ASTM C 685, Concrete Made by Volumetric Batching and Continuous Mixing

3. UBC Standard 19-2, Mill-mixed Gypsum Concrete and Poured Gypsum Roof Diaphragms.

4. ASTM C 109, Compressive Strength of Hydraulic Cement Mortars

5. ASTM C 567, Unit Weight of Structural Lightweight Concrete

1903A.10 Welding. *The welding of reinforcing steel, metal inserts and connections in reinforced concrete construction shall conform to UBC Standard 19-1.*

1903A.11 Glass Fiber Reinforced Concrete. Recommended Practice for Glass Fiber Reinforced Concrete Panels, *PCI* Manual 128.

SECTION 1904A — DURABILITY REQUIREMENTS

1904A.0 Notation.

f'_c = specified compressive strength of concrete, psi (MPa).

1904A.1 Water-Cementitious Materials Ratio.

1904A.1.1 The water-cementitious materials ratios specified in Tables 19A-A-2 and 19A-A-4 shall be calculated using the weight of cement meeting ASTM C 150, C 595 or C 845 plus the weight of fly ash and other pozzolans meeting ASTM C 618, slag meeting ASTM C 989, and silica fume meeting ASTM C 1240, if any, except that when concrete is exposed to deicing chemicals, Section 1904A.2.3 further limits the amount of fly ash, pozzolans, silica fume, slag or the combination of these materials.

1904A.2 Freezing and Thawing Exposures.

1904A.2.1 Normal-weight and lightweight concrete exposed to freezing and thawing or deicing chemicals shall be air entrained with air content indicated in Table 19A-A-1. Tolerance on air content as delivered shall be ± 1.5 percent. For specified compressive strength f'_c greater than 5,000 psi (34.47 MPa), reduction of air content indicated in Table 19A-A-1 by 1.0 percent shall be permitted.

1904A.2.2 Concrete that will be subjected to the exposures given in Table 19A-A-2 shall conform to the corresponding maximum water-cementitious materials ratios and minimum specified concrete compressive strength requirements of that table. In addition, concrete that will be exposed to deicing chemicals shall conform to the limitations of Section 1904A.2.3.

1904A.2.3 *For concrete exposed to deicing chemicals, the maximum weight of fly ash, or other pozzolans, silica fume or slag that is included in the concrete shall not exceed the percentages of the total weight of cementitious materials given in Table 19A-A-3.*

1904A.3 Sulfate Exposure.

1904A.3.1 Concrete to be exposed to sulfate-containing solutions or soils shall conform to the requirements of Table 19A-A-4 or shall be concrete made with a cement that provides sulfate resistance and that has a maximum water-cementitious materials ratio and minimum compressive strength set forth in Table 19A-A-4.

1904A.3.2 Calcium chloride as an admixture shall not be used in concrete to be exposed to severe or very severe sulfate-containing solutions, as defined in Table 19A-A-4.

1904A.4 Corrosion Protection of Reinforcement.

1904A.4.1 For corrosion protection of reinforcement in concrete, maximum water soluble chloride ion concentrations in hardened concrete at ages from 28 to 42 days contributed from the ingredients, including water, aggregates, cementitious materials and admixtures shall not exceed the limits of Table 19A-A-5. When testing is performed to determine water soluble chloride ion content, test procedures shall conform to ASTM C 1218.

1904A.4.2 If concrete with reinforcement will be exposed to chlorides from deicing chemicals, salt, salt water, brackish water, sea water or spray from these sources, requirements of Table 19A-A-2 for water-cementitious materials ratio and concrete strength and the minimum concrete cover requirements of Section 1907A.7 shall be satisfied. In addition, see Section 1918A.14 for unbonded prestressed tendons.

SECTION 1905A — CONCRETE QUALITY, MIXING AND PLACING

1905A.0 Notations.

f'_c = specified compressive strength of concrete, psi (MPa).

f'_{cr} = required average compressive strength of concrete used as the basis for selection of concrete proportions, psi (MPa).

f_{ct} = average splitting tensile strength of lightweight aggregate concrete, psi (MPa).

s = standard deviation, psi (MPa).

1905A.1 General.

1905A.1.1 Concrete shall be proportioned to provide *a required average compressive strength, f'_{cr} as prescribed in Section 1905A.3.2, and satisfy the durability criteria of Section 1904A. Concrete shall be produced to minimuze frequency of strength below f'_c as prescribed in Section 1905A.6.2.3.*

1905A.1.2 Requirements for f'_c shall be based on tests of cylinders made and tested as prescribed in Section 1905A.6.2.

1905A.1.3 Unless otherwise specified, f'_c shall be based on 28-day tests. If other than 28 days, test age for f'_c shall be as indicated *in* design drawings or specifications.

Design drawings shall show specified compressive strength of concrete f'_c for which each part of structure is designed.

The compressive strength used in design of normal-weight aggregate concrete shall not be less than 3,000 psi (20.7 MPa) except that 2,500 psi (17.2 MPa) concrete may be used in the design of footings for light one-story wood- or steel-framed buildings or other minor structures. In addition to these minima, the provisions of Section 1904A.2 shall apply where concrete is exposed to freezing and thawing exposure.

The compressive strength used in design of lightweight aggregate concrete shall not be less than 3,000 psi (20.7 MPa). The compressive strength used in the design of lightweight aggregate structural concrete shall not exceed 4,000 psi (27.6 MPa) unless specifically approved by the enforcement agency in accordance with Section 1921A.2.4.2.

1905A.1.4 Where design criteria in Sections 1909A.5.2.3, 1911A.2; and 1912A.2.4, provide for use of a splitting tensile strength value of concrete, laboratory tests shall be made to establish value of f_{ct} corresponding to specified values of f'_c.

1905A.1.5 Splitting tensile strength tests shall not be used as a basis for field acceptance of concrete.

1905A.2 Selection of Concrete Proportions.

1905A.2.1 Proportions of materials for concrete shall be established to provide:

1. Workability and consistency to permit concrete to be worked readily into forms and around reinforcement under conditions of placement to be employed without segregation or excessive bleeding.

2. Resistance to special exposures as required by Section 1904A.

3. Conformance with strength test requirements of Section 1905A.6.

1905A.2.2 Where different materials are to be used for different portions of proposed work, each combination shall be evaluated.

1905A.2.3 *Concrete specified by compressive strength shall be proportioned by one of the following methods:*

Method A. *As an alternate to Methods B and C, below, the compressive strength of 2,500 psi (17.2 MPa) and 3,000 psi (20.7 MPa) concrete shall be permitted to be used for structural concrete in accordance with Table 19A-A-8.*

Method B. *Method B mixes shall be based on existing, approved compressive strength test data for concrete mixes in accordance with Section 1905A.3.1.1.*

Method C. *Method C mixes shall be based on laboratory test results for trial concrete mixes in accordance with Section 1905A.3.3.2.*

A registered civil engineer with experience in concrete/mix design shall select the relative amounts of ingredients to be used as basic proportions of the concrete mixes proposed for use under the provisions of Methods B and C.

Method C shall be used to determine the mix proportions when materials not having a record of satisfactory performance are to be used.

1905A.3 Proportioning on the Basis of Field Experience *(Method B)* and Trial Mixtures *(Method C)*.

1905A.3.1 Standard deviation.

1905A.3.1.1 (Method B) with test records. Where a *testing laboratory acceptable to the enforcement agency has records of compressive strength tests,* a standard deviation shall be established. Test records *from which* a standard deviation *is* calculated *shall*:

1. Represent materials, quality control procedures and conditions similar to those expected, and changes in materials and proportions within the test records shall not have been more restricted than those for proposed work.

2. Represent concrete produced to meet a specified strength or strengths f'_c within *1,000* psi *(6.89* MPa) of that specified for proposed work.

3. Must consist of at least 30 consecutive tests or two groups of consecutive tests totaling at least 30 tests as defined in Section*s* *1905A.3.3 and* *1905A.6.1.4*, except as provided in Section 1905A.3.1.2.

1905A.3.1.2 Where a concrete production facility does not have test records meeting requirements of Section 1905A.3.1.1, but does have a record based on 15 to 29 consecutive tests, a standard deviation may be established as the product of the calculated standard deviation and the modification factor of Table 19A-A-6. *The compressive test records used for this calculation* must meet the requirements of Section 1905A.3.1.1, Items 1 and 2, and represent only a single record of consecutive tests that span a period of not less than 45 calendar days.

1905A.3.2 Required average strength.

1905A.3.2.1 Required average compressive strength f'_{cr} used as the basis for selection of concrete proportions shall be the larger of Formula (5A-1) or (5A-2) using a standard deviation calculated in accordance with Section 1905A.3.1.1 or 1905A.3.1.2.

$$f'_{cr} = f'_c + 1.34s \qquad \text{(5A-1)}$$

or

$$f'_{cr} = f'_c + 2.33s - 500 \qquad \text{(5A-2)}$$

For **SI:** $f'_{cr} = f'_c + 2.33s - 3.45$

1905A.3.2.2 *Where test records meeting the requirements of Section 1905A.3.1.1 or 1905A.3.1.2 above are not available, Method A or C shall be used.*

1905A.3.3 Documentation of average strength. Documentation that proposed concrete proportions will produce an average compressive strength equal to or greater than required average compressive strength (see Section 1905A.3.2) shall consist of a field strength test record, several strength test records, or trial mixtures.

1905A.3.3.1 When test records are used to demonstrate that proposed concrete proportions will produce the required average strength f'_{cr} (see Section 1905A.3.2), such records shall represent materials and conditions similar to those expected. Changes in materials, conditions and proportions within the test records shall not have been more restricted than those for proposed work. For the purpose of documenting average strength potential, test records consisting of less than 30 but not less than 10 consecutive tests may be used, provided test records encompass a period of time not less than 45 days. Required concrete proportions may be established by interpolation between the strengths and proportions of two or more test records each of which meets other requirements of this section.

1905A.3.3.2 ***(Method C) Design mix based on laboratory tests.*** When an acceptable record of field test results is not available, concrete proportions established from trial mixtures meeting the following restrictions shall be permitted:

1. Combination of materials shall be those for proposed work.
2. Trial mixtures having proportions and consistencies required for proposed work shall be made using at least three different water-cementitious materials ratios or cementitious materials contents that will produce a range of strengths encompassing the required average strength f'_{cr}.
3. Trial mixture shall be designed to produce a slump within ± 0.75 inch (± 19 mm) of maximum permitted, and for air-entrained concrete, within ± 0.5 percent of maximum allowable air content.
4. ***Water-cementitious ratio/content***
   4.1 [For DSA/SS] For each water-cementitious materials ratio or cementitious materials content, at least three test cylinders for each test age shall be made and cured *in accordance with ASTM standards.* Cylinders shall be tested at 28 days or at test age designated for determination of f'_c.
   4.2 [For OSHPD 1 & 4] For each water-cementitious materials ratio or cementitious materials content, at least three test cylinders for each test age shall be made and cured *in accordance with ASTM C31.* Cylinders shall be tested at 28 days or at test age designated for determination of f'_c.
5. From results of cylinder tests, a curve shall be plotted showing relationship between water-cementitious materials ratio or cementitious materials content and compressive strength at designated test age.
6. Maximum water-cementitious materials ratio or minimum cementitious materials content for concrete to be used in proposed work shall be that shown by the curve to produce the average strength required by Section 1905A.3.2, unless a lower water-cementitious materials ratio or higher strength is required by Section 1904A.
7. *Trial mixtures shall be selected by a registered civil engineer with experience in concrete mix design and tested by a testing laboratory acceptable to the enforcement agency.*

1905A.4 Proportioning without Field Experience or Trial Mixtures.

1905A.4.1 If data required by Section 1905A.3 are not available, concrete *shall be proportioned in accordance with Section 1905A.2.3 (Method A) but shall also be subject to the provisions of Section 1904A for special exposure requirements and to the compressive strength test criteria of Section 1905A.6.*

1905A.4.2 Concrete proportioned by *Method A* shall conform to the durability requirements of Section 1904A and to compressive strength test criteria of Section 1905A.6.

1905A.5 Average Strength Reduction.

1905A.5.1 [For OSHPD 1 & 4 and DSA/SS] As data becomes available during construction, *it shall be permitted to reduce* the amount by which f'_{cr} must exceed the specified value of f'_c, provided:

1. Thirty or more test results are available and average of test results exceeds that required by Section 1905A.3.2.1, using a standard deviation calculated in accordance with Section 1905A.3.1.1, or

2. Fifteen to 29 test results are available and average of test results exceeds that required by Section 1905A.3.2.1, using a standard deviation calculated in accordance with Section 1905A.3.1.2, and

3. Special exposure requirements of Section 1904A are met.

1905A.6 Evaluation and Acceptance of Concrete.

1905A.6.1 Frequency of testing.

1905A.6.1.1 Samples for strength tests of each class of concrete placed each day shall be taken not less than once a day, or not less than once for each *50* cubic yards *(345* m^3) of concrete, or not less than once for each *2,000* square feet *(186* m^2) of surface area for slabs or walls. *Additional samples for seven-day compressive strength tests shall be taken for each class of concrete at the beginning of the concrete work or whenever the mix or aggregate is changed.*

1905A.6.1.2 On a given project, if the total volume of concrete is such that the frequency of testing required by Section 1905A.6.1.1 would provide less than five strength tests for a given class of concrete, tests shall be made from at least five randomly selected batches or from each batch if fewer than five batches are used.

1905A.6.1.3 *Not adopted by the State of California.*

1905A.6.1.4 A strength test shall be the average of the strengths of two cylinders made from the same sample of concrete and tested at 28 days or at test age designated for determination of f'_c.

1905A.6.2 Laboratory-cured specimens.

1905A.6.2.1 ***Samples for strength.***

1905A.6.2.1.1 [For DSA/SS] Samples for strength tests shall be taken *in accordance with Section 1903A.8.*

SECTION 1921A — REINFORCED CONCRETE STRUCTURES RESISTING FORCES INDUCED BY EARTHQUAKE MOTIONS

1921A.0 Notations.

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

A_{cp} = area of concrete section, resisting shear, of an individual pier or horizontal wall segment, square inches (mm^2).

A_{cv} = net area of concrete section bounded by web thickness and length of section in the direction of shear force considered, square inches (mm^2).

A_g = gross area of section, square inches (mm^2).

A_j = effective cross-sectional area within a joint (see Section 1921A.5.3.1) in a plane parallel to plane of reinforcement generating shear in the joint. The joint depth shall be the overall depth of the column. Where a beam frames into a support of larger width, the effective width of the joint shall not exceed the smaller of:

1. beam width plus the joint depth
2. twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side. See Section 1921A.5.3.1.

A_{sh} = total cross-sectional area of transverse reinforcement (including crossties) within spacing, s, and perpendicular to dimension, h_c.

b = effective compressive flange width of a structural member, inches (mm).

b_w = web width, or diameter of circular section, inches (mm).

d = effective depth of section.

d_b = bar diameter.

E = load effects of earthquake, or related internal moments and forces.

f'_c = specified compressive strength of concrete, psi (MPa).

f_y = specified yield strength of reinforcement, psi (MPa).

f_{yh} = specified yield strength of transverse reinforcement, psi (MPa).

CA CAC H = loads due to weight and pressure of soil, water in soil, or other materials, or related to internal moments and forces.

h = overall dimension of member in the direction of action considered.

h_c = cross-sectional dimension of a column core or shear wall boundary zone measured center-to-center of confining reinforcement.

h_w = height of entire wall (diaphragm) or of the segment of wall (diaphragm) considered.

l_d = development length for a straight bar.

l_{dh} = development length for a bar with a standard hook as defined in Formula (21A-5).

l_o = minimum length, measured from joint face along axis of structural member, over which transverse reinforcement must be provided, inches (mm).

CA l_u = [For DSA/SS] unsupported length of compression member (see Section 1910A.11.3.1).

CAC l_u = [For OSHPD 1 & 4] height of column, center-to-center of floors and roofs.

l_w = length of entire wall (diaphragm) or of segment of wall (diaphragm) considered in direction of shear force.

M_{pr} = probable flexural strength of members, with or without axial load, determined using the properties of the member at the joint faces assuming a tensile strength in the longitudinal bars of at least 1.25 f_y and a strength-reduction factor ϕ of 1.0.

M_s = portion of slab moment balanced by support moment.

s = spacing of transverse reinforcement measured along the longitudinal axis of the structural member, inches (mm).

$S_{e\ CONNECTION}$ = *moment, shear or axial force at connection cross section other than the nonlinear action location corresponding to probable strength at the nonlinear action location, taking gravity load effects into consideration, per Section 1921A.2.7.3.*

$S_{n\ CONNECTION}$ = *nominal strength of connection cross section in flexural, shear or axial action, per Section 1921A.2.7.3.*

s_o = maximum spacing of transverse reinforcement, inches (mm).

V_c = nominal shear strength provided by concrete.

V_e = design shear force determined from Section 1921A.3.4.1 or 1921A.4.5.1.

V_n = nominal shear strength.

V_u = factored shear force at section.

α_c = coefficient defining the relative contribution of concrete strength to wall strength.

ρ = ratio of nonprestressed tension reinforcement = A_s/bd.

ρ_g = ratio of total reinforcement area to cross-sectional area of column.

ρ_n = ratio of distributed shear reinforcement on a plane perpendicular to plane of A_{cv}.

ρ_s = ratio of volume of spiral reinforcement to the core volume confined by the spiral reinforcement (measured out-to-out).

ρ_v = A_{sv}/A_{cv}; where A_{sv} is the projection on A_{cv} of area of distributed shear reinforcement crossing the plane of A_{cv}.

ϕ = strength-reduction factor.

l_u = height of column, center-to-center of floors and roofs. CAC

Δ_M = *0.7* $R\Delta_s$. A

Δ_s = *Design Level Response Displacement, which is the total drift or total story drift that occurs when the structure is subjected to the design seismic forces.*

ψ = *Dynamic Amplification Factor from Sections 1921A.2.7.3 and 1921A.2.7.4.*

1921A.1 Definitions. For the purposes of this section, certain terms are defined as follows:

BASE OF STRUCTURE is the level at which earthquake motions are assumed to be imparted to a building. This level does not necessarily coincide with the ground level.

BOUNDARY ELEMENTS ***(or ZONES)*** are portions along wall and diaphragm edges strengthened by longitudinal and transeverse reinforcement. Boundary elements do not necessarily require an increase in the thickness of the wall or diaphragm. Edges of openings within walls and diaphragms shall be provided with boundary elements if required by Sections 1921A.6.6.1 and 1921A.6.7.1.

COLLECTOR ELEMENTS are elements that serve to transmit the inertial forces with the diaphragms to members of the lateral-force-resisting systems.

CONFINED CORE is the area within the core defined by h_c.

CONNECTION *is an element that joins two precast members or a precast member and a cast-in-place member.*

COUPLING BEAMS are a horizontal element in plane with and connecting two shear walls.

CROSSTIE is a continuous reinforcing bar having a seismic hook at one end and a hook of not less than 90 degrees with at least six diameters at the other end. The hooks shall engage peripheral longitudinal bars. The 90-degree hooks of two successive crossties engaging the same longitudinal bar shall be alternated end for end.

DESIGN LOAD COMBINATIONS are combinations of factored loads and forces specified in Sections *1612A.2.1 and* 1909A.2.

DEVELOPMENT LENGTH FOR A BAR WITH A STANDARD HOOK is the shortest distance between the critical section (where the strength of the bar is to be developed) and a tangent to the outer edge of the 90-degree hook.

DRY CONNECTION *is a connection used between precast members, which does not qualify as a wet connection.*

FACTORED LOADS AND FORCES are the specified loads and forces modified by the factors in Sections *1612A.2.1 and 1909A.2.*

HOOP is a closed tie or continuously wound tie. A closed tie can be made up of several reinforcing elements, each having seismic hooks at both ends. A continuously wound tie shall have a seismic hook at both ends.

JOINT *is the geometric volume common to intersecting members.*

LATERAL-FORCE-RESISTING SYSTEM is that portion of the structure composed of members proportioned to resist forces related to earthquake effects.

LIGHTWEIGHT-AGGREGATE CONCRETE is all lightweight or sanded lightweight aggregate concrete made with lightweight aggregates conforming to Section 1903A.3.

NONLINEAR ACTION LOCATION *is the center of the region of yielding in flexure, shear or axial action.*

NONLINEAR ACTION REGION *is the member length over which nonlinear action takes place. It shall be taken as extending a distance of no less than* h/2 *on either side of the nonlinear action location.*

SEISMIC HOOK is a hook on a stirrup, hoop or crosstie having a bend not less than 135 degrees with a six-bar-diameter [but not less than 3 inches (76 mm)], extension that engages the longitudinal reinforcement and projects into the interior of the stirrup or hoop.

SHELL CONCRETE is concrete outside the transverse reinforcement confining the concrete.

SPECIFIED LATERAL FORCES are lateral forces corresponding to the appropriate distribution of the design base shear force prescribed by the governing code for earthquake-resistant design.

STRONG CONNECTION *is a connection that remains elastic, while the designated nonlinear action regions undergo inelastic response under the Design Basis Ground Motion.*

STRUCTURAL DIAPHRAGMS are structural members, such as floor and roof slabs, which transmit inertial forces to lateral-force-resisting members.

STRUCTURAL TRUSSES are assemblages of reinforced concrete members subjected primarily to axial forces.

STRUT is an element of a structural diaphragm used to provide continuity around an opening in the diaphragm.

TIE ELEMENTS are elements which serve to transmit inertia forces and prevent separation of such building components as footings and walls.

WALL PIER *is a wall segment with a horizontal length-to-thickness ratio between 2.5 and 6, and whose clear height is at least two times its horizontal length.*

WET CONNECTION *uses any of the splicing methods, per Section 1921A.2.6.1 or 1921A.3.2.3, to connect precast members and uses cast-in-place concrete or grout to fill the splicing closure.*

1921A.2 General Requirements.

1921A.2.1 Scope.

1921A.2.1.1 *Section 1921A* contains special requirements for design and construction of reinforced concrete members of a structure for which the design forces, related to earthquake motions, have been determined on the basis of energy dissipation in the nonlinear range of response.

1921A.2.1.2 The provisions of Sections 1901A through 1918A shall apply except as modified by the provisions of Section 1921A.

1921A.2.1.3 <u>Not adopted by the State of California.</u>

1921A.2.1.4 *In Seismic Zones 3 and 4,* all reinforced concrete structural members that are part of the lateral-force-resisting system shall satisfy the requirements of Sections 1921A.2 through 1921A.7, in addition to the requirements of Sections 1901A through 1917A.

1921A.2.1.5 A reinforced concrete structural system not satisfying the requirements of this section may be used if it is demonstrated by experimental evidence and analysis that the proposed system will have strength and toughness equal to or exceeding those provided by a comparable monolithic reinforced concrete structure satisfying this section.

1921A.2.1.6 *Precast lateral-force-resisting systems shall satisfy either of the following criteria:*

1. Emulate the behavior of monolithic reinforced concrete construction and satisfy Section 1921A.2.2.5, or

2. Rely on the unique properties of a structural system composed of interconnected precast elements and conform to Section 1629A.9.2.

1921A.2.1.7 *In structures having precast gravity systems, the lateral-force-resisting system shall be one of the systems listed in Table 16A-N and shall be well distributed using one of the following methods:*

1. The lateral-force-resisting systems shall be spaced such that the span of the diaphragm or diaphragm segment between lateral-force-resisting systems shall be no more than three times the width of the diaphragm or diaphragm segment.

Where the lateral-force-resisting system consists of moment-resisting frames, at least $[(N_b/4) + 1]$ *of the bays (rounded up to the nearest integer) along any frame line at any story shall be part of the lateral-force-resisting system, where* N_b *is the total number of bays along that line at that story. This requirement applies to only* *the lower two thirds of the stories of buildings three stories or taller.*

2. All beam-to-column connections that are not part of the lateral-force-resisting system shall be designed in accordance with the following:

Connection design force. *The connection shall be designed to develop strength M. M is the moment developed at the connection when the frame is displaced by Δ_S assuming fixity at the connection and a beam flexural stiffness of no more than one-half of the gross section stiffness. M shall be sustained through a deformation of Δ_M.*

Connection characteristics. *The connection shall be permitted to resist moment in one direction only, positive or negative. The connection at the opposite end of the member shall resist moment with same positive or negative sign. The connection shall be permitted to have zero flexural stiffness up to a frame displacement of Δ_S.*

In addition, complete calculations for the deformation compatibility of the gravity load carrying system shall be made in accordance with Section 1633A.2.4 using cracked section stiffnesses in the lateral-force-resisting system and the diaphragm.

Where gravity columns are not provided with lateral support on all sides, a positive connection shall be provided along each unsupported direction parallel to a principal plan axis of the structure. The connection shall be designed for a horizontal force equal to 4 percent of the axial load strength (P_0) of the column.

The bearing length shall be 2 inches (51 mm) more than that required for bearing strength.

1921A.2.2 Analysis and proportioning of structural members.

1921A.2.2.1 The interaction of all structural and nonstructural members which materially affect the linear and nonlinear response of the structure to earthquake motions shall be considered in the analysis.

1921A.2.2.2 Rigid members assumed not to be a part of the lateral-force-resisting system shall be permitted, provided their effect on the response of the system is considered and accommodated in the structural design. Consequences of failure of structural and nonstructural members which are not a part of the lateral-force-resisting system shall also be considered.

1921A.2.2.3 Structural members below base of structure required to transmit to the foundation forces resulting from earthquake effects shall also comply with the requirements of Section 1921A.

1921A.2.2.4 All structural members assumed not to be part of the lateral-force-resisting system shall conform to Section 1921A.7.

1921A.2.2.5 *Precast structural systems using frames and emulating the behavior of monolithic reinforced concrete construction shall satisfy either Section 1921A.2.2.6 or 1921A.2.2.7.*

1921A.2.2.6 *Precast structural systems, utilizing wet connections, shall comply with all the applicable requirements of monolithic concrete construction for resisting seismic forces.*

1921A.2.2.7 *Precast structural systems not meeting Section 1921A.2.2.6 shall utilize strong connections resulting in nonlinear response away from connections. Design shall satisfy the requirements of Section 1921A.2.7 in addition to all the applicable requirements of monolithic concrete construction for resisting seismic forces, except that provisions of Section 1921A.3.1.2 shall apply to the segments between nonlinear action locations.*

1921A.2.3 Strength-reduction factors. Strength-reduction factors shall be as given in Section 1909A.3.4.

1921A.2.4 Concrete in members resisting earthquake-induced forces.

1921A.2.4.1 Compressive strength f'_c shall not be less than 3,000 psi (20.69 MPa). *Members with compressive strengths that shall not be less than 3,000 psi (20.7 MPa) include all structural members above and below the base, including foundations which are required to resist the forces resulting from earthquake loading, except foundations for one-story wood-frame or one-story light-steel buildings and isolated mat-type foundations supporting equipment only may be designed and constructed for compressive strengths not less than 2,500 psi (17.2 MPa).*

1921A.2.4.2 Compressive strength of lightweight-aggregate concrete used in design shall not exceed 4,000 psi (27.58 MPa). Lightweight aggregate concrete with higher design compressive strength shall be permitted if demonstrated by experimental evidence that structural members made with that lightweight aggregate concrete provide strength and toughness equal to or exceeding those of comparable members made with normal-weight aggregate concrete of the same strength. *In no case shall the compressive strength of lightweight concrete used in design exceed 6,000 psi (41.37 MPa).*

1921A.2.5 Reinforcement in members resisting earthquake-induced forces.

1921A.2.5.1 ***Alloy A 706 reinforcement.*** *Except as permitted in Sections 1921A.2.5.2 through 1921A.2.5.5,* reinforcement resisting earthquake-induced flexural and axial forces in frame members and in wall boundary elements shall comply with *low alloy* A 706 except as allowed in Section 1921A.2.5.2.

1921A.2.5.2 Billet steel A 615 reinforcement. *Billet steel A 615 Grades 40 and 60 reinforcement shall be permitted to be used in frame members and wall boundary elements* if (1) the actual yield strength based on mill tests does not exceed the specified yield strength by more than 18,000 psi (124.1 MPa) [retests shall not exceed this value by more than an additional 3,000 psi (20.69 MPa)], and (2) the ratio of the actual ultimate tensile stress to the actual yield strength is not less than 1.25.

1921A.2.5.3 *The average prestress f_{pc}, calculated for an area equal to the member's shortest cross-sectional dimension multiplied by the perpendicular dimension, shall be the lesser of 350 psi (2.41 MPa) or $f'_c/12$ at locations of nonlinear action, where prestressing tendons are used in members of frames.*

1921A.2.5.4 *For members in which prestressing tendons are used together with mild reinforcement to resist earthquake-induced forces, prestressing tendons shall not provide more than one-quarter of the strength for both positive and negative moments at the joint face and shall extend through exterior joints and be anchored at the exterior face of the joint or beyond.*

1921A.2.5.5 *Shear strength provided by prestressing tendons shall not be considered in design.*

1921A.2.6 Welded splices and mechanically connected reinforcement.

1921A.2.6.1 Reinforcement resisting earthquake-induced flexural or axial forces in frame members or in wall boundary members shall be permitted to be spliced using welded splices or mechanical connectors conforming to Section 1912A.14.3.3 or 1912A.14.3.4.

Splice locations in frame members shall conform to Section 1921A.2.6.1.1 or 1921A.2.6.1.2.

1921A.2.6.1.1 ***Welded splices.*** *In Seismic Zones 2, 3 and 4, welded splices on billet steel A 615 or low allow A 706 reinforcement shall not be used within an anticipated plastic hinge region nor within a distance of one beam depth on either side of the plastic hinge region or within a joint.*

1921A.2.6.1.2 ***Mechanical connection splices.*** *Splices with mechanical connections shall be classified according to strength capacity as follows:*

Type 1 splice. *Mechanical connections meeting the requirements of Sections 1912A.14.3.4 and 1912A.14.3.5.*

Type 2 splice. *Mechanical connections that develop in tension the lesser of 95 percent of the ultimate tensile strength or 160 percent of specified yield strength,* f_y, *of the bar.*

Mechanical connection splices shall be permitted to be located as follows:

Type 1 splice. *In Seismic Zone 1, a Type 1 splice shall be permitted in any location within a member. In Seismic Zones 2, 3 and 4, a Type 1 splice shall not be used within an anticipated plastic hinge region or within a distance of one beam depth on either side of the plastic hinge region or within a joint.*

Type 2 splice. *A Type 2 splice shall be permitted in any location within a member.*

1921A.2.6.2 Welding of stirrups, ties, inserts or other similar elements to longitudinal reinforcement required by design shall not be permitted.

1921A.2.7 ***Emulation of monolithic construction using strong connections.*** *Members resisting earthquake-induced forces in precast frames using strong connections shall satisfy the following:*

1921A.2.7.1 ***Location.*** *Nonlinear action location shall be selected so that there is a strong column/weak beam deformation mechanism under seismic effects. The nonlinear action location shall be no closer to the near face of strong connection than h/2. For column-to-footing connections, where nonlinear action may occur at the column base to complete the mechanism, the nonlinear action location shall be no closer to the near face of the connection than h/2.*

1921A.2.7.2 ***Anchorage and splices.*** *Reinforcement in the nonlinear action region shall be fully developed outside both the strong connection region and the nonlinear action region. Noncontinuous anchorage reinforcement of strong connection shall be fully developed between the connection and the beginning of nonlinear action region. Lap splices are prohibited within connections adjacent to a joint.*

1921A.2.7.3 ***Design forces.*** *Design strength of strong connections shall be based on*

$$\phi S_{n\,CONNECTION} > \psi S_{e\,CONNECTION} \qquad (21A\text{-}1)$$

Dynamic amplification factor ψ *shall be taken as 1.0.*

1921A.2.7.4 ***Column-to-column connection.*** *The strength of such connections shall comply with Section 1921A.2.7.3 with* ψ *taken as 1.4. Where column-to-column connections occur, the columns shall be provided with transverse reinforcement as specified in Sections 1921A.4.4.1 through 1921A.4.4.3 over their full height if the factored axial compressive force in these members, including seismic effects, exceeds* $A_g f'_c/10$.

EXCEPTION: *Where column-to-column connection is located within the middle third of the column clear height, the following shall apply: (1) The design moment strength* ϕM_n *of the connection shall not be less than 0.4 times the maximum* M_{pr} *for the column within the story height, and (2) the design shear strength* ϕV_n *of the connection shall not be less than that determined per Section 1921A.4.5.1.*

1921A.2.7.5 ***Column-face connection.*** *Any strong connection located outside the middle half of a beam span shall be a wet connection, unless a dry connection can be substantiated by approved cyclic test results. Any mechanical connector located within such a column-face strong connection shall develop in tension or compression, as required, at least 140 percent of specified yield strength,* f_y, *of the bar.*

1921A.3 Flexural Members of Frames.

1921A.3.1 Scope. Requirements of this section apply to frame members (1) resisting earthquake-induced forces and (2) proportioned primarily to resist flexure. These frame members shall also satisfy the following conditions:

1921A.3.1.1 Factored axial compressive force on the member shall not exceed $(A_g f'_c/10)$.

1921A.3.1.2 Clear span for the members shall not be less than four times its effective depth.

1921A.3.1.3 The width-to-depth ratio shall not be less than 0.3.

1921A.3.1.4 The width shall not be (1) less than 10 inches (254 mm) and (2) more than the width of the supporting member (measured on a plane perpendicular to the longitudinal axis of the flexural member) plus distances on each side of the supporting member not exceeding three fourths of the depth of the flexural member.

1921A.3.2 Longitudinal reinforcement.

1921A.3.2.1 At any section of a flexural member, except as provided in Section 1910A.5.3, for top as well as for bottom reinforcement, the amount of reinforcement shall not be less than that given by Formula (10A-3) but not less than 200 $b_w d/f_y$, (For **SI:** 1.38 $b_w d/f_y$) and the reinforcement ratio, ρ, shall not exceed 0.025. At least two bars shall be provided continuously, both top and bottom.

1921A.3.2.2 Positive-moment strength at joint face shall not be less than one half of the negative-moment strength provided at that face of the joint. Neither the negative nor the positive-moment strength at any section along member length shall be less than one fourth the maximum moment strength provided at face of either joint.

1921A.3.2.3 Lap splices of flexural reinforcement shall be permitted only if hoop or spiral reinforcement is provided over the lap length. Maximum spacing of the transverse reinforcement enclosing the lapped bars shall not exceed $d/4$ or 4 inches (102 mm). Lap splices shall not be used (1) within the joints, (2) within a distance of twice the member depth from the face of joint, and (3) at locations where analysis indicates flexural yielding caused by inelastic lateral displacements of the frame.

1921A.3.2.4 Welded splices and mechanical connections shall conform to Section 1921A.2.6.1.

1921A.3.3 Transverse reinforcement.

1921A.3.3.1 Hoops shall be provided in the following regions of frame members:

1. Over a length equal to twice the member depth measured from the face of the supporting member toward midspan, at both ends of the flexural members.

2. Over lengths equal to twice the member depth on both sides of a section where flexural yielding may occur in connection with inelastic lateral displacements of the frame.

For joints confined on all four faces . . . . $20\sqrt{f'_c}A_j$
(For **SI:** $1.66\sqrt{f'_c}A_j$)

For joints confined on three faces or on two opposite faces . . . . $15\sqrt{f'_c}A_j$
(For **SI:** $1.25\sqrt{f'_c}A_j$)

For others . . . . $12\sqrt{f'_c}A_j$
(For **SI:** $1.00\sqrt{f'_c}A_j$)

A member that frames into a face is considered to provide confinement to the joint if at least three fourths of the face of the joint is covered by the framing member. A joint is considered to be confined *if* such confining members frame into all faces of the joint.

1921A.5.3.2 For lightweight aggregate concrete, the nominal shear strength of the joint shall not exceed three fourths of the limits for normal-weight aggregate concrete.

1921A.5.4 Development length for reinforcement in tension.

1921A.5.4.1 The development length, l_{dh}, for a bar with a standard 90-degree hook in normal-weight aggregate concrete shall not be less than $8d_b$, 6 inches (152 mm), and the length required by Formula (21A-5).

$$l_{dh} = f_y d_b / 65\sqrt{f'_c} \qquad \text{(21A-5)}$$

For **SI:** $$l_{dh} = f_y d_b / 5.4\sqrt{f'_c}$$

for bar sizes No. 3 through No. 11.

For lightweight aggregate concrete, the development length for a bar with a standard 90-degree hook shall not be less than $10d_b$, 7.5 inches (191 mm), and 1.25 times that required by Formula (21A-5).

The 90-degree hook shall be located within the confined core of a column or of a boundary member.

1921A.5.4.2 For bar sizes No. 3 through No. 11, the development length, l_d, for a straight bar shall not be less than (1) 2.5 times the length required by Section 1921A.5.4.1 if the depth of the concrete cast in one lift beneath the bar does not exceed 12 inches (305 mm), and (2) 3.5 times the length required by Section 1921A.5.4.1 if the depth of the concrete cast in one lift beneath the bar exceeds 12 inches (305 mm).

1921A.5.4.3 Straight bars terminated at a joint shall pass through the confined core of a column or of a boundary member. Any portion of the straight embedment length not within the confined core shall be increased by a factor of 1.6.

1921A.5.4.4 If epoxy-coated reinforcement is used, the development lengths in Sections 1921A.5.4.1 through Section 1921A.5.4.3 shall be multiplied by the applicable factor specified in Section 1912A.2.4 or 1912A.5.3.6.

***1921A.5.4.5** Splices shall be based on the development length, ℓ_d, for a straight bar as determined by Sections 1921A.5.4.1 and 1921A.5.4.2 and modified by the factors in Section 1912A.*

1921A.6 *Shear* Walls, Diaphragms and Trusses.

1921A.6.1 Scope. The requirements of this section apply to *shear* walls and trusses serving as parts of the earthquake-force-resisting systems as well as to diaphragms, struts, ties, chords and collector members which transmit forces induced by earthquake.

1921A.6.2 Reinforcement.

1921A.6.2.1 The reinforcement ratio, ρ_v, for *shear* walls shall not be less than 0.0025 along the longitudinal and transverse axes. If the design shear force does not exceed $A_{cv}\sqrt{f'_c}$ (For **SI:** $0.08A_{cv}\sqrt{f'_c}$) , the minimum reinforcement for *shear* walls shall be in conformance with Section 1914A.3. The minimum reinforcement ratio for structural diaphragms shall be in conformance with Section 1907A.12. Reinforcement spacing each way in *shear* walls and diaphragms shall not exceed 18 inches (457 mm). Reinforcement provided for shear strength shall be continuous and shall be distributed across the shear plane.

1921A.6.2.2 At least two curtains of reinforcement shall be used in a wall if the in-plane factored shear force assigned to the wall exceeds $2A_{cv}\sqrt{f'_c}$ (For **SI:** $0.166A_{cv}\sqrt{f'_c}$).

When V_u in the plane of the wall exceeds $A_{cv}\sqrt{f'_c}$ (For **SI:** $0.08A_{cv}\sqrt{f'_c}$) *, horizontal reinforcement terminating at the edges of shear walls shall have a standard hook engaging the edge reinforcement, or the edge reinforcement shall be enclosed in "U" stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement.*

Where boundary members are not required by Section 1921A.6.2.3, minimum reinforcement parallel to the edges of all diaphragms and the boundaries of all openings shall consist of twice the cross-sectional area of the minimum shear reinforcement required per lineal foot of wall.

1921A.6.2.3 Structural-truss elements, struts, ties and collector elements with compressive stresses exceeding $0.2\,f'_c$ shall have special transverse reinforcement, as specified in Section 1921A.4.4, over the total length of the element. The special transverse reinforcement may be discontinued at a section where the calculated compressive stress is less than $0.15\,f'_c$. Stresses shall be calculated for the factored forces using a linearly elastic model and gross-section properties of the elements considered.

1921A.6.2.4 All continuous reinforcement in *shear* walls, diaphragms, trusses, struts, ties, chords and collector elements shall be anchored or spliced in accordance with the provisions for reinforcement in tension as specified in Section 1921A.5.4.

1921A.6.3 Design forces. The design shear force V_u shall be obtained from the lateral load analysis in accordance with the factored loads and combinations specified in Section 1909A.2 *and as modified in Section 1612A.2.1.*

1921A.6.4 Diaphragms. *See Sections 1921A.6.11 and 1921A.6.12.*

1921A.6.5 Shear strength.

1921A.6.5.1 Nominal shear strength of *shear* walls and diaphragms shall be determined using either Section 1921A.6.5.2 or 1921A.6.5.3.

1921A.6.5.2 Nominal shear strength, V_n, of *shear* walls and diaphragms shall be assumed not to exceed the shear force calculated from

$$V_n = A_{cv}(2\sqrt{f'_c} + \rho_n f_y) \qquad \text{(21A-6)}$$

For **SI:** $$V_n = A_{cv}(0.166\sqrt{f'_c} + \rho_n f_y)$$

1921A.6.5.3 For walls (diaphragms) and wall (diaphragm) segments having a ratio of (h_w/l_w) less than 2.0, nominal shear strength of wall (diaphragm) shall be determined from Formula (21A-7)

$$V_n = A_{cv}(\alpha_c\sqrt{f'_c} + \rho_n f_y) \qquad \text{(21A-7)}$$

For **SI:** $V_n = A_{cv}(0.08\alpha_c\sqrt{f'_c} + \rho_n f_y)$

Where the coefficient α_c varies linearly from 3.0 for $h_w/l_w = 1.5$ to 2.0 for $h_w/l_w = 2.0$.

1921A.6.5.4 In Section 1921A.6.5.3 above, the value of ratio (h_w/l_w) used for determining V_n for segments of a wall or diaphragm shall be the largest of the ratios for the entire wall (diaphragm) and the segment of wall (diaphragm) considered.

1921A.6.5.5 Walls (diaphragms) shall have distributed shear reinforcement providing resistance in two orthogonal directions in the plane of the wall (diaphragm). If the ratio (h_w/l_w) does not exceed 2.0, reinforcement ratio ρ_v shall not be less than reinforcement ratio ρ_n.

1921A.6.5.6 Nominal shear strength of all wall piers sharing a common lateral force shall not be assumed to exceed $8A_{cv}\sqrt{f'_c}$ (For **SI:** $0.66A_{cv}\sqrt{f'_c}$) where A_{cv} is the total cross-sectional area and the nominal shear strength of any one of the individual wall piers shall not be assumed to exceed $10A_{cp}\sqrt{f'_c}$ (For **SI:** $0.83A_{cp}\sqrt{f'_c}$) where A_{cp} represents the cross-sectional area of the pier considered.

1921A.6.5.7 Nominal shear strength of horizontal wall segments shall not be assumed to exceed $10A_{cp}\sqrt{f'_c}$ (For **SI:** $0.83A_{cv}\sqrt{f'_c}$) where A_{cp} represents the cross-sectional area of a horizontal wall segment.

1921A.6.6 Design of shear walls for flexural and axial loads.

1921A.6.6.1 Shear walls and portions of shear walls subject to combined flexural and axial loads shall be designed in accordance with Sections 1910A.2 and 1910A.3, except Section 1910A.3.6 and the nonlinear strain requirements of Section 1910A.2.2 do not apply. The strength-reduction factor ϕ shall be in accordance with Section 1909A.3.

1921A.6.6.2 The effective flange widths to be used in the design of I-, L-, C- or T-shaped sections shall not be assumed to extend further from the face of the web than (1) one half the distance to an adjacent shear wall web, or (2) 15 percent of the total wall height for the flange in compression or 30 percent of the total wall height for the flange in tension, not to exceed the total projection of the flange.

1921A.6.6.3 Walls for resisting earthquake-induced forces.

1921A.6.6.3.1 [For DSA/SS] Walls and portions of walls with $P_u > 0.35P_o$ shall not be considered to contribute to the calculated strength of the structure for resisting earthquake-induced forces. Such walls shall conform to the requirements of Section 1631A.2, Item 4.

1921A.6.6.3.2 [For OSHPD 1 & 4] Walls and portions of walls with $P_u > 0.35P_o$ shall not be considered to contribute to the calculated strength of the structure for resisting earthquake-induced forces. Such walls shall conform to the requirements of Section 1633A.2.4, Item 4.

1921A.6.6.4 Shear wall boundary zone detail requirements as defined in Section 1921A.6.6.6 need not be provided in shear walls or portions of shear walls meeting the following conditions:

1. $P_u \le 0.10A_g f'_c$ for geometrically symmetrical wall sections $P_u \le 0.05A_g f'_c$ for geometrically unsymmetrical wall sections

and either

2. $\frac{M_u}{V_u l_w} \le 1.0$

or

3. $V_u \le 3A_{cv}\sqrt{f'_c}$ *and* $\frac{M_u}{V_u l_w} \le 3$ (For **SI:** $V_u \le 0.25A_{cv}\sqrt{f'_c}$)

Shear walls and portions of shear walls not meeting the conditions of Section 1921A.6.6.4 and having $P_u < 0.35P_o$ shall have boundary zones at each end a distance varying linearly from 0.25 l_w to 0.15 l_w for P_u varying from 0.35 P_o to 0.15 P_o. The boundary zone shall have minimum length of 0.15 l_w and shall be detailed in accordance with Section 1921A.6.6.6.

Where boundary members are not required by Section 1921A.6.6.4, minimum reinforcement parallel to the edges of all shear walls and the boundaries of all openings shall consist of twice the cross-sectional area of the minimum shear reinforcement required per lineal foot of wall.

1921A.6.6.5 Boundary zones.

1921A.6.6.5.1 [For DSA/SS] Alternatively, the requirements for boundary zones in shear walls or portions of shear walls not meeting the conditions of Section 1921A.6.6.4 may be based on determination of the compressive strain levels at edges when the wall or portion of wall is subjected to displacement levels resulting from the ground motions specified in Section 1629A.2 using cracked section properties and considering the response modification effects of possible nonlinear behavior of the building.

Boundary zone detail requirements as defined in Section 1921A.6.6.6 shall be provided over those portions of the wall where compressive strains exceed 0.003. In no instance shall designs be permitted in which compressive strains exceed ε_{max}.

1921A.6.6.5.2 [For OSHPD 1 & 4] Alternatively, the requirements for boundary zones in shear walls or portions of shear walls not meeting the conditions of Section 1921A.6.6.4 may be based on determination of the compressive strain levels at edges when the wall or portion of wall is subjected to displacement levels resulting from the ground motions specified in Section 1631A.2 using cracked section properties and considering the response modification effects of possible nonlinear behavior of the building.

Boundary zone detail requirements as defined in Section 1921A.6.6.6 shall be provided over those portions of the wall where compressive strains exceed 0.003. In no instance shall designs be permitted in which compressive strains exceed ε_{max}.

WHERE:

$$\epsilon_{max} = 0.015 \quad (21A\text{-}8)$$

1. Using the displacement of Section 1921A.6.6.5, determine the curvature of the wall cross section at each location of potential flexural yielding assuming the possible nonlinear response of the wall and its elements. Using a strain compatibility analysis of the wall cross section, determine the compressive strains resulting from these curvatures.

2. For shear walls in which the flexural limit state response is governed by yielding at the base of the wall, compressive strains at wall edges may be approximated as follows:

Determine the total curvature demand (ϕ_t) as given in Formula (21A-9):

$$\phi_t = \frac{\Delta_i}{(h_w - l_p/2)l_p} + \phi_y \quad (21A\text{-}9)$$

WHERE:

c'_u = *neutral axis depth at P'_u and M'_n.*

$$V_n = 2A_{mv}\sqrt{f'_m} \qquad (8\text{-}26)$$

For **SI:** $$V_n = 0.166A_{mv}\sqrt{f'_m}$$

2108.2.4.6 Deflection design. The midheight deflection, Δ_s, under service lateral and vertical loads (without load factors) shall be limited by the relation:

$$\Delta_s = 0.007h \qquad (8\text{-}27)$$

$P\Delta$ effects shall be included in deflection calculation. The midheight deflection shall be computed with the following formula:

$$\Delta_s = \frac{5\,M_s h^2}{48\,E_m I_g} \text{ for } M_{ser} \leq M_{cr} \qquad (8\text{-}28)$$

$$\Delta_s = \frac{5\,M_{cr} h^2}{48\,E_m I_g} + \frac{5\,(M_{ser} - M_{cr})h^2}{48\,E_m I_{cr}} \text{ for } M_{cr} < M_{ser} < M_n \qquad (8\text{-}29)$$

The cracking moment strength of the wall shall be determined from the formula:

$$M_{cr} = Sf_r \qquad (8\text{-}30)$$

The modulus of rupture, f_r, shall be as follows:

1. For fully grouted hollow-unit masonry,

$$f_r = 4.0\sqrt{f'_m}\text{, 235 psi maximum} \qquad (8\text{-}31)$$

For **SI:** $$f_r = 0.33\sqrt{f'_m}\text{, 1.6 MPa maximum}$$

2. For partially grouted hollow-unit masonry,

$$f_r = 2.5\sqrt{f'_m}\text{, 125 psi maximum} \qquad (8\text{-}32)$$

For **SI:** $$f_r = 0.21\sqrt{f'_m}\text{, 861 kPa maximum}$$

3. For two-wythe brick masonry,

$$f_r = 2.0\sqrt{f'_m}\text{, 125 psi maximum} \qquad (8\text{-}33)$$

For **SI:** $$f_r = 0.166\sqrt{f'_m}\text{, 861 kPa maximum}$$

2108.2.5 Wall design for in-plane loads.

2108.2.5.1 General. The requirements of this section are for the design of walls for in-plane loads.

The value of f'_m shall not be less than 1,500 psi (10.3 MPa) nor greater than 4,000 psi (27.6 MPa).

2108.2.5.2 Reinforcement. Reinforcement shall be in accordance with the following:

1. Minimum reinforcement shall be provided in accordance with Section 2106.1.12.4, Item 2.3, for all seismic areas using this method of analysis.

2. When the shear wall failure mode is in flexure, the nominal flexural strength of the shear wall shall be at least 1.8 times the cracking moment strength of a fully grouted wall or 3.0 times the cracking moment strength of a partially grouted wall from Formula (8-30).

3. The amount of vertical reinforcement shall not be less than one half the horizontal reinforcement.

4. Spacing of horizontal reinforcement within the region defined in Section 2108.2.5.5, Item 3, shall not exceed three times the nominal wall thickness nor 24 inches (610 mm).

2108.2.5.3 Design strength. Design strength provided by the shear wall cross section in terms of axial force, shear and moment shall be computed as the nominal strength multiplied by the applicable strength-reduction factor, ϕ, specified in Section 2108.1.4.3.

2108.2.5.4 Axial strength. The nominal axial strength of the shear wall supporting axial loads only shall be calculated by Formula (8-34).

$$P_o = 0.85\,f'_m\,(A_e - A_s) + f_y A_s \qquad (8\text{-}34)$$

Axial design strength provided by the shear wall cross section shall satisfy Formula (8-35).

$$P_u \leq 0.80\,\phi\,P_o \qquad (8\text{-}35)$$

2108.2.5.5 Shear strength. Shear strength shall be as follows:

1. The nominal shear strength shall be determined using either Item 2 or 3 below. Maximum nominal shear strength values are determined from Table 21-J.

2. The nominal shear strength of the shear wall shall be determined from Formula (8-36), except as provided in Item 3 below

$$V_n = V_m + V_s \qquad (8\text{-}36)$$

WHERE:

$$V_m = C_d A_{mv}\sqrt{f'_m} \qquad (8\text{-}37)$$

For **SI:** $$V_m = 0.083\,C_d A_{mv}\sqrt{f'_m}$$

and

$$V_s = A_{mv}\,\rho_n f_y \qquad (8\text{-}38)$$

3. For a shear wall whose nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength, two shear regions exist.

For all cross sections within the region defined by the base of the shear wall and a plane at a distance L_w above the base of the shear wall, the nominal shear strength shall be determined from Formula (8-39).

$$V_n = A_{mv}\,\rho_n f_y \qquad (8\text{-}39)$$

The required shear strength for this region shall be calculated at a distance $L_w/2$ above the base of the shear wall, but not to exceed one half story height.

For the other region, the nominal shear strength of the shear wall shall be determined from Formula (8-36).

2108.2.5.6 Boundary members. Boundary members shall be as follows:

1. Boundary members shall be provided at the boundaries of shear walls when the compressive strains in the wall exceed 0.0015. The strain shall be determined using factored forces and R equal to 1.1.

2. The minimum length of the boundary member shall be three times the thickness of the wall, but shall include all areas where the compressive strain per Section 2108.2.6.2.7 is greater than 0.0015.

3. Lateral reinforcement shall be provided for the boundary elements. The lateral reinforcement shall be a minimum of No. 3 bars at a maximum of 8-inch (203 mm) spacing within the grouted core or equivalent confinement which can develop an ultimate compressive masonry strain of at least 0.006.

2108.2.6 Design of moment-resisting wall frames.

2108.2.6.1 General requirements.

2108.2.6.1.1 Scope. The requirements of this section are for the design of fully grouted moment-resisting wall frames constructed of reinforced open-end hollow-unit concrete or hollow-unit clay masonry.

2108.2.6.1.2 Dimensional limits. Dimensions shall be in accordance with the following.

Beams. Clear span for the beam shall not be less than two times its depth.

The nominal depth of the beam shall not be less than two units or 16 inches (406 mm), whichever is greater. The nominal beam depth to nominal beam width ratio shall not exceed 6.

The nominal width of the beam shall be the greater of 8 inches (203 mm) or $^1/_{26}$ of the clear span between pier faces.

Piers. The nominal depth of piers shall not exceed 96 inches (2438 mm). Nominal depth shall not be less than two full units or 32 inches (813 mm), whichever is greater.

The nominal width of piers shall not be less than the nominal width of the beam, nor less than 8 inches (203 mm) or $^1/_{14}$ of the clear height between beam faces, whichever is greater.

The clear height-to-depth ratio of piers shall not exceed 5.

2108.2.6.1.3 Analysis. Member design forces shall be based on an analysis which considers the relative stiffness of pier and beam members, including the stiffening influence of joints.

The calculation of beam moment capacity for the determination of pier design shall include any contribution of floor slab reinforcement.

The out-of-plane drift ratio of all piers shall satisfy the drift-ratio limits specified in Section 1630.10.2.

2108.2.6.2 Design procedure.

2108.2.6.2.1 Required strength. Except as required by Sections 2108.2.6.2.7 and 2108.2.6.2.8, the required strength shall be determined in accordance with Section 2108.1.3.

2108.2.6.2.2 Design strength. Design strength provided by frame member cross sections in terms of axial force, shear and moment shall be computed as the nominal strength multiplied by the applicable strength-reduction factor, ϕ, specified in Section 2108.1.4.4.

Members shall be proportioned such that the design strength exceeds the required strength.

2108.2.6.2.3 Design assumptions for nominal strength. The nominal strength of member cross sections shall be based on assumptions prescribed in Section 2108.2.1.2.

The value of f'_m shall not be less than 1,500 psi (10.3 MPa) or greater than 4,000 psi (27.6 MPa).

2108.2.6.2.4 Reinforcement. The nominal moment strength at any section along a member shall not be less than one fourth of the higher moment strength provided at the two ends of the member.

Lap splices shall be as defined in Section 2108.2.2.7. The center of the lap splice shall be at the center of the member clear length.

Welded splices and mechanical connections conforming to Section 1912.14.3, Items 1 through 4, may be used for splicing the reinforcement at any section provided not more than alternate longitudinal bars are spliced at a section, and the distance between splices of alternate bars is at least 24 inches (610 mm) along the longitudinal axis.

Reinforcement shall not have a specified yield strength greater than 60,000 psi (413 MPa). The actual yield strength based on mill tests shall not exceed the specified yield strength times 1.3.

2108.2.6.2.5 Flexural members (beams). Requirements of this section apply to beams proportioned primarily to resist flexure as follows:

The axial compressive force on beams due to factored loads shall not exceed $0.10\, A_n\, f'_m$.

1. **Longitudinal reinforcement.** At any section of a beam, each masonry unit through the beam depth shall contain longitudinal reinforcement.

The variation in the longitudinal reinforcement area between units at any section shall not be greater than 50 percent, except multiple No. 4 bars shall not be greater than 100 percent of the minimum area of longitudinal reinforcement contained by any one unit, except where splices occur.

Minimum reinforcement ratio calculated over the gross cross section shall be 0.002.

Maximum reinforcement ratio calculated over the gross cross section shall be $0.15 f'_m / f_y$.

2. **Transverse reinforcement.** Transverse reinforcement shall be hooked around top and bottom longitudinal bars with a standard 180-degree hook, as defined in Section 2108.2.2.4, and shall be single pieces.

Within an end region extending one beam depth from pier faces and at any region at which beam flexural yielding may occur during seismic or wind loading, maximum spacing of transverse reinforcement shall not exceed one fourth the nominal depth of the beam.

The maximum spacing of transverse reinforcement shall not exceed one half the nominal depth of the beam.

Minimum reinforcement ratio shall be 0.0015.

The first transverse bar shall not be more than 4 inches (102 mm) from the face of the pier.

2108.2.6.2.6 Members subjected to axial force and flexure.

2108.2.6.2.6.1 The requirements set forth in this subsection apply to piers proportioned to resist flexure in conjunction with axial loads. CA

1. **Longitudinal reinforcement.** A minimum of four longitudinal bars shall be provided at all sections of every pier.

Flexural reinforcement shall be distributed across the member depth. Variation in reinforcement area between reinforced cells shall not exceed 50 percent.

Minimum reinforcement ratio calculated over the gross cross section shall be 0.002.

Maximum reinforcement ratio calculated over the gross cross section shall be $0.15 f'_m / f_y$.

Maximum bar diameter shall be one eighth nominal width of the pier.

2. **Transverse reinforcement.** Transverse reinforcement shall be hooked around the extreme longitudinal bars with standard 180-degree hook as defined in Section 2108.2.2.4.

Within an end region extending one pier depth from the end of the beam, and at any region at which flexural yielding may occur during seismic or wind loading, the maximum spacing of transverse reinforcement shall not exceed one fourth the nominal depth of the pier.

The maximum spacing of transverse reinforcement shall not exceed one half the nominal depth of the pier.

The minimum transverse reinforcement ratio shall be 0.0015.

3. **Lateral reinforcement.** Lateral reinforcement shall be provided to confine the grouted core when compressive strains due to axial and bending forces exceed 0.0015, corresponding to factored forces with R_w equal to 1.5. The unconfined portion of the cross section with strain exceeding 0.0015 shall be neglected in computing the nominal strength of the section.

The total cross-sectional area of rectangular tie reinforcement for the confined core shall not be less than:

$$A_{sh} = 0.09sh_c\, f'_m \,/\, f_{yh} \qquad (8\text{-}40)$$

Alternatively, equivalent confinement which can develop an ultimate compressive strain of at least 0.006 may be substituted for rectangular tie reinforcement.

CA *2108.2.6.2.6.2 [For BSC]* The requirements set forth in this subsection apply to piers proportioned to resist flexure in conjunction with axial loads.

1. **Longitudinal reinforcement.** A minimum of four longitudinal bars shall be provided at all sections of every pier.

Flexural reinforcement shall be distributed across the member depth. Variation in reinforcement area between reinforced cells shall not exceed 50 percent.

Minimum reinforcement ratio calculated over the gross cross section shall be 0.002.

Maximum reinforcement ratio calculated over the gross cross section shall be $0.15f'_m \,/\, f_y$.

Maximum bar diameter shall be one eighth nominal width of the pier.

2. **Transverse reinforcement.** Transverse reinforcement shall be hooked around the extreme longitudinal bars with standard 180-degree hook as defined in Section 2108.2.2.4.

Within an end region extending one pier depth from the end of the beam, and at any region at which flexural yielding may occur during seismic or wind loading, the maximum spacing of transverse reinforcement shall not exceed one fourth the nominal depth of the pier.

The maximum spacing of transverse reinforcement shall not exceed one half the nominal depth of the pier.

The minimum transverse reinforcement ratio shall be 0.0015.

3. **Lateral reinforcement.** Lateral reinforcement shall be provided to confine the grouted core when compressive strains due to axial and bending forces exceed 0.0015, corresponding to factored forces with R equal to 1.0. The unconfined portion of the cross section with strain exceeding 0.0015 shall be neglected in computing the nominal strength of the section.

The total cross-sectional area of rectangular tie reinforcement for the confined core shall not be less than:

$$A_{sh} = 0.09sh_c\, f'_m \,/\, f_{yh} \qquad (8\text{-}40)$$

Alternatively, equivalent confinement which can develop an ultimate compressive strain of at least 0.006 may be substituted for rectangular tie reinforcement.

2108.2.6.2.7 Pier design forces. Pier nominal moment strength shall not be less than 1.6 times the pier moment corresponding to the development of beam plastic hinges, except at the foundation level.

Pier axial load based on the development of beam plastic hinges in accordance with the paragraph above and including factored dead and live loads shall not exceed $0.15\, A_n\, f'_m$.

The drift ratio of piers shall satisfy the limits specified in Chapter 16.

The effects of cracking on member stiffness shall be considered.

The base plastic hinge of the pier must form immediately adjacent to the level of lateral support provided at the base or foundation.

2108.2.6.2.8 Shear design.

1. **General.** Beam and pier nominal shear strength shall not be less than 1.4 times the shears corresponding to the development of beam flexural yielding.

It shall be assumed in the calculation of member shear force that moments of opposite sign act at the joint faces and that the member is loaded with the tributary gravity load along its span.

2. **Vertical member shear strength.** The nominal shear strength shall be determined from Formula (8-41):

$$V_n = V_m + V_s \qquad (8\text{-}41)$$

WHERE:

$$V_m = C_d A_{mv} \sqrt{f'_m} \qquad (8\text{-}42)$$

For **SI:** $V_m = 0.083\, C_d A_{mv} \sqrt{f'_m}$

and

$$V_s = A_{mv}\, \rho_n\, f_y \qquad (8\text{-}43)$$

The value of V_m shall be zero within an end region extending one pier depth from beam faces and at any region where pier flexural yielding may occur during seismic loading, and at piers subjected to net tension factored loads.

The nominal pier shear strength, V_n, shall not exceed the value determined from Table 21-J.

3. **Beam shear strength.** The nominal shear strength shall be determined from Formula (8-44),

WHERE:

$$V_m = 1.2\, A_{mv} \sqrt{f'_m} \qquad (8\text{-}44)$$

For **SI:** $V_m = 0.01\, A_{mv} \sqrt{f'_m}$

The value of V_m shall be zero within an end region extending one beam depth from pier faces and at any region at which beam flexural yielding may occur during seismic loading.

The nominal beam shear strength, V_n, shall be determined from Formula (8-45).

$$V_n \le 4\, A_{mv} \sqrt{f'_m} \qquad (8\text{-}45)$$

For **SI:** $V_n \le 0.33\, A_{mv} \sqrt{f'_m}$

2108.2.6.2.9 Joints.

1. **General requirements.** Where reinforcing bars extend through a joint, the joint dimensions shall be proportioned such that

$$h_p > 4800\, d_{bb} / \sqrt{f'_g} \qquad (8\text{-}46)$$

For **SI:** $h_p > 400\, d_{bb} / \sqrt{f'_g}$

and

$$h_b > 1800\, d_{bp} / \sqrt{f'_g} \qquad (8\text{-}47)$$

For **SI:** $h_b > 150\, d_{bp} / \sqrt{f'_g}$

The grout strength shall not exceed 5,000 psi (34.4 MPa) for the purposes of Formulas (8-46) and (8-47).

Joint shear forces shall be calculated on the assumption that the stress in all flexural tension reinforcement of the beams at the pier faces is $1.4\, f_y$.

Strength of joint shall be governed by the appropriate strength-reduction factors specified in Section 2108.1.4.4.

Beam longitudinal reinforcement terminating in a pier shall be extended to the far face of the pier and anchored by a standard 90- or 180-degree hook, as defined in Section 2108.2.2.4, bent back to the beam.

Pier longitudinal reinforcement terminating in a beam shall be extended to the far face of the beam and anchored by a standard 90- or 180-degree hook, as defined in Section 2108.2.2.4, bent back to the beam.

2. **Transverse reinforcement.** Special horizontal joint shear reinforcement crossing a potential corner-to-corner diagonal joint shear crack, and anchored by standard hooks, as defined in Section 2108.2.2.4, around the extreme pier reinforcing bars shall be provided such that

$$A_{jh} = 0.5\ V_{jh} / f_y \qquad (8\text{-}48)$$

Vertical shear forces may be considered to be carried by a combination of masonry shear-resisting mechanisms and truss mechanisms involving intermediate pier reinforcing bars.

3. **Shear strength.** The nominal horizontal shear strength of the joint shall not exceed $7\sqrt{f'_m}$ (For **SI:** $0.58\sqrt{f'_m}$) or 350 psi (2.4 MPa), whichever is less.

SECTION 2109 — EMPIRICAL DESIGN OF MASONRY

2109.1 General. The design of masonry structures using empirical design located in those portions of Seismic Zones 0 and 1 as defined in Part III of Chapter 16 where the basic wind speed is less than 80 miles per hour as defined in Part II of Chapter 16 shall comply with the provisions of Section 2106 and this section, subject to approval of the building official.

2109.2 Height. Buildings relying on masonry walls for lateral load resistance shall not exceed 35 feet (10 668 mm) in height.

2109.3 Lateral Stability. Where the structure depends on masonry walls for lateral stability, shear walls shall be provided parallel to the direction of the lateral forces resisted.

Minimum nominal thickness of masonry shear walls shall be 8 inches (203 mm).

In each direction in which shear walls are required for lateral stability, the minimum cumulative length of shear walls provided shall be 0.4 times the long dimension of the building. The cumulative length of shear walls shall not include openings.

The maximum spacing of shear walls shall not exceed the ratio listed in Table 21-L.

2109.4 Compressive Stresses.

2109.4.1 General. Compressive stresses in masonry due to vertical dead loads plus live loads, excluding wind or seismic loads, shall be determined in accordance with Section 2109.4.3. Dead and live loads shall be in accordance with this code with permitted live load reductions.

2109.4.2 Allowable stresses. The compressive stresses in masonry shall not exceed the values set forth in Table 21-M. The allowable stresses given in Table 21-M for the weakest combination of the units and mortar used in any load wythe shall be used for all loaded wythes of multiwythe walls.

2109.4.3 Stress calculations. Stresses shall be calculated based on specified rather than nominal dimensions. Calculated compressive stresses shall be determined by dividing the design load by the gross cross-sectional area of the member. The area of openings, chases or recesses in walls shall not be included in the gross cross-sectional area of the wall.

2109.4.4 Anchor bolts. Bolt values shall not exceed those set forth in Table 21-N.

2109.5 Lateral Support. Masonry walls shall be laterally supported in either the horizontal or vertical direction not exceeding the intervals set forth in Table 21-O.

Lateral support shall be provided by cross walls, pilasters, buttresses or structural framing members horizontally or by floors, roof or structural framing members vertically.

Except for parapet walls, the ratio of height to nominal thickness for cantilever walls shall not exceed 6 for solid masonry or 4 for hollow masonry.

In computing the ratio for cavity walls, the value of thickness shall be the sums of the nominal thickness of the inner and outer wythes of the masonry. In walls composed of different classes of units and mortars, the ratio of height or length to thickness shall not exceed that allowed for the weakest of the combinations of units and mortar of which the member is composed.

2109.6 Minimum Thickness.

2109.6.1 General. The nominal thickness of masonry bearing walls in buildings more than one story in height shall not be less than 8 inches (203 mm). Solid masonry walls in one-story buildings may be of 6-inch nominal thickness when not over 9 feet (2743 mm) in height, provided that when gable construction is used, an additional 6 feet (1829 mm) is permitted to the peak of the gable.

> **EXCEPTION:** The thickness of unreinforced grouted brick masonry walls may be 2 inches (51 mm) less than required by this section, but in no case less than 6 inches (152 mm).

2109.6.2 Variation in thickness. Where a change in thickness due to minimum thickness occurs between floor levels, the greater thickness shall be carried up to the higher floor level.

2109.6.3 Decrease in thickness. Where walls of masonry of hollow units or masonry-bonded hollow walls are decreased in thickness, a course or courses of solid masonry shall be constructed between the walls below and the thinner wall above, or special units or construction shall be used to transmit the loads from face shells or wythes to the walls below.

2109.6.4 Parapets. Parapet walls shall be at least 8 inches (203 mm) in thickness and their height shall not exceed three times their thickness. The parapet wall shall not be thinner than the wall below.

2109.6.5 Foundation walls. Mortar used in masonry foundation walls shall be either Type M or S.

Where the height of unbalanced fill (height of finished grade above basement floor or inside grade) and the height of the wall between lateral support does not exceed 8 feet (2438 mm), and when the equivalent fluid weight of unbalanced fill does not exceed 30 pounds per cubic foot (480 kg/m^2), the minimum thickness of foundation walls shall be as set forth in Table 21-P. Maximum depths of unbalanced fill permitted in Table 21-P may be increased with the approval of the building official when local soil conditions warrant such an increase.

Where the height of unbalanced fill, height between lateral supports or equivalent fluid weight of unbalanced fill exceeds that set forth above, foundation walls shall be designed in accordance with Chapter 18.

2109.7 Bond.

2109.7.1 General. The facing and backing of multiwythe masonry walls shall be bonded in accordance with this section.

2109.7.2 Masonry headers. Where the facing and backing of solid masonry construction are bonded by masonry headers, not less than 4 percent of the wall surface of each face shall be composed of headers extending not less than 3 inches (76 mm) into the backing. The distance between adjacent full-length headers shall not exceed 24 inches (610 mm) either vertically or horizontally. In walls in which a single header does not extend through the wall, headers from opposite sides shall overlap at least 3 inches (76 mm), or headers from opposite sides shall be covered with another header course overlapping the header below at least 3 inches (76 mm).

Where two or more hollow units are used to make up the thickness of the wall, the stretcher courses shall be bonded at vertical intervals not exceeding 34 inches (864 mm) by lapping at least 3 inches (76 mm) over the unit below, or by lapping at vertical intervals not exceeding 17 inches (432 mm) with units which are at least 50 percent greater in thickness than the units below.

2109.7.3 Wall ties. Where the facing and backing of masonry walls are bonded with $^3/_{16}$-inch-diameter (4.8 mm) wall ties or metal ties of equivalent stiffness embedded in the horizontal mortar joints, there shall be at least one metal tie for each $4^1/_2$ square feet (0.42 m^2) of wall area. Ties in alternate courses shall be staggered, the maximum vertical distance between ties shall not exceed 24 inches (610 mm), and the maximum horizontal distance shall not exceed 36 inches (914 mm). Rods bent to rectangular shape shall be used with hollow-masonry units laid with the cells vertical. In other walls, the ends of ties shall be bent to 90-degree angles to provide hooks not less than 2 inches (51 mm) long. Additional ties shall be provided at all openings, spaced not more than 3 feet (914 mm) apart around the perimeter and within 12 inches (305 mm) of the opening.

The facing and backing of masonry walls may be bonded with prefabricated joint reinforcement. There shall be at least one cross wire serving as a tie for each $2^2/_3$ square feet (0.25 m^2) of wall area. The vertical spacing of the joint reinforcement shall not exceed 16 inches (406 mm). Cross wires of prefabricated joint reinforcement shall be at least No. 9 gage wire. The longitudinal wire shall be embedded in mortar.

2109.7.4 Longitudinal bond. In each wythe of masonry, head joints in successive courses shall be offset at least one fourth of the

(Text continues on page 2-227.)

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1. Compressive stress in the masonry:

$$f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right) \tag{7A-31}$$

2. Tensile stress in the longitudinal reinforcement:

$$f_s = \frac{M}{A_s jd} \tag{7A-32}$$

3. Design coefficients:

$$k = \sqrt{(n\rho)^2 + 2n\rho} - n\rho \tag{7A-33}$$

or

$$k = \frac{1}{1 + \frac{f_s}{nf_b}} \tag{7A-34}$$

$$j = 1 - \frac{k}{3} \tag{7A-35}$$

2107A.2.16 Bond of flexural reinforcement. In flexural members in which tensile reinforcement is parallel to the compressive face, the bond stress shall be computed by the formula:

$$u = \frac{V}{\Sigma_o jd} \tag{7A-36}$$

2107A.2.17 Shear in flexural members and shear walls. The shear stress in flexural members and shear walls shall be computed by:

$$f_v = \frac{V}{bjd} \tag{7A-37}$$

For members of T or I section, b' shall be substituted for b. Where f_v as computed by Formula (7A-37) exceeds the allowable shear stress in masonry, F_v, web reinforcement shall be provided and designed to carry the total shear force. Both vertical and horizontal shear stresses shall be considered.

The area required for shear reinforcement placed perpendicular to the longitudinal reinforcement shall be computed by:

$$A_v = \frac{sV}{F_s d} \tag{7A-38}$$

Where web reinforcement is required, it shall be so spaced that every 45-degree line extending from a point at $d/2$ of the beam to the longitudinal tension bars shall be crossed by at least one line of web reinforcement.

2107A.3 ***Not adopted by the State of California.***

SECTION 2108A — STRENGTH DESIGN OF MASONRY

2108A.1 General.

2108A.1.1 General provisions. The design of hollow-unit clay and concrete masonry structures using strength design shall comply with the provisions of Section 2106A and this section.

> **EXCEPTION:** Two-wythe solid-unit masonry may be used under Sections 2108A.2.1 and 2108A.2.4.

2108A.1.2 Quality assurance provisions. Special inspection during construction shall be provided as set forth in Section 1701A.5, Item 7.

2108A.1.3 Required strength. The required strength shall be determined in accordance with the factored load combinations of Section 1612A.2.

2108A.1.4 Design strength. Design strength is the nominal strength, multiplied by the strength-reduction factor, ϕ, as specified in this section. Masonry members shall be proportioned such that the design strength exceeds the required strength.

2108A.1.4.1 Beams, piers and columns.

2108A.1.4.1.1 Flexure. Flexure with or without axial load, the value of ϕ shall be determined from Formula (8A-1):

$$\phi = 0.8 - \frac{P_u}{A_e f'_m} \tag{8A-1}$$

and $0.60 \leq \phi \leq 0.80$

2108A.1.4.1.2 Shear. Shear: $\phi = 0.60$

2108A.1.4.2 Wall design for out-of-plane loads.

2108A.1.4.2.1 Walls with unfactored axial load of 0.04 f'_m or less. Flexure: $\phi = 0.80$.

2108A.1.4.2.2 Walls with unfactored axial load greater than 0.04 f'_m. Axial load and axial load with flexure: $\phi = 0.80$. Shear: $\phi = 0.60$.

2108A.1.4.3 Wall design for in-plane loads.

2108A.1.4.3.1 Axial load. Axial load and axial load with flexure: $\phi = 0.65$.

For walls with symmetrical reinforcement in which f_y does not exceed 60,000 psi (413 MPa), the value of ϕ may be increased linearly to 0.85 as the value of ϕP_n decreases from $0.10 f'_m A_e$ or $0.25 P_b$ to zero.

For solid grouted walls, the value of P_b may be calculated by Formula (8A-2)

$$P_b = 0.85 f'_m ba_b \tag{8A-2}$$

WHERE:

$$a_b = 0.85d \{e_{mu} / [e_{mu} + (f_y / E_s)]\} \tag{8A-3}$$

2108A.1.4.3.2 Shear. Shear: $\phi = 0.60$.

The value of ϕ may be 0.80 for any shear wall when its nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength for the factored-load combination.

2108A.1.4.4 Moment-resisting wall frames.

2108A.1.4.4.1 Flexure with or without axial load. The value of ϕ shall be as determined from Formula (8A-4); however, the value of ϕ shall not be less than 0.65 nor greater than 0.85.

$$\phi = 0.85 - 2\left(\frac{P_u}{A_e f'_m}\right) \tag{8A-4}$$

2108A.1.4.4.2 Shear. Shear: $\phi = 0.80$.

2108A.1.4.5 Anchor. Anchor bolts: $\phi = 0.80$.

2108A.1.4.6 Reinforcement.

2108A.1.4.6.1 Development. Development: $\phi = 0.80$.

2108A.1.4.6.2 Splices. Splices: $\phi = 0.80$.

2108A.1.5 Anchor bolts.

2108A.1.5.1 Required strength. The required strength of embedded anchor bolts shall be determined from factored loads as specified in Section 2108A.1.3.

2108A.1.5.2 Nominal anchor bolt strength. The nominal strength of anchor bolts times the strength-reduction factor shall equal or exceed the required strength.

The nominal tensile capacity of anchor bolts shall be determined from the lesser of Formula (8A-5) or (8A-6).

$$B_{tn} = 1.0A_p \sqrt{f'_m} \quad (8A\text{-}5)$$

For **SI:** $$B_{tn} = 0.084A_p \sqrt{f'_m}$$

$$B_{tn} = 0.4A_b f_y \quad (8A\text{-}6)$$

The area A_p shall be the lesser of Formula (8A-7) or (8A-8) and where the projected areas of adjacent anchor bolts overlap, the value of A_p of each anchor bolt shall be reduced by one half of the overlapping area.

$$A_p = \pi\, l_b^2 \quad (8A\text{-}7)$$

$$A_p = \pi\, l_{be}^2 \quad (8A\text{-}8)$$

The nominal shear capacity of anchor bolts shall be determined from the lesser of Formula (8A-9) or (8A-10).

$$B_{sn} = 900 \sqrt[4]{f'_m A_b} \quad (8A\text{-}9)$$

For **SI:** $$B_{sn} = 2750 \sqrt[4]{f'_m A_b}$$

$$B_{sn} = 0.25A_b f_y \quad (8A\text{-}10)$$

Where the anchor bolt edge distance, l_{be}, in the direction of load is less than 12 bolt diameters, the value of B_{tn} in Formula (8A-9) shall be reduced by linear interpolation to zero at an l_{be} distance
CA *equal to the greater of four diameters or* $1^1/_2$ inches (38 mm).

Anchor bolts subjected to combined shear and tension shall be designed in accordance with Formula (8A-11).

$$\frac{b_{tu}}{\phi B_{tn}} + \frac{b_{su}}{\phi B_{sn}} \le 1.0 \quad (8A\text{-}11)$$

2108A.1.5.3 Anchor bolt placement. Anchor bolts shall be placed so as to meet the edge distance, embedment depth and spacing requirements of Sections 2106A.2.14.2, 2106A.2.14.3 and 2106A.2.14.4.

2108A.2 Reinforced Masonry.

2108A.2.1 General.

2108A.2.1.1 Scope. The requirements of this section are in addition to the requirements of Sections 2106A and 2108A.1 and govern masonry in which reinforcement is used to resist forces.

2108A.2.1.2 Design assumptions. The following assumptions apply:

Masonry carries no tensile stress greater than the modulus of rupture.

Reinforcement is completely surrounded by and bonded to masonry material so that they work together as a homogeneous material.

Nominal strength of singly reinforced masonry wall cross sections for combined flexure and axial load shall be based on applicable conditions of equilibrium and compatibility of strains. Strain in reinforcement and masonry walls shall be assumed to be directly proportional to the distance from the neutral axis.

Maximum usable strain, e_{mu}, at the extreme masonry compression fiber shall:

1. Be 0.003 for the design of beams, piers, columns and walls.

2. Not exceed 0.003 for moment-resisting wall frames, unless lateral reinforcement as defined in Section 2108A.2.6.2.6 is utilized.

Strain in reinforcement and masonry shall be assumed to be directly proportional to the distance from the neutral axis.

Stress in reinforcement below specified yield strength f_y for grade of reinforcement used shall be taken as E_s times steel strain. For strains greater than that corresponding to f_y, stress in reinforcement shall be considered independent of strain and equal to f_y.

Tensile strength of masonry walls shall be neglected in flexural calculations of strength, except when computing requirements for deflection.

Relationship between masonry compressive stress and masonry strain may be assumed to be rectangular as defined by the following:

Masonry stress of $0.85 f'_m$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = 0.85c$ from the fiber of maximum compressive strain. Distance c from fiber of maximum strain to the neutral axis shall be measured in a direction perpendicular to that axis.

2108A.2.2 Reinforcement requirements and details.

2108A.2.2.1 Maximum reinforcement. The maximum size of reinforcement shall be No. 9. The diameter of a bar shall not exceed one fourth the least dimension of a cell. No more than two bars shall be placed in a cell of a wall or a wall frame.

2108A.2.2.2 Placement. The placement of reinforcement shall comply with the following:

In columns and piers, the clear distance between vertical reinforcing bars shall not be less than one and one-half times the nominal bar diameter, nor less than $1^1/_2$ inches (38 mm).

2108A.2.2.3 Cover. All reinforcing bars shall be completely embedded in mortar or grout and shall have a cover of not less than $1^1/_2$ inches (38 mm) nor less than 2.5 d_b.

2108A.2.2.4 Standard hooks. A standard hook shall be one of the following:

1. A 180-degree turn plus an extension of at least four bar diameters, but not less than $2^1/_2$ inches (63 mm) at the free end of the bar.

2. A 135-degree turn plus an extension of at least six bar diameters at the free end of the bar.

3. A 90-degree turn plus an extension of at least 12 bar diameters at the free end of the bar.

2108A.2.2.5 Minimum bend diameter for reinforcing bars. Diameter of bend measured on the inside of a bar other than for stirrups and ties in sizes No. 3 through No. 5 shall not be less than the values in Table 21A-G.

Inside diameter of bends for stirrups and ties shall not be less than $4d_b$ for No. 5 bars and smaller. For bars larger than No. 5, diameter of bend shall be in accordance with Table 21A-G.

2108A.2.2.6 Development. The calculated tension or compression reinforcement shall be developed in accordance with the following provisions:

The embedment length of reinforcement shall be determined by Formula (8A-12).

$$l_d = l_{de} / \phi \quad (8A\text{-}12)$$

WHERE:

$$l_{de} = \frac{0.15 d_b^2 f_y}{K\sqrt{f'_m}} \le 52 d_b \quad (8A\text{-}13)$$ CA

For **SI:** $$l_{de} = \frac{1.8 d_b^2 f_y}{K\sqrt{f'_m}} \le 52 d_b$$ CA

K shall not exceed $3d_b$.

ing seismic or wind loading, maximum spacing of transverse reinforcement shall not exceed one fourth the nominal depth of the beam.

The maximum spacing of transverse reinforcement shall not exceed one half the nominal depth of the beam.

Minimum reinforcement ratio shall be 0.0015.

The first transverse bar shall not be more than 4 inches (102 mm) from the face of the pier.

2108A.2.6.2.6 Members subjected to axial force and flexure.

The requirements set forth in this subsection apply to piers proportioned to resist flexure in conjunction with axial loads.

1. **Longitudinal reinforcement.** A minimum of four longitudinal bars shall be provided at all sections of every pier.

Flexural reinforcement shall be distributed across the member depth. Variation in reinforcement area between reinforced cells shall not exceed 50 percent.

Minimum reinforcement ratio calculated over the gross cross section shall be 0.002.

Maximum reinforcement ratio calculated over the gross cross section shall be $0.15f'_m / f_y$.

Maximum bar diameter shall be one eighth nominal width of the pier.

2. **Transverse reinforcement.** Transverse reinforcement shall be hooked around the extreme longitudinal bars with standard 180-degree hook as defined in Section 2108A.2.2.4.

Within an end region extending one pier depth from the end of the beam, and at any region at which flexural yielding may occur during seismic or wind loading, the maximum spacing of transverse reinforcement shall not exceed one fourth the nominal depth of the pier.

The maximum spacing of transverse reinforcement shall not exceed one half the nominal depth of the pier.

The minimum transverse reinforcement ratio shall be 0.0015.

3. **Lateral reinforcement.** Lateral reinforcement shall be provided to confine the grouted core when compressive strains due to axial and bending forces exceed 0.0015, corresponding to factored forces with R equal to *1.1*. The unconfined portion of the cross section with strain exceeding 0.0015 shall be neglected in computing the nominal strength of the section.

The total cross-sectional area of rectangular tie reinforcement for the confined core shall not be less than:

$$A_{sh} = 0.09sh_c\, f'_m / f_{yh} \qquad \text{(8A-40)}$$

Alternatively, equivalent confinement which can develop an ultimate compressive strain of at least 0.006 may be substituted for rectangular tie reinforcement.

2108A.2.6.2.7 Pier design forces. Pier nominal moment strength shall not be less than 1.6 times the pier moment corresponding to the development of beam plastic hinges, except at the foundation level.

Pier axial load based on the development of beam plastic hinges in accordance with the paragraph above and including factored dead and live loads shall not exceed $0.15\, A_n\, f'_m$.

The drift ratio of piers shall satisfy the limits specified in Chapter 16A.

The effects of cracking on member stiffness shall be considered.

The base plastic hinge of the pier must form immediately adjacent to the level of lateral support provided at the base or foundation.

2108A.2.6.2.8 Shear design.

1. **General.** Beam and pier nominal shear strength shall not be less than 1.4 times the shears corresponding to the development of beam flexural yielding.

It shall be assumed in the calculation of member shear force that moments of opposite sign act at the joint faces and that the member is loaded with the tributary gravity load along its span.

2. **Vertical member shear strength.** The nominal shear strength shall be determined from Formula (8A-41):

$$V_n = V_m + V_s \qquad \text{(8A-41)}$$

WHERE:

$$V_m = C_d A_{mv} \sqrt{f'_m} \qquad \text{(8A-42)}$$

For **SI:** $V_m = 0.083\, C_d A_{mv} \sqrt{f'_m}$

and

$$V_s = A_{mv}\, \rho_n\, f_y \qquad \text{(8A-43)}$$

The value of V_m shall be zero within an end region extending one pier depth from beam faces and at any region where pier flexural yielding may occur during seismic loading, and at piers subjected to net tension factored loads.

The nominal pier shear strength, V_n, shall not exceed the value determined from Table 21A-J.

3. **Beam shear strength.** The nominal shear strength shall be determined from Formula (8A-44),

WHERE:

$$V_m = 1.2\, A_{mv} \sqrt{f'_m} \qquad \text{(8A-44)}$$

For **SI:** $V_m = 0.01\, A_{mv} \sqrt{f'_m}$

The value of V_m shall be zero within an end region extending one beam depth from pier faces and at any region at which beam flexural yielding may occur during seismic loading.

The nominal beam shear strength, V_n, shall be determined from Formula (8A-45).

$$V_n \leq 4\, A_{mv} \sqrt{f'_m} \qquad \text{(8A-45)}$$

For **SI:** $V_n \leq 0.33\, A_{mv} \sqrt{f'_m}$

2108A.2.6.2.9 Joints.

1. **General requirements.** Where reinforcing bars extend through a joint, the joint dimensions shall be proportioned such that

$$h_p > 4800\, d_{bb} / \sqrt{f'_g} \qquad \text{(8A-46)}$$

For **SI:** $h_p > 400\, d_{bb} / \sqrt{f'_g}$

and

$$h_b > 1800\, d_{bp} / \sqrt{f'_g} \qquad \text{(8A-47)}$$

For **SI:** $h_b > 150\, d_{bp} / \sqrt{f'_g}$

The grout strength shall not exceed 5,000 psi (34.4 MPa) for the purposes of Formulas (8A-46) and (8A-47).

Joint shear forces shall be calculated on the assumption that the stress in all flexural tension reinforcement of the beams at the pier faces is $1.4\, f_y$.

Strength of joint shall be governed by the appropriate strength-reduction factors specified in Section 2108A.1.4.4.

Beam longitudinal reinforcement terminating in a pier shall be extended to the far face of the pier and anchored by a standard 90- or 180-degree hook, as defined in Section 2108A.2.2.4, bent back to the beam.

Pier longitudinal reinforcement terminating in a beam shall be extended to the far face of the beam and anchored by a standard 90- or 180-degree hook, as defined in Section 2108A.2.2.4, bent back to the beam.

2. **Transverse reinforcement.** Special horizontal joint shear reinforcement crossing a potential corner-to-corner diagonal joint shear crack, and anchored by standard hooks, as defined in Section 2108A.2.2.4, around the extreme pier reinforcing bars shall be provided such that

$$A_{jh} = 0.5\ V_{jh} / f_y \qquad (8A\text{-}48)$$

Vertical shear forces may be considered to be carried by a combination of masonry shear-resisting mechanisms and truss mechanisms involving intermediate pier reinforcing bars.

3. **Shear strength.** The nominal horizontal shear strength of the joint shall not exceed $7\sqrt{f'_m}$ (For **SI:** $0.58\sqrt{f'_m}$) or 350 psi (2.4 MPa), whichever is less.

SECTION 2109A — EMPIRICAL DESIGN OF MASONRY *[NOT ADOPTED BY OSHPD]*

CA

2109A.1 General. The design of masonry structures using empirical design located in those portions of Seismic Zones 0 and 1 as defined in Part III of Chapter 16A where the basic wind speed is less than 80 miles per hour as defined in Part II of Chapter 16A shall comply with the provisions of Section 2106A and this section, subject to approval of the building official.

2109A.2 Height. Buildings relying on masonry walls for lateral load resistance shall not exceed 35 feet (10 668 mm) in height.

2109A.3 Lateral Stability. Where the structure depends on masonry walls for lateral stability, shear walls shall be provided parallel to the direction of the lateral forces resisted.

Minimum nominal thickness of masonry shear walls shall be 8 inches (203 mm).

In each direction in which shear walls are required for lateral stability, the minimum cumulative length of shear walls provided shall be 0.4 times the long dimension of the building. The cumulative length of shear walls shall not include openings.

The maximum spacing of shear walls shall not exceed the ratio listed in Table 21A-L.

2109A.4 Compressive Stresses.

2109A.4.1 General. Compressive stresses in masonry due to vertical dead loads plus live loads, excluding wind or seismic loads, shall be determined in accordance with Section 2109A.4.3. Dead and live loads shall be in accordance with this code with permitted live load reductions.

2109A.4.2 Allowable stresses. The compressive stresses in masonry shall not exceed the values set forth in Table 21A-M. The allowable stresses given in Table 21A-M for the weakest combination of the units and mortar used in any load wythe shall be used for all loaded wythes of multiwythe walls.

2109A.4.3 Stress calculations. Stresses shall be calculated based on specified rather than nominal dimensions. Calculated compressive stresses shall be determined by dividing the design load by the gross cross-sectional area of the member. The area of openings, chases or recesses in walls shall not be included in the gross cross-sectional area of the wall.

2109A.4.4 Anchor bolts. Bolt values shall not exceed those set forth in Table 21A-N.

2109A.5 Lateral Support. Masonry walls shall be laterally supported in either the horizontal or vertical direction not exceeding the intervals set forth in Table 21A-O.

Lateral support shall be provided by cross walls, pilasters, buttresses or structural framing members horizontally or by floors, roof or structural framing members vertically.

Except for parapet walls, the ratio of height to nominal thickness for cantilever walls shall not exceed 6 for solid masonry or 4 for hollow masonry.

In computing the ratio for cavity walls, the value of thickness shall be the sums of the nominal thickness of the inner and outer wythes of the masonry. In walls composed of different classes of units and mortars, the ratio of height or length to thickness shall not exceed that allowed for the weakest of the combinations of units and mortar of which the member is composed.

2109A.6 Minimum Thickness.

2109A.6.1 General. The nominal thickness of masonry bearing walls in buildings more than one story in height shall not be less than 8 inches (203 mm). Solid masonry walls in one-story buildings may be of 6-inch nominal thickness when not over 9 feet (2743 mm) in height, provided that when gable construction is used, an additional 6 feet (1829 mm) is permitted to the peak of the gable.

> **EXCEPTION:** The thickness of unreinforced grouted brick masonry walls may be 2 inches (51 mm) less than required by this section, but in no case less than 6 inches (152 mm).

2109A.6.2 Variation in thickness. Where a change in thickness due to minimum thickness occurs between floor levels, the greater thickness shall be carried up to the higher floor level.

2109A.6.3 Decrease in thickness. Where walls of masonry of hollow units or masonry-bonded hollow walls are decreased in thickness, a course or courses of solid masonry shall be constructed between the walls below and the thinner wall above, or special units or construction shall be used to transmit the loads from face shells or wythes to the walls below.

2109A.6.4 Parapets. Parapet walls shall be at least 8 inches (203 mm) in thickness and their height shall not exceed three times their thickness. The parapet wall shall not be thinner than the wall below.

2109A.6.5 Foundation walls. Mortar used in masonry foundation walls shall be either Type M or S.

Where the height of unbalanced fill (height of finished grade above basement floor or inside grade) and the height of the wall between lateral support does not exceed 8 feet (2438 mm), and when the equivalent fluid weight of unbalanced fill does not exceed 30 pounds per cubic foot (480 kg/m^2), the minimum thickness of foundation walls shall be as set forth in Table 21A-P. Maximum depths of unbalanced fill permitted in Table 21A-P may be increased with the approval of the building official when local soil conditions warrant such an increase.

Where the height of unbalanced fill, height between lateral supports or equivalent fluid weight of unbalanced fill exceeds that set forth above, foundation walls shall be designed in accordance with Chapter 18A.

2109A.7 Bond.

2109A.7.1 General. The facing and backing of multiwythe masonry walls shall be bonded in accordance with this section.

Chapter 22
STEEL
Division I—GENERAL

SECTION 2201 — SCOPE

The quality, testing and design of steel used structurally in buildings or structures shall conform to the requirements specified in this chapter.

SECTION 2202 — STANDARDS OF QUALITY

The standards listed below labeled a "UBC Standard" are also listed in Chapter 35, Part II, and are part of this code. The other standards listed below are recognized standards. (See Sections 3503 and 3504.)

2202.1 Material Standards.

UBC Standard 22-1, Material Specifications for Structural Steel

2202.2 Design Standards.

ANSI/ASCE 8, Specification for the Design of Cold-formed Stainless Steel Structural Members, American Society of Civil Engineers

2202.3 Connectors.

ASTM A 502, Structural Rivet Steel

SECTION 2203 — MATERIAL IDENTIFICATION

2203.1 General. Steel furnished for structural load-carrying purposes shall be properly identified for conformity to the ordered grade in accordance with approved national standards, the provisions of this chapter and the appropriate UBC standards. Steel which is not readily identifiable as to grade from marking and test records shall be tested to determine conformity to such standards.

2203.2 Structural Steel. Structural steel shall be identified by the mill in accordance with approved national standards. When such steel is furnished to a specified minimum yield point greater than 36,000 pounds per square inch (psi) (248 MPa), the American Society for Testing and Materials (ASTM) or other specification designation shall be so indicated.

The fabricator shall maintain identity of the material and shall maintain suitable procedures and records attesting that the specified grade has been furnished in conformity with the applicable standard. The fabricator's identification mark system shall be established and on record prior to fabrication.

When structural steel is furnished to a specified minimum yield point greater than 36,000 psi (248 MPa), the ASTM or other specification designation shall be included near the erection mark on each shipping assembly or important construction component over any shop coat of paint prior to shipment from the fabricator's plant. Pieces of such steel which are to be cut to smaller sizes shall, before cutting, be legibly marked with the fabricator's identification mark on each of the smaller-sized pieces to provide continuity of identification. When subject to fabrication operations, prior to assembling into members, which might obliterate paint marking, such as blast cleaning, galvanizing or heating for forming, such pieces of steel shall be marked by steel die stamping or by a substantial tag firmly attached.

Individual pieces of steel having a minimum specified yield point in excess of 36,000 psi (248 MPa), which are received by the fabricator in a tagged bundle or lift or which have only the top shape or plate in the bundle or lift marked by the mill shall be marked by the fabricator prior to use in accordance with the fabricator's established identification marking system.

2203.3 Cold-formed Carbon and Low-alloy Steel. Cold-formed carbon and low-alloy steel used for structural purposes shall be identified by the mill in accordance with approved national standards. When such steel is furnished to a specified minimum yield point greater than 33,000 psi (228 MPa), the fabricator shall indicate the ASTM or other specification designation, by painting, decal, tagging or other suitable means, on each lift or bundle of fabricated elements.

When cold-formed carbon and low-alloy steel used for structural purposes has a specified yield point equal to or greater than 33,000 psi (228 MPa), which was obtained through additional treatment, the resulting minimum yield point shall be identified in addition to the specification designation.

2203.4 Cold-formed Stainless Steel. Cold-formed stainless steel structural members designed in accordance with recognized standards shall be identified as to grade through mill test reports. (See reference to ANSI/ASCE 8 in Chapter 35.) A certification shall be furnished that the chemical and mechanical properties of the material supplied equals or exceeds that considered in the design. Each lift or bundle of fabricated elements shall be identified by painting, decal, tagging or other suitable means.

2203.5 Open-web Steel Joists. Open-web steel joists and similar fabricated light steel load-carrying members shall be identified in accordance with Division II as to type, size and manufacturer by tagging or other suitable means at the time of manufacture or fabrication. Such identification shall be maintained continuously to the point of their installation in a structure.

SECTION 2204 — DESIGN METHODS

Design shall be by one of the following methods.

2204.1 Load and Resistance Factor Design.

2204.1.1 Steel design based on load and resistance factor design methods shall resist the factored load combinations of Section 1612.2 in accordance with the applicable requirements of Section 2205. Seismic design of structures, where required, shall comply with Division IV for structures designed in accordance with Division II (LRFD). CA

2204.1.2 [For BSC, HCD 1 & HCD 2] *Steel design based on load and resistance factor design method shall resist the factored load combinations of Section 1612.2 in accordance with the applicable requirements of Section 2205.* ✳ ✳ ✳ CA CA CA CA

2204.2 Allowable Stress Design.

2204.2.1 Steel design based on allowable stress design methods shall resist the load combinations of Section 1612.3 in accordance with the applicable requirements of Section 2205. Seismic design of structures, where required, shall comply with Division V for structures designed in accordance with Division III (ASD). CA

2204.2.2 [For BSC, HCD 1 & HCD 2] *Steel design based on allowable stress design methods shall resist the factored load combinations of Section 1612.3 in accordance with the applicable requirements of Section 2205.* ✳ ✳ ✳ CA CA CA

SECTION 2205 — DESIGN AND CONSTRUCTION PROVISIONS

2205.1 General. The following design standards shall apply.

2205.2 Structural Steel Construction. The design, fabrication and erection of structural steel shall be in accordance with the requirements of Division II for Load and Resistance Factor Design or Division III for Allowable Stress Design.

2205.3 Seismic Design Provisions for Structural Steel. Steel structural elements that resist seismic forces shall, in addition to the requirements of Section 2205.2, be designed in accordance with Division IV or V.

2205.4 Cold-formed Steel Construction. The design of cold-formed carbon or low-alloy steel structural members shall be in accordance with the requirements of Division VI for Load and Resistance Factor Design or Division VII for Allowable Stress Design.

2205.5 Cold-formed Stainless Steel Construction. The design of cold-formed stainless steel structural members shall be in accordance with approved national standards (see Section 2202).

2205.6 Design Provisions for Stud Wall Systems. Cold-formed steel stud wall systems that serve as part of the lateral-force-resisting system shall, in addition to the requirements of Section 2205.4 or 2205.5, be designed and constructed in accordance with Division VIII.

2205.7 Open-web Steel Joists and Joist Girders. The design, manufacture and use of steel joist, K, LH, and KLH series and joist girders shall be in accordance with Division IX.

2205.8 Steel Storage Racks. Steel storage racks may be designed in accordance with the provisions of Division X, except that in Seismic Zones 3 and 4 wholesale and retail sales areas, the W used in the design of racks over 8 feet (2438 mm) in height shall be equal to the weight of the rack structure and contents with no reductions.

2205.9 Steel Cables. Structural applications of steel cables for buildings shall be in accordance with the provisions of Division XI.

2205.10 Welding. Welding procedures, welder qualification requirements and welding electrodes shall be in accordance with Division II, III, VI or VII and approved national standards.

2205.11 Bolts. The use of high-strength A 325 and A 490 bolts shall be in accordance with the requirements of Divisions II and III.

Anchor bolts shall be set accurately to the pattern and dimensions called for on the plans. The protrusion of the threaded ends through the connected material shall be sufficient to fully engage the threads of the nuts, but shall not be greater than the length of threads on the bolts. Base plate holes for anchor bolts may be oversized as follows:

Bolt Size, inches (mm)	Hole Size, inches (mm)
$^3/_4$ (19.1)	$^5/_{16}$ (7.9) oversized
$^7/_8$ (22.2)	$^5/_{16}$ (7.9) oversized
1 < 2 (25.4 < 50.8)	$^1/_2$ (12.7) oversized
> 2 (> 50.8)	1 (25.4) > bolt diameter

Division IV—SEISMIC PROVISIONS FOR STRUCTURAL STEEL BUILDINGS

NOTE: This Division shall not apply to applications regulated by the Building Standards Commission as referenced in Section 10.17.3 and applications regulated by the Department of Housing and Community Development as referenced in Sections 101.17.9 and 101.17.10. See Chapter 22B, Division IV.

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Based on Seismic Provisions for Structural Steel Buildings, of the American Institute of Steel Construction (June 15, 1992)

Section 2210 of this division contains the exceptions to the referenced specification. Section 2211 of this division, "Seismic Provisions for Structural Steel Buildings" is reproduced with permission of the publisher.

SECTION 2210 — AMENDMENTS

The American Institute of Steel Construction Specification transcribed in this division applies to the seismic design of structural steel members and is to be used in conjunction with Division II (LRFD). Where other codes, standards or specifications are referred to in this division, they are to be considered as only an indication of an acceptable method or material that can be used with the approval of the building official.

1. Part I, Sec. 1. Revise as follows:

1. SCOPE

These special seismic requirements are to be applied in conjunction with the building code and Chapter 22, Division II (AISC-LRFD) hereinafter referred to as the *Specification.* They are intended for the design and construction of structural steel members and connections in buildings for which the design forces resulting from earthquake motions have been determined on the basis of energy dissipation in the nonlinear range of response.

2. Part I, Sec. 2. Revise as follows:

2. REQUIREMENTS IN SEISMIC ZONES

2.1 Seismic Zone 0 and 1 and Zone 2 (I = 1.0).

Buildings in Seismic Zones 0 and 1 and buildings in Seismic Zone 2 having an importance factor equal to 1.0 shall be designed in accordance with solely the *Specification* or in accordance with the *Specification* and these provisions.

2.2 Seismic Zone 2 (I > 1.0).

Buildings in Seismic Zone 2 having an importance factor I greater than 1.0 shall be designed in accordance with the *Specification* as modified by the additional provisions of this section.

2.2.a. Steel used in seismic-resisting systems shall be limited by the provisions of Section 5.

2.2.b. Columns in seismic-resisting systems shall be designed in accordance with Section 6.

2.2.c. Ordinary Moment Frames (OMF) shall be designed in accordance with the provisions of Section 7.

2.2.d. Special Moment Frames (SMF) are required to conform only to the requirements of Sections 8.2, 8.7 and 8.8.

2.2.e. Braced framed systems shall conform to the requirements of Section 9 or 10 when used alone or in combination with the moment frames of the seismic-resisting system.

2.3 Seismic Zones 3 and 4.

Buildings in Seismic Zones 3 and 4 shall be designed in accordance with the *Specification* as modified by the additional provisions of this section.

2.3.a. Steel used in seismic-resisting systems shall be limited by the provisions of Section 5.

2.3.b. Columns in seismic-resisting systems shall be designed in accordance with Section 6.

2.3.c. Ordinary Moment Frames (OMF) shall be designed in accordance with the provisions of Section 7.

2.3.d. Special Moment Frames (SMF) shall be designed in accordance with the provisions of Section 8.

2.3.e. Braced framed systems shall conform to the requirements of Section 9 (CBF) or 10 (EBF) when used alone or in combination with the moment frames of the seismic-resisting system.

The use of K-bracing systems shall not be permitted as part of the seismic resisting system except as permitted by Section 9.5 (Low Buildings).

2.3.f. A quality assurance plan shall be submitted to the regulatory agency for the seismic-force-resisting system of the building.

3. Part I, Sec. 3. Revise as follows:

3. LOADS, LOAD COMBINATIONS AND NOMINAL STRENGTHS

3.1 Loads and Load Combinations

The required strength of the structure and its elements shall be determined from the appropriate critical combination of factored loads, as determined by the load combinations in accordance with Section 1612.2. Wherever load combinations 3-1 through 3-6 are used in these provisions, they shall be taken as combinations 12-1 through 12-6 in Section 1612.2.1 of this code.

Orthogonal earthquake effects shall be included in the analysis unless noted specifically otherwise in Chapter 16.

Where required by these provisions, an amplified horizontal earthquake load of Ω_o as defined in Section 1630.3.1 shall be applied in load combinations 3-7 through 3-8 below.

The additional load combinations using the amplified horizontal earthquake are:

$$1.2\ D + f_1 L + \Omega_o\ E_h$$

$$0.9\ D \pm \Omega_o\ E_h$$

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WHERE:

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

= 0.5 for other live loads.

Where the amplified load is required, orthogonal effects are not required to be included.

3.2 Nominal Strengths

The nominal strengths shall be as provided in the *Specification.*

4. Sec. 5. Modify the second sentence by adding A 913 at the end of the sentence.

5. Sec. 6.1. Revise as follows:

6.1 Column Strength

When $P_u/\phi P_n > 0.5$, columns in seismic resisting frames, in addition to complying with the *Specification,* shall be limited by the following requirements.

6.1.a. The required axial compression strength shall be determined from Load Combination 3-7.

6.1.b. The required axial tension strength shall be determined from Load Combination 3-8.

6.1.c. The axial load combinations 3-7 and 3-8 are not required to exceed either of the following:

1. The maximum loads transferred to the column, considering 1.25 times the design strength of the connecting beams or brace elements of the structure.

2. The limit as determined by the foundation capacity to resist overturning uplift.

6. Part I, Sec. 6.2.a. Delete.

7. Sec. 7.2.c.2. Revise as follows:

2. The connections have been demonstrated by cyclic tests to have adequate rotation capacity at a design story drift Δ_m, as defined in Section 1630.9.

8. Part I, Sec. 8. Revise as follows:

8.2.c. Connection Strength. Connection configurations utilizing welds or high-strength bolts shall demonstrate, by approved cyclic testing results or calculation, the ability to sustain inelastic rotation and to develop the strength criteria in Section 8.2.a considering the expected value of yield strength and strain hardening.

8.2.d. Delete.

Sec. 8.4.b. Add the following to the end of the section:

The outside wall width thickness ratio of rectangular tubes used for columns shall not exceed $110/\sqrt{F_y}$ (For **SI:** $0.65\sqrt{E/F_y}$), unless otherwise stiffened.

Sec. 8.7.b.1. Revise as follows:

1. The required column strength shall be determined as the lesser of:

a. The loads resulting from the application of Load Combination No. 12.5 in Section 1612.2.1 except $\Omega_o E$ shall be substituted for E, or

b. 125 percent of the frame design strength based on either beam or panel zone design strengths.

8.9 Add section as follows:

8.9 Moment Frame Drift Calculations.

Moment frame drift calculations shall include bending and shear contributions from the clear girder and column spans, column axial deformation and the rotation and distortion of the panel zone.

8.9.a. Drift calculations may be based on column and girder center lines where either of the following conditions is met:

1. Where it can be demonstrated that drift so computed for frames of similar configuration is typically within 15 percent of that determined above, or

2. The nominal panel zone strength is equal to or greater than 0.8 ΣM_p of girders framing into the column flanges at the connection.

8.9.b. Column axial deformations may be neglected if they contribute less than 10 percent to the total drift.

9. Part I, Sec. 10.5. Add the following to the end of the section:

Intermediate bracing shall be provided at the top and bottom flanges of the link at intervals not exceeding $76/\sqrt{F_y}$ times the beam flange width. Such intermediate bracing shall have a design strength of 1.0 percent of the link flange nominal strength computed as $F_y b_f t_f$.

10. Part I. Add new requirements for Special Concentrically Braced Frames Requirements for Special Concentrically Braced Frames (SCBF) in Section 12:

Special CBFs shall be designed in accordance with the requirements of Section 9 except as modified herein. The following modifications shall apply to SCBFs and shall not modify the requirements for ordinary CBFs in Section 9.

a. Sec. 9.2.a. Revise as follows:

Slenderness: Bracing members shall have an $L/r < 1{,}000/\sqrt{F_y}$ (For **SI:** $5.87\sqrt{E/F_y}$).

b. Sec. 9.2.b. Revise as follows:

9.2.b. Compressive Design Strength. The design strength of a bracing member in axial compression shall not exceed $\phi_c P_n$.

c. Sec. 9.2.d. Revise as follows:

9.2.d. Width-thickness Ratio. Width-thickness ratios of stiffened and unstiffened compression elements of braces shall comply with Section B5 of the *Specification.* Braces shall be compact (i.e., $\lambda < \lambda p$). The width-thickness ratio of angle sections shall not exceed $52/\sqrt{F_y}$. Circular sections shall have an outside diameter to wall thickness ratio not exceeding $1{,}300/\sqrt{F_y}$; rectangular tubes shall have an outside wall width-thickness ratio not exceeding $100/\sqrt{F_y}$ unless the circular section or tube walls are stiffened.

d. Sec. 9.2.e. Revise as follows:

9.2.e. Built-up Member Stitches. For all built-up braces, the spacing of stitches shall be uniform and not less than two stitches shall be used:

1. For a brace in which stitches can be subjected to postbuckling shear, the spacing of the stitches shall be such that the slenderness ratio, *L/r,* of individual elements between the stitches does not exceed 0.4 times the governing slenderness ratio of the built-up member. The total shear strength of the stitches shall be at least equal to the tensile strength of each element. Bolted stitches shall not be located within the middle one-fourth of the clear brace length.
2. For braces that can buckle without causing shear in the stitches, the spacing of the stitches shall be such that the slenderness ratio, *L/r,* of the individual elements between the stitches does not exceed 0.75 times the governing slenderness ratio of the built-up member.

e. Sec. 9.4.a. Revise as follows:

9.4.a. V and Inverted V Type Bracing. V braced and inverted V braced frames shall comply with the following:

1. A beam intersected by braces shall be continuous between columns.
2. A beam intersected by braces shall be capable of supporting all tributary dead and live loads assuming the bracing is not present.
3. A beam intersected by braces shall be capable of resisting the combination of load effects caused by the application of the load combinations in Section 2213.5.1, Items 1 and 2,

Division V—SEISMIC PROVISIONS FOR STRUCTURAL STEEL BUILDINGS FOR USE WITH ALLOWABLE STRESS DESIGN

NOTE: This Division shall not apply to applications regulated by the Building Standards Commission as referenced in Section 101.17.3 and applications regulated by the Department of Housing and Community Development as referenced in Sections 101.17.9 and 101.17.10. See Chapter 22B, Division V.

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SECTION 2212 — GENERAL

When the load combinations of Section 1612.3 for Allowable Stress Design are used, structural steel buildings shall be designed in accordance with the provisions of Chapter 22, Division III (AISC-ASD), and this division where applicable.

SECTION 2213 — SEISMIC PROVISIONS FOR STRUCTURAL STEEL BUILDINGS IN SEISMIC ZONES 3 AND 4

2213.1 General. Design and construction of steel framing in lateral-force-resisting systems in Seismic Zones 3 and 4 shall conform to the requirements of the code and to the requirements of this section.

2213.2 Definitions.

ALLOWABLE STRESSES are prescribed in Divisons III and VII.

CHEVRON BRACING is that form of bracing where a pair of braces located either above or below a beam terminates at a single point within the clear beam span.

CONNECTION is the group of elements that connect the member to the joint.

DIAGONAL BRACING is that form of bracing that diagonally connects joints at different levels.

ECCENTRICALLY BRACED FRAME (EBF) is a diagonal braced frame in which at least one end of each bracing member connects to a beam a short distance from a beam-to-column connection or from another beam-to-brace connection.

GIRDER is the horizontal member in a seismic frame. The words beam and girder may be used interchangeably.

JOINT is the entire assemblage at the intersections of the members.

K BRACING is that form of bracing where a pair of braces located on one side of a column terminates at a single point within the clear column height.

LINK BEAM is that part of a beam in an eccentrically braced frame which is designed to yield in shear and/or bending so that buckling of the bracing members is prevented.

STRENGTH is the strength as prescribed in Section 2213.4.2.

V BRACING is that form of chevron bracing that intersects a beam from above and inverted V bracing is that form of chevron bracing that intersects a beam from below.

X BRACING is that form of bracing where a pair of diagonal braces cross near midlength of the bracing members.

2213.3 Symbols and Notations. The symbols and notations unique to this section are as follows:

M_s = flexural strength.
P_{DL} = axial dead load.
P_E = axial load on member due to earthquake.
P_{LL} = axial live load.
P_{sc} = compressive axial strength of member.
P_{st} = tensile axial strength of member.
V_s = shear strength of member.
Z = plastic section modulus.

2213.4 Materials.

2213.4.1 Quality. Structural steel used in lateral-force-resisting systems shall conform to A 36, A 500, A 501, A 572 (Grades 42 and 50), A 913 (Grades 50 and 65) and A 588. Structural steel conforming to A 283 (Grade D) may be used for base plates and anchor bolts.

EXCEPTION: Other steels permitted in this code may be used for the following:

1. One-story buildings.
2. Light-framed wall systems in accordance with Division VIII.

2213.4.2 Member strength. Where this section requires the strength of the member to be developed, the following shall be used:

	Strength
Moment	$M_s = ZF_y$
Shear	$V_s = 0.55\ F_y dt$
Axial compression	$P_{sc} = 1.7\ F_a A$
Axial tension	$P_{st} = F_y A$
Connectors	
Full-penetration welds	$F_y A$
Partial penetration welds	$1.7\ F_s$
Bolts and fillet welds	$1.7\ F_s$

Where F_s is the allowable stress value defined in the applicable chapter of Division III. For the purpose of determining member or connection strengths, the allowable stress values specified in Division III shall not be increased by the one-third allowable stress increase per Section 1612.3.2.

Members need not be compact unless otherwise required by this section.

2213.5 Column Requirements.

2213.5.1 Column strength. Columns shall satisfy the load combinations required by Section 1612.2 at load and resistance factor limits or Section 1612.3 at allowable stress limits with stress increases allowed by Section 1612.3.2. In addition, in Seismic Zones 3 and 4, columns in frames shall have the strength to resist the axial loads resulting from the load combinations in Items 1 and 2.

1. **Axial compression**

$$1.0\ P_{DL} + 0.7\ P_{LL} + \Omega_o\ P_E$$

2. **Axial tension**

$$0.85\ P_{DL} \pm \Omega_o\ P_E$$

EXCEPTION: The axial load combination as outlined in Items 1 and 2 above:

1. Need not exceed either the maximum force that can be transferred to the column, by elements of the structure, or the limit as determined by the overturning uplift which the foundation is capable of resisting.

2. Need not apply to columns in moment-resisting frames complying with Formula (13-3-1) or (13-3-2) where f_a is equal to or less than $0.3\ F_y$ for all load combinations.

The load combinations from Items 1 and 2 need be used only when specifically referred to.

2213.5.2 Column splices. Column splices shall have sufficient strength to develop the column forces determined from Section 2213.5.1. Welded column splices subject to net tensile forces shall comply with the more critical of the following:

1. Partial penetration welds shall be designed to resist 150 percent of the force determined from Section 2213.5.1, Item 2.

2. Welding shall develop not less than 50 percent of the flange area strength of the smaller column.

Splices employing partial penetration welds shall be located at least three feet (914 mm) from girder flanges.

2213.5.3 Slenderness evaluation. This paragraph is applicable when the provisions are applied to the effective length determination of columns of moment frames resisting earthquake forces. In the plane of the earthquake forces the factor K may be taken as unity when all of the following conditions are met:

1. The column is either continuous or is fixed at each joint.

2. The maximum axial compressive stress, f_a, does not exceed $0.4\,F_y$ under design loads.

3. The calculated drift ratios are less than the values given in Section 1630.8.

2213.6 Ordinary Moment Frame Requirements. Ordinary moment frames (OMF) shall be designed to resist the load combinations in Section 1612.3.

All beam-to-column connections in OMFs which resist earthquake forces shall meet one of the following requirements:

1. Fully restrained (Type F.R. or Type 1) conforming with Section 2213.7.1.

2. Fully restrained (Type F.R. or Type 1) connections with the design strengths of the connections capable of resisting a combination of gravity loads and Ω_o times the design seismic forces.

3. Partially restrained (Type P.R. or Type 3) connections are permitted provided:

3.1 The connections are designed to resist the load combinations in Section 1612.2 or 1612.3, and

3.2 The connections have been demonstrated by cyclic tests to have adequate rotation capacity to accommodate a story drift due to Ω_o times the design seismic forces.

3.3 The moment frame drift calculations shall include the contribution due to the rotation and distortion of the connection.

See Divisions II and III for definitions of fully restrained and partially restrained connections.

2213.7 Special Moment-resisting Frame (SMRF) Requirements.

2213.7.1 Girder-to-column connection.

2213.7.1.1 Required strength. The girder-to-column connection shall be adequate to develop the lesser of the following:

1. The strength of the girder in flexure.

2. The moment corresponding to development of the panel zone shear strength as determined from Formula (13-1).

EXCEPTION: Where a connection is not designed to contribute flexural resistance at the joint, it need not develop the required strength if it can be shown to meet the deformation compatibility requirements of Section 1633.2.4.

2213.7.1.2 Connection strength. Connection configurations utilizing welds or high-strength bolts shall demonstrate, by approved cyclic test results or calculation, the ability to sustain inelastic rotation and develop the strength criteria in Section 2213.7.1.1 considering the effect of steel overstrength and strain hardening.

2213.7.1.3 Flange detail limitations. For steel whose specified ultimate strength is less than 1.5 times the specified yield strength, plastic hinges shall not form at locations in which the beam flange area has been reduced, such as for bolt holes. Bolted connections of flange plates of beam-column joints shall have the net-to-gross area ratio A_e/A_g equal to or greater than $1.2\,F_y/F_u$.

2213.7.2 Panel zone.

2213.7.2.1 Strength. The panel zone of the joint shall be capable of resisting the shear induced by beam bending moments due to gravity loads plus 1.85 times the prescribed seismic forces, but the shear strength need not exceed that required to develop $0.8\Sigma M_s$ of the girders framing into the column flanges at the joint. The joint panel zone shear strength may be obtained from the following formula:

$$V = 0.55\,F_y d_c t \left[1 + \frac{3 b_c t_{cf}^{\,2}}{d_b d_c t} \right] \qquad (13\text{-}1)$$

WHERE:

b_c = the width of the column flange.

d_b = the depth of the beam.

d_c = the column depth.

t = the total thickness of the joint panel zone including doubler plates.

t_{cf} = the thickness of the column flange.

2213.7.2.2 Thickness. The panel zone thickness, t_z, shall conform to the following formula:

$$t_z \geq (d_z + w_z)/90 \qquad (13\text{-}2)$$

WHERE:

d_z = the panel zone depth between continuity plates.

w_z = the panel zone width between column flanges.

For this purpose, t_z, shall not include any double plate thickness unless the doubler plate is connected to the column web with plug welds adequate to prevent local buckling of the plate.

2213.7.2.3 Doubler plates. Doubler plates provided to reduce panel zone shear stress or to reduce the web depth thickness ratio shall be placed not more than $^1/_{16}$ inch (1.6 mm) from the column web and shall be welded across the plate width top and bottom with at least a $^3/_{16}$-inch (4.7 mm) fillet weld. They shall be either butt or fillet welded to the column flanges to develop the shear strength of the doubler plate. Weld strength shall be as given in Section 2213.4.2.

2213.7.3 Width-thickness ratio. Girders shall comply with Division III, except that the flange width-thickness ratio, $b_f / 2t_f$, shall not exceed $52/\sqrt{F_y}$ (For **SI:** $0.31\sqrt{E/F_y}$). The width-thickness ratio of column sections shall meet the requirements of Division III, Section 2251N7. The outside wall width-thickness ratio of rectangular tubes used for columns shall not exceed $110/\sqrt{F_y}$ (For **SI:** $0.65\sqrt{E/F_y}$), unless otherwise stiffened.

2213.7.4 Continuity plates. When determining the need for girder tension flange continuity plates, the value of P_{bf} in Division III shall be taken as $1.8\,(bt_f)F_{yb}$.

2213.7.5 Strength ratio. At any moment frame joint, the following relationships shall be satisfied:

$$\Sigma Z_c\,(F_{yc} - f_a) / \Sigma M_c > 1.0 \qquad (13\text{-}3\text{-}1)$$

or

Division V—SEISMIC PROVISIONS FOR STRUCTURAL STEEL BUILDINGS FOR USE WITH ALLOWABLE STRESS DESIGN

SECTION 2212A — GENERAL

When the load combinations of Section 1612A.3 for Allowable Stress Design are used, structural steel buildings shall be designed in accordance with the provisions of Chapter 22A, Division III (AISC-ASD), and this division where applicable.

SECTION 2213A — SEISMIC PROVISIONS FOR STRUCTURAL STEEL BUILDINGS IN SEISMIC ZONES 3 AND 4

2213A.1 General. Design and construction of steel framing in lateral-force-resisting systems in Seismic Zones 3 and 4 shall conform to the requirements of the code and to the requirements of this section.

2213A.2 Definitions.

ALLOWABLE STRESSES are prescribed in Divisons III and VII.

CHEVRON BRACING is that form of bracing where a pair of braces located either above or below a beam terminates at a single point within the clear beam span.

CONNECTION is the group of elements that connect the member to the joint.

DIAGONAL BRACING is that form of bracing that diagonally connects joints at different levels.

ECCENTRICALLY BRACED FRAME (EBF) is a diagonal braced frame in which at least one end of each bracing member connects to a beam a short distance from a beam-to-column connection or from another beam-to-brace connection.

GIRDER is the horizontal member in a seismic frame. The words beam and girder may be used interchangeably.

JOINT is the entire assemblage at the intersections of the members.

K BRACING is that form of bracing where a pair of braces located on one side of a column terminates at a single point within the clear column height.

LINK BEAM is that part of a beam in an eccentrically braced frame which is designed to yield in shear and/or bending so that buckling of the bracing members is prevented.

STRENGTH is the strength as prescribed in Section 2213A.4.2.

V BRACING is that form of chevron bracing that intersects a beam from above and inverted V bracing is that form of chevron bracing that intersects a beam from below.

X BRACING is that form of bracing where a pair of diagonal braces cross near midlength of the bracing members.

2213A.3 Symbols and Notations. The symbols and notations unique to this section are as follows:

F_{yb} = *design strength including effects of strain hardening and overstrength for use in Formula (13A-3.1).*

M_s = flexural strength.

P_{DL} = axial dead load.

P_E = axial load on member due to earthquake.

P_{LL} = axial live load.

P_{sc} = compressive axial strength of member.

P_{st} = tensile axial strength of member.

V_s = shear strength of member.

Z = plastic section modulus.

2213A.4 Materials.

2213A.4.1 Quality. Structural steel used in lateral-force-resisting systems shall conform to A 36, A 500, A 501, *A 992,* A 572 (Grades 42 and 50), A 913 (Grades 50 and 65) and A 588. Structural steel conforming to A 283 (Grade D) may be used for base plates and anchor bolts.

> **EXCEPTION:** Other steels permitted in this code may be used for the following:
> 1. One-story buildings.
> 2. Light-framed wall systems in accordance with Division VIII.

All welds used in primary members and connections in the lateral force-resisting systems shall be made with a filler metal that has a minimum Charpy V-notch toughness of 20 ft-lbs at minus 20 degrees F, as determined by AWS Classification or manufacturer certification.

2213A.4.2 Member strength. Where this section requires the strength of the member to be developed, the following shall be used:

	Strength
Moment	$M_s = ZF_y$
Shear	$V_s = 0.55\ F_y dt$
Axial compression	$P_{sc} = 1.7\ F_a A$
Axial tension	$P_{st} = F_y A$
Connectors	
Full-penetration welds	$F_y A$
Partial penetration welds	$1.7\ F_s$
Bolts and fillet welds	$1.7\ F_s$

where F_s is the allowable stress value defined in the applicable chapter of Division III for member and connection strength. The allowable stress values specified in Division III shall not be increased by the one third allowable stress increase per Section 1612A.3.2.

Members need not be compact unless otherwise required by this section.

2213A.5 Column Requirements.

2213A.5.1 Column strength. Columns shall satisfy the load combinations required by Section 1612A.2 at load and resistance factor limits or Section 1612A.3 at allowable stress limits with stress increases allowed by Section 1612A.3.2. In addition, in Seismic Zones 3 and 4, columns in frames shall have the strength to resist the axial loads resulting from the load combinations in Items 1 and 2.

1. **Axial compression**

$$1.0\ P_{DL} + 0.7\ P_{LL} + \Omega_o\ P_E$$

2. **Axial tension**

$$0.85\ P_{DL} \pm \Omega_o\ P_E$$

> **EXCEPTION:** The axial load combination as outlined in Items 1 and 2 above:
>
> 1. Need not exceed either the maximum force that can be transferred to the column, by elements of the structure, or the limit as determined by the overturning uplift which the foundation is capable of resisting.
>
> 2. Need not apply to columns in moment-resisting frames complying with Formula (13A-3-1) or (13A-3-2) where f_a is equal to or less than 0.3 F_y for all load combinations.

The load combinations from Items 1 and 2 need be used only when specifically referred to.

2213A.5.2 Column splices. Column splices shall have sufficient strength to develop the column forces determined from Section 2213A.5.1. Welded column splices subject to net tensile forces shall comply with the more critical of the following:

1. Partial penetration welds shall be designed to resist 150 percent of the force determined from Section 2213A.5.1, Item 2.

2. Welding shall develop not less than 50 percent of the flange area strength of the smaller column.

Splices employing partial penetration welds shall be located at least three feet (914 mm) from girder flanges.

2213A.5.3 Slenderness evaluation. This paragraph is applicable when the provisions are applied to the effective length determination of columns of moment frames resisting earthquake forces. In the plane of the earthquake forces the factor K may be taken as unity when all of the following conditions are met:

1. The column is either continuous or is fixed at each joint.

2. The maximum axial compressive stress, f_a, does not exceed 0.4 F_y under design loads.

3. The calculated drift ratios are less than the values given in Section 1630A.8.

2213A.6 Ordinary Moment Frame Requirements. *[Not adopted by OSHPD]* Ordinary moment frames (OMF) shall be designed to resist the load combinations in Section 1612A.3.

All beam-to-column connections in OMFs which resist earthquake forces shall meet one of the following requirements:

1. Fully restrained (Type F.R. or Type 1) conforming with Section 2213A.7.1.

2. Fully restrained (Type F.R. or Type 1) connections with the design strengths of the connections capable of resisting a combination of gravity loads and Ω_o times the design seismic forces.

3. Partially restrained (Type P.R. or Type 3) connections are permitted provided:

3.1 The connections are designed to resist the load combinations in Section 1612A.2 or 1612A.3, and

3.2 The connections have been demonstrated by cyclic tests to have adequate rotation capacity to accommodate a story drift due to Ω_o times the design seismic forces.

3.3 The moment frame drift calculations shall include the contribution due to the rotation and distortion of the connection.

See Divisions I and III for definitions of fully restrained and partially restrained connections.

2213A.7 Special Moment-resisting Frame (SMRF) Requirements.

2213A.7.1 Girder-to-column connection.

2213A.7.1.1 Required strength. The girder-to-column connection shall be adequate to develop the lesser of the following:

1. The strength of the girder in flexure.

2. The moment corresponding to development of the panel zone shear strength *including strain hardening and overstrength* as determined from Formula (13A-1).

EXCEPTION: Where a connection is not designed to contribute flexural resistance at the joint, it need not develop the required strength if it can be shown to meet the deformation compatibility requirements of Section 1633A.2.4.

2213A.7.1.2 Connection strength. Connection configurations utilizing welds or high-strength bolts shall demonstrate, by approved cyclic test results or calculation, the ability to sustain inelastic rotation and develop the strength criteria in Section 2213A.7.1.1 considering the effect of steel overstrength and strain hardening.

Design of beam-to-column joints shall be substantiated by testing to have an inelastic rotation of at least 0.03 radians.

2213A.7.1.3 Flange detail limitations. For steel whose specified ultimate strength is less than 1.5 times the specified yield strength, plastic hinges shall not form at locations in which the beam flange area has been reduced, such as for bolt holes. Bolted connections of flange plates of beam-column joints shall have the net-to-gross area ratio A_e/A_g equal to or greater than 1.2 F_y/F_u.

2213A.7.2 Panel zone.

2213A.7.2.1 Strength. The panel zone of the joint shall be capable of resisting the shear induced by beam bending moments due to gravity loads plus 1.85 times the prescribed seismic forces, but the shear strength need not exceed that required to develop $0.8\Sigma M_s$ of the girders framing into the column flanges at the joint. The joint panel zone shear strength may be obtained from the following formula:

$$V = 0.55\, F_y d_c t \left[1 + \frac{3 b_c t_{cf}^{\,2}}{d_b d_c t} \right] \qquad \text{(13A-1)}$$

WHERE:

b_c = the width of the column flange.

d_b = the depth of the beam.

d_c = the column depth.

t = the total thickness of the joint panel zone including doubler plates.

t_{cf} = the thickness of the column flange.

2213A.7.2.2 Thickness. The panel zone thickness, t_z, shall conform to the following formula:

$$t_z \geq (d_z + w_z)/90 \qquad \text{(13A-2)}$$

WHERE:

d_z = the panel zone depth between continuity plates.

w_z = the panel zone width between column flanges.

For this purpose, t_z, shall not include any double plate thickness unless the doubler plate is connected to the column web with plug welds adequate to prevent local buckling of the plate.

2213A.7.2.3 Doubler plates. Doubler plates provided to reduce panel zone shear stress or to reduce the web depth thickness ratio shall be placed not more than $^1/_{16}$ inch (1.6 mm) from the column web and shall be welded across the plate width top and bottom with at least a $^3/_{16}$-inch (4.7 mm) fillet weld. They shall be either butt or fillet welded to the column flanges to develop the shear strength of the doubler plate. Weld strength shall be as given in Section 2213A.4.2.

2213A.7.3 Width-thickness ratio. Girders shall comply with Division III, except that the flange width-thickness ratio, $b_f / 2t_f$, shall not exceed $52/\sqrt{F_y}$ (For **SI:** $0.31\sqrt{E/F_y}$). The width-thickness ratio of column sections shall meet the requirements of Division III, Section *2208N7*. The outside wall width-thickness

|| CA *1627A.* All members and connections in special braced frames shall be designed and detailed to resist shear and flexure caused by eccentricities in the geometry of the members comprising the frame in accordance with Section 2213A.9. Any member intersected by a brace shall be continuous through the connection. Horizontal bracing that transfers forces between horizontally offset bracing in the vertical plane shall be subject to the requirements of Section 2213A.9, except Sections 2213A.9.2.3; 2213A.9.4.1, Item 3; and 2213A.9.4.2. Horizontal bracing other than the above is not subjected to the requirements of Section 2213A.9.

2213A.9.2 Bracing members.

2213A.9.2.1 Slenderness. The *kl/r* ratio for bracing members shall not exceed $1,000/\sqrt{F_y}$ (For **SI:** $5.87\sqrt{E/F_y}$), except as permitted in Section 2213A.9.6.

2213A.9.2.2 Lateral-force distribution. The seismic lateral force along any line of bracing shall be distributed to the various members so that neither the sum of the horizontal components of forces in members acting in compression or tension exceed 70 percent of the total force.

> **EXCEPTION:** Where compression bracing acting alone has the strength to resist Ω_o times the design seismic force, such distribution is not required.

A line of bracing is defined, for the purposes of this provision, as a single line or parallel lines within 10 percent of the dimension of the structure perpendicular to the line of bracing.

2213A.9.2.3 Built-up members. The spacing of stitches shall be such that the slenderness ratio (*l/r*) of individual elements between the stitches does not exceed 0.4 times the governing slenderness ratio of the built-up member. The total shear strength of the stitches shall be at least equal to the tensile strength of each element. The spacing of the stitches shall be uniform and not less than two stitches shall be used. Bolted stitches shall not be located within the middle one fourth of the clear brace length.

> **EXCEPTION:** Where it can be shown that braces can buckle without causing shear in the stitches, the spacing of the stitches shall be such that the slenderness ration (*l/r*) of the individual element between the stitches does not exceed 0.75 times the governing slenderness ratio of the built-up member.

2213A.9.2.4 Compression elements in braces. The width-thickness ratio of compression elements used in braces shall meet the requirements of Division III, Table B5.1, for compact sections. The width-thickness ratio of angle section shall be limited to $52/\sqrt{F_y}$ (For **SI:** $0.31\sqrt{E/F_y}$). Circular sections shall have outside diameter-wall thickness ratio not exceeding $1,300/F_y$ (For **SI:** $7.63\,E/F_y$), rectangular tubes shall have outside wall width-thickness ratio not exceeding $110/\sqrt{F_y}$ (For **SI:** $0.65\sqrt{E/F_y}$).

> **EXCEPTION:** Compression elements stiffened to resist local buckling.

2213A.9.3 Bracing connections.

2213A.9.3.1 Forces. Bracing connections shall have the strength to resist the lesser of the following:

1. The strength of the brace in axial tension, P_{st}.

2. Ω_o times the force in the brace due to the design seismic forces, in combination with gravity loads.

3. The maximum force that can be transferred to the brace by the system.

Bracing connections shall, as a minimum, satisfy the load combinations required by Section 1612A.3 at allowable stress limits with stress increases allowed by Section 1612A.3.2. Beam-to-column connections for beams that are part of the bracing system shall have the capacity to transfer the force determined above. Where eccentricities in the frame geometry or connection load path exist, the affected members and connections shall have the strength to resist all secondary forces resulting from the eccentricities in combination with all primary forces using the lesser of the forces determined above.

2213A.9.3.2 Net area. In bolted brace connections, the ratio of effective net section area to gross section shall satisfy Formula (13A-6) of Section 2213A.8.3.2.

2213A.9.3.3 Gusset plates. End connections of braces shall provide a flexural strength in excess of that of the brace gross section about the critical buckling axis.

> **EXCEPTION:** Where the out-of-plane buckling strength of the brace is less than the in-plane buckling strength, the brace is permitted to terminate on a single gusset plate connection with a setback of two times the gusset thickness from a line about which the gusset plate may bend unrestrained by the column or beam joints. The gusset plate shall be designed to carry the compressive strength of the brace without buckling.

2213A.9.4 Bracing configuration.

2213A.9.4.1 Chevron bracing. Chevron bracing shall conform with the following:

1. The beam intersected by chevron braces shall be continuous between columns.

2. Where chevron braces intersect a beam from below, i.e., inverted V brace, the beam shall be capable of supporting all tributary gravity loads presuming the bracing not to exist.

3. A beam intersected by chevron braces shall have the strength to support the following tributary gravity loads and unbalanced brace force combinations:

$$1.2D + 0.5L + P_b$$
$$0.9D - P_b$$

WHERE:

D = tributary dead load.

L = tributary live load.

P_b = the maximum unbalanced post-buckling force that can be applied to the beam by the braces. For this purpose, the maximum unbalanced force may be computed using a minimum of P_{st} for the tension and a maximum of $0.3\,P_{sc}$ for the compression brace.

4. Both flanges of beams at the point of intersection of chevron braces shall be laterally supported directly or indirectly.

> **EXCEPTION:** Limitations 2 and 3 need not apply to penthouses, one-story buildings or the top story of buildings.

2213A.9.4.2 K bracing. K bracing is prohibited.

2213A.9.5 Columns. Columns in braced frames shall meet the requirements of Section 2213A.7.3. In addition to meeting the requirements of Sections 2213A.5.1 and 2213A.5.2, column splices shall be designed to develop the full shear strength and 50 percent of the full moment strength of the section. Splices shall be located in the middle one third of the column clear height.

2213A.9.6 Nonbuilding structures. Nonbuilding structures with R_w values defined by Table 16A-P need comply only with the provisions of Sections 2213A.9.3.1 and 2213A.9.3.2.

2213A.10 Eccentrically Braced Frame (EBF) Requirements.

2213A.10.1 General. Eccentrically braced frames shall be designed in accordance with this section.

2213A.10.2 Link beam. There shall be a link beam provided at least at one end of each brace. Beams in EBFs shall comply with the requirements of Division III, except that the flange width-thickness ratio, $b_f/2t_f$, shall not exceed $52/\sqrt{F_y}$. (For **SI:** $0.31\sqrt{E/F_y}$.)

2213A.10.3 Link beam strength. Link beam shear strength, V_s, and flexural strength, M_s, are the strengths as defined in Section 2213A.4.2. Where link beam strength is governed by shear, the flexural and axial capacities within the link shall be calculated using the beam flanges only.

A reduced flexural strength, M_{rs}, for use in Sections 2213A.10.8 and 2213A.10.13 is defined as $Z(F_y - f_a)$. Where f_a is less than $0.15F_y$, f_a may be neglected.

2213A.10.4 Link beam rotation. The rotation of the link segment relative to the rest of the beam, at a total frame drift of Δ_M, shall not exceed the following:

1. 0.090 radian for link segments having clear lengths of 1.6 M_s/V_s or less.

2. 0.030 radian for link segments having clear lengths of 3.0 M_s/V_s or greater.

3. A value obtained by linear interpolation for clear lengths between the above limits.

2213A.10.5 Link beam web. The web of the link beam shall be single thickness without doubler plate reinforcement. No openings shall be placed in the web of a link beam. The web shear shall not exceed $0.8V_s$ under prescribed lateral forces.

2213A.10.6 Beam connection braces. Brace-to-beam connections shall develop the compression strength of the brace and transfer this force to the beam web. No part of the brace-to-beam connection shall extend into the web area of a link beam.

2213A.10.7 Link beam stiffeners. Link beams shall have full-depth web stiffeners on both sides of the beam web at the brace end of the link beam. In addition, for link beams with clear lengths within the limits in Section 2213A.10.4, Item 3, full-depth stiffeners shall be placed at a distance b_f from each end of the link. The stiffeners shall have a combined width not less than $b-2t_w$ and a thickness not less than 0.75 t_w or less than $^3/_8$ inch (9.5 mm).

2213A.10.8 Intermediate stiffeners. Intermediate full-depth web stiffeners shall be provided in either of the following conditions:

1. Where the link beam strength is controlled by V_s.

2. Where the link beam strength is controlled by flexure and the shear determined by applying the reduced flexural strength, M_{rs}, exceeds 0.45 $F_y dt$.

2213A.10.9 Web stiffener spacing. Where intermediate web stiffeners are required, the spacing shall conform to the requirements given below.

1. For link beams with rotation angle of 0.09 radian, the spacing shall not exceed $38t_w - d/5$.

2. For link beams with a rotation angle of 0.03 radian or less, the spacing shall not exceed $56t_w - d/5$. Interpolation may be used for rotation angles between 0.03 and 0.09 radian.

2213A.10.10 Web stiffener location. For beams 24 inches (610 mm) in depth and greater, intermediate full-depth web stiffeners are required on both sides of the web. Such web stiffeners are required only on one side of the beam web for beams less than 24 inches (610 mm) in depth. The stiffener thickness, t_w, of one side stiffeners shall not be less than $^3/_8$ inch (9.5 mm) and the width shall not be less than $(b_f/2) - t_w$.

2213A.10.11 Stiffener welds. Fillet welds connecting the stiffener to the beam web shall develop a stiffener force of $A_{st}F_y$. Fillet welds connecting the stiffener to the flanges shall develop a stiffener force of $A_{st}F_y/4$.

WHERE:

A_{st} = bt of stiffener.

b = width of stiffener plate.

2213A.10.12 Link beam-column connections. Length of link beam connected to columns shall not exceed 1.6 M_s/V_s.

1. Where a link beam is connected to the column flange, the following requirements shall be met:

   1.1 The beam flanges shall have full-penetration welds to the column.

   1.2 Where the link beam strength is controlled by shear in conformance with Section 2213A.10.8, the web connection shall be welded to develop the full link beam web shear strength.

2. Where the link beam is connected to the column web, the beam flanges shall have full-penetration welds to the connection plates and the web connection shall be welded to develop the link beam web shear strength. Rotation between the link beam and the column shall not exceed 0.015 radian at a total frame drift of Δ_M.

3. The link-to-column connection design shall be substantiated by cyclic test results that equal or exceed the rotation angle as prescribed in Section 2213A.10.12, Item 2. CACAC

2213A.10.13 Brace and beam strengths. The controlling link beam strength is either the shear strength, V_s, or the reduced flexural strength, M_{rs}, whichever results in the lesser axial force in the brace.

Each brace and beam outside the link shall have the axial strength or reduced flexural strength, M_{rs}, at least 1.5 times the forces corresponding to the controlling link beam strength. Each brace and beam assembly outside the link shall have combined reduced flexural strengths, M_{rs}, at least 1.3 times the forces corresponding to the controlling link beam strength.

2213A.10.14 Column strength. Columns shall be designed to remain elastic at 1.25 times the strength of the EBF bay, as defined in Section 2213A.10.13. Column strength need not exceed the requirements of Section 2213A.5.

2213A.10.15 Roof link beam. A link beam is not required in roof beams for EBF over five stories.

2213A.10.16 Concentric brace in combination. The first story of an EBF bay over five stories in height may be concentrically braced if this story can be shown to have an elastic capacity 50 percent greater than the yield capacity of the story frames above the first story.

2213A.10.17 Axial forces. Axial forces in beams of EBF frames due to braces and due to transfer of seismic force to the end of the frames shall be included in the frame calculations.

2213A.10.18 Beam flanges. Top and bottom flanges of EBF beams shall be laterally braced at the ends of link beams and at intervals not exceeding $76/\sqrt{F_y}$ (For **SI:** $0.45\sqrt{E/F_y}$) times the beam flange width. End bracing shall be designed to resist 6.0 percent of the beam flange strength, defined as $F_y b_f t_f$. Intermediate bracing shall be designed to resist 1.0 percent of the beam flange force at the brace point using the link beam strength determined in Section 2213A.10.13.

2213A.10.19 Beam-column connection. Beam connections to columns may be designed as pins in the plane of the beam web if

Division VII—SPECIFICATION FOR DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

SECTION 2217A — ADOPTION

Except for the modifications as set forth in Section 2218A of this division and the requirements of the building code, the design of cold-formed steel structural members shall be in accordance with the *Specification for Design of Cold-Formed Steel Structural Members,* 1986 (with December 1989 Addendum), published by the American Iron and Steel Institute, 1101 17th Street, NW, Suite 1300, Washington, DC, as if set out at length herein. The Specification for Design of Cold-Formed Steel Structural Members shall hereinafter be referred to as the AISI-ASD.

Where other codes, standards or specifications are referred to in this specification, they are to be considered as only an indication of an acceptable method or material that can be used with the approval of the *enforcement agency.*

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SECTION 2218A — AMENDMENTS

The following amendments shall be made to the AISI ASD specification, as adopted in Section 2217A:

1. Secs. A4.1 and A4.2. are deleted in their entirety without replacement.

2. Sec. A4.4. Revise as follows:

Where load combinations specified by Section 1612A.3 include wind or earthquake loads, the resulting forces may be multiplied by 0.75 for strength determination. Such reduction shall not be allowed in combination with stress increases in accordance with Section 1612A.3.1.

3. Sec. E6. Revise as follows:

The following notations apply to this section:

d = nominal screw diameter.

F_{u1} = tensile strength of member in contact with the screw head.

F_{u2} = tensile strength of member not in contact with the screw head.

P_{as} = allowable shear force per screw.

P_{at} = allowable tension force per screw.

P_{not} = pull-out force per screw.

P_{nov} = pull-over force per screw.

P_{ns} = nominal shear strength per screw.

P_{nt} = nominal tension strength per screw.

t_1 = thickness of member in contact with the screw head.

t_2 = thickness of member not in contact with the screw head.

Ω = factor of safety = 3.0.

All E6 requirements shall apply to self-tapping screws with 0.08 inch (2.03 mm) < d < 0.25 inch (6.35 mm). The screws shall be thread-forming or thread-cutting, with or without a self-drilling point. Alternatively, design values for a particular application shall be permitted to be based on tests according to Section F. For diaphragm applications, Section D5 shall be used.

Screws shall be installed and tightened in accordance with the manufacturer's recommendations.

The tension force on the net section of each member joined by a screw connection shall not exceed T_a from Section C2 or P_a from Section E3.2.

E6.1 Minimum Spacing. The distance between the centers of fasteners shall not be less than 3d.

E6.2 Minimum Edge and End Distance. The distance from the center of a fastener to the edge of any part shall not be less than 3d. If the connection is subjected to shear force in one direction only, the minimum edge distance shall be reduced to 1.5d in the direction perpendicular to the force.

E6.3 Shear.

E6.3.1 Connection shear. The shear force per screw shall not exceed P_{as} calculated as follows:

$$P_{as} = P_{ns}/\Omega$$

For $t_2/t_1 \leq 1.0$, P_{ns} = shall be taken as the smallest of

$$P_{ns} = 4.2\,(t_2^3\,d)^{1/2}\,F_{u2} \qquad \text{(Eq. E6.3.1)}$$

$$P_{ns} = 2.7 t_1\,d\,F_{u1} \qquad \text{(Eq. E6.3.2)}$$

$$P_{ns} = 2.7\,t_2\,d\,F_{u2} \qquad \text{(Eq. E6.3.3)}$$

For $t_2/t_1 \geq 2.5$, P_{ns} shall be taken as the smaller of

$$P_{ns} = 2.7 t_1\,d\,F_{u1} \qquad \text{(Eq. E6.3.4)}$$

$$P_{ns} = 2.7\,t_2\,d\,F_{u2} \qquad \text{(Eq. E6.3.5)}$$

For $1.0 < t_2/t_1 < 2.5$, P_{ns} shall be determined by linear interpolation between the above two cases.

E6.3.2 Shear in screws. The shear capacity of the screw shall be determined by test according to Section F1(a). The shear capacity of the screw shall not be less than 1.25 P_{ns}.

E6.4 Tension. For screws that carry tensile loads, the head of the screw or washer, if a washer is provided, shall have a diameter d_w, not less than $^5/_{16}$ inch (7.95 mm). Washers shall be at least 0.050 inch (1.27 mm) thick.

The tension force per screw shall not exceed P_{at}, calculated as follows:

$$P_{at} = P_{nt}/\Omega \qquad \text{(Eq. E6.4.l)}$$

P_{nt} = shall be taken as the lesser of P_{not} and

P_{nov} as determined in Sections E4.4.l and E4.4.2.

E6.4.1 Pull-out. The pull-out force, P_{not}, shall be calculated as follows:

$$P_{not} = 0.85 t_c\,d\,F_{u2} \qquad \text{(Eq. E6.4.1.)}$$

where t_c is the lesser of the depth of the penetration and the thickness, t_2.

E6.4.2 Pull-over. The pull-over force, P_{nov}, shall be calculated as follows:

$$P_{nov} = 1.5 t_1\,d_w\,F_{u1} \qquad \text{(Eq. E6.4.2.l)}$$

where d_w is the larger of the screw head diameter or the washer diameter and shall be taken not larger than $^1/_2$ inch (12.7 mm).

E6.4.3 Tension in screws. The tensile capacity of the screw shall be determined by test according to Section F1(a). The tensile capacity of the screw shall not be less than 1.25 P_{nt}.

Division VIII—LATERAL RESISTANCE FOR STEEL STUD WALL SYSTEMS

SECTION 2219A — GENERAL

Steel stud wall systems in which shear panels are used to resist lateral loads produced by wind or earthquake shall comply with the requirements of this section. The nominal shear value used to establish the allowable shear value or design shear value shall not exceed the values set forth in Table 22A-VIII-A ✳ ✳ ✳ for wind loads or Table 22A-VIII-C for seismic loads. The allowable shear value (ASD) or design *shear* value (LRFD) shall be determined using the ϕ or Ω factors as set forth in Section 2219A.3.

All boundary members and connections thereto shall be proportioned to transmit the induced forces. Framing members shall be of a minimum size, shape and of a minimum specified yield stress as listed in Table 22A-VIII-A ✳ ✳ ✳ or 22A-VIII-C. Fasteners between framing members and between the panels and the framing members shall be as specified in Table 22A-VIII-A ✳ ✳ ✳ or 22A-VIII-C. Fasteners along the edges in shear panels shall be placed not less than $^3/_8$ inch (9.5 mm) in from panel edges. Screws shall be of sufficient length to ensure penetration into the steel stud by at least two full diameter threads.

Panel thickness shown in Table 22A-VIII-A ✳ ✳ ✳ shall be considered as minimum.

No panels less than 12 inches (305 mm) wide shall be used. All panel edges shall be fully blocked. Where horizontal strap blocking is used, it shall be a minimum $1^1/_2$ inches (38 mm) wide and of the same material and thickness as the track and studs. Studs shall be doubled (back to back) at shear wall ends.

The height to length ratio of wall systems listed in Tables 22A-VIII-A ✳ ✳ ✳ and 22A-VIII-C shall not exceed 2:1.

2219A.1 ***Plywood*** **Structural Panel Sheathing.** ✳ ✳ ✳ *Plywood* structural panels may be applied either parallel to or perpendicular to framing. No increase of the nominal loads shown in Tables 22A-VIII-A and 22A-VIII-C shall be permitted for duration of load nor shall an increase in nominal loads be permitted for installing sheathing on the opposite side unless indicated herein.

2219A.2 *Not adopted by the State of California*

2219A.3 Design. Where allowable stress design is used, the allowable shear value shall be determined by dividing the nominal shear value, shown in Table 22A-VIII-A, ✳ ✳ ✳ by a factor of safety (Ω) which shall be taken as 3.0. The factor of safety (Ω) for the nominal loads shown in Table 22A-VIII-C shall be taken as *3.0*.

Where Load and Resistance Factor Design is used, the design shear value shall be determined by multiplying the nominal shear value, shown in Table 22A-VIII-A by a resistance factor *(ϕ)* which shall be taken as 0.45. The resistance factor *(ϕ)* for the nominal loads shown in Table 22A-VIII-C shall be taken as 0.55.

SECTION 2220A — SPECIAL REQUIREMENTS IN SEISMIC ZONES 3 AND 4

2220A.1 General. In Seismic Zones 3 and 4, in addition to the requirements of Section 2219A, steel stud wall systems may be used to resist the specified seismic forces in buildings not over *three* stories in height. Such systems shall comply with the following:

1. The l/r of the brace may exceed 200 and is unlimited.

2. All boundary members, chords and collectors shall be designed and detailed to transmit the induced axial forces.

3. Connection of the diagonal bracing member, top chord splices, boundary members and collectors shall be designed to develop the full tensile strength of the member or Ω_o times the otherwise prescribed seismic forces.

4. Vertical and diagonal members of the braced bay shall be anchored so the bottom track is not required to resist uplift forces by bending of the track web.

5. Both flanges of studs in a bracing panel shall be braced to prevent lateral torsional buckling. Wire-tied bridging shall not be considered to provide such restraint.

6. Screws shall not be used to resist lateral forces by pullout resistance.

7. Provision shall be made for pretensioning or other methods of installation of tension-only bracing to guard against loose diagonal straps.

2220A.2 Boundary Members and Anchorage. Boundary members and the uplift anchorage thereto shall have the strength to resist the forces determined by the *following* load *combinations*:

1. *Axial compression* $1.0P_{DL} + 0.7P_{LL} + \Omega_O P_E$
2. *Axial tension* $0.85P_{DL} - \Omega_O P_E$

2220A.3 ***Plywood*** **Panel Sheathing.** Where *plywood* structural panels provide lateral resistance, the design and construction of such walls shall be in accordance with the additional requirements of this section. Perimeter members at openings shall be provided and shall be detailed to distribute the shearing stresses.

Wood sheathing shall not be used to splice these members. *Plywood* structural panels shall be manufactured using exterior glue.

Wall studs and track shall have a minimum uncoated base metal thickness of not less than 0.033 inch (0.84 mm) and shall not have an uncoated base metal thickness greater than 0.043 inch (1.10 mm).

Chapter 22B *[BSC, HCD 1 & HCD 2]*

STEEL

Division I—GENERAL

See Chapter 22, STEEL, Division I for content of this division.

Division II—DESIGN STANDARD FOR LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR STRUCTURAL STEEL BUILDINGS

See Chapter 22, STEEL, Division II for content of this division.

Division III—DESIGN STANDARD FOR SPECIFICATION FOR STRUCTURAL STEEL BUILDINGS ALLOWABLE STRESS DESIGN AND PLASTIC DESIGN

See Chapter 22, STEEL, Division III for content of this division.

Division IV—SEISMIC PROVISIONS FOR STRUCTURAL STEEL BUILDINGS

Based on Seismic Provisions for Structural Steel Buildings of the American Institute of Steel Construction.

(Part I, dated April 15, 1997 and Supplement No. 2, dated November 10, 2000.)

SECTION 2210B — ADOPTION

Except for the modifications as set forth in Sections 2211B and 2212B of this division and the requirements of the Building Code, the seismic design, fabrication, and erection of structural steel shall be in accordance with the Seismic Provisions for Structural Steel Buildings, April 15, 1997, published by the American Institute of Steel Construction, 1 East Wacker Drive, Suite 3100, Chicago, IL 60601, as if set out at length herein. The adoption of Seismic Provisions for Structural Steel Buildings in this Division, hereinafter referred to as AISC-Seismic, shall include Parts I (LRFD), and Supplement No. 2, dated November 10, 2000.

Where other codes, standards, or specifications are referred to in this specification, they are to be considered as only an indication of an acceptable method or material that can be used with the approval of the Building Official.

SECTION 2211B — DESIGN METHODS

When the load combinations from Section 1612.2 for LRFD are used, structural steel buildings shall be designed in accordance with Chapter 22 Division II (AISC-LRFD) and Part I of AISC-Seismic as modified by this Division.

SECTION 2212B — AMENDMENTS

The AISC-Seismic adopted by this Division applies to the seismic design of structural steel members except as modified by this Section.

The following terms that appear in AISC-Seismic shall be taken as indicated in the 1997 Uniform Building Code.

AISC-Seismic	***1997 Uniform Building Code***
Seismic Force Resisting System	*Lateral Force Resisting System*
Design Earthquake	*Design Basis Ground Motion*
Load Combinations Eqs. (4-1) and (4-2)	*Chapter 16 Eqs. (12-17) and (12-18) respectively*
LRFD Specification Section Eqs. (A4-1) through (A4-6)	Chapter 16 Eqs. (12-1) through (12-6) respectively
$\Omega_o Q_E$	E_m

1. Part I, Sec. 1. of the AISC Seismic Provisions is revised as follows:

1. SCOPE

These provisions are intended for the design and construction of structural steel members and connections in the Seismic Force Resisting Systems in buildings for which the design forces resulting from earthquake motions have been determined on the basis of various levels of energy dissipation in the inelastic range of response. These provisions shall apply to buildings in Seismic Zone 2 with an importance factor I greater than one, in Seismic Zones 3 and 4 or when required by the Engineer of Record.

These provisions shall be applied in conjunction with Chapter 22, Division II, hereinafter referred to as the LRFD Specification. All members and connections in the Lateral Force Resisting System shall have a design strength as provided in the LRFD Specification to resist load combinations 12-1 through 12-6 (in Chapter 16) and shall meet the requirements in these provisions.

Part I includes a Glossary, which is specifically applicable to this Part, and Appendix S.

2. Part I, Sec. 4.1. of the AISC Seismic Provisions is deleted and replaced as follows:

4.1 Loads and Load Combinations

The loads and load combinations shall be those in Section 1612.2 except as modified throughout these provisions.

E_h is the horizontal component of earthquake load E required in Chapter 16. Where required in these provisions, an amplified horizontal earthquake load $\Omega_o E_h$ shall be used in lieu of E_h as given in the load combinations below. The term Ω_o is the system overstrength factor as defined in chapter 16. The additional load combinations using amplified horizontal earthquake load are:

$$1.2\,D + 0.5\,L + 0.2S + \Omega_o E_h \qquad (4\text{-}1)$$

$$0.9\,D + \Omega_o E_h \qquad (4\text{-}2)$$

EXCEPTION: *The load factor on L in load combination 4-1 shall be equal to 1.0 for garages, areas occupied as places of public assembly and all areas where the live load is greater than 100 psf.*

Division V—*SEISMIC PROVISIONS FOR STRUCTURAL STEEL BUILDINGS FOR USE WITH ALLOWABLE STRESS DESIGN*

Based on Seismic Provisions for Structural Steel Buildings, of the American Institute of Steel Construction.

(Part III, dated April 15, 1997 and Supplement No. 2, dated November 10, 2000.)

SECTION 2213B — ADOPTION [HCD 1 & HCD 2]

Except for the modifications as set forth in Sections 2211B and 2212B of this division and the requirements of the Building Code, the seismic design, fabrication and erection of structural steel shall be in accordance with the Seismic Provisions for Structural Steel Buildings, April 15, 1997 published by the American Institute of Steel Construction, 1 East Wacker Drive, Suite 3100, Chicago, IL 60601, as if set out at length herein. The adoption of Seismic Provisions for Structural Steel Buildings in this Division, hereinafter referred to as AISC-Seismic, shall include Parts III (ASD) and Supplement No. 2, dated November 10, 2000.

Where other codes, standards or specifications are referred to in this specification, they are to be considered as only an indication of an acceptable method or material that can be used with the approval of the Building Official.

SECTION 2214B — DESIGN METHODS [HCD 1 & HCD2]

When the Allowable Stress Design (ASD) method is used for design of members, structural steel buildings shall be designed in accordance with Chapter 22 Division III (AISC-ASD) and Part III of AISC-Seismic as modified by this Division.

SECTION 2215B — AMENDMENTS

The AISC-Seismic adopted by this Division apply to the seismic design of structural steel members except as modified by this Section.

The following terms that appear in AISC-Seismic shall be taken as indicated in the 1997 Uniform Building Code.

AISC-Seismic	*1997 Uniform Building Code*
Seismic Force Resisting System	*Lateral Force Resisting System*
Design Earthquake	*Design Basis Ground Motion*
Load Combinations Eqs. (4-1) and (4-2)	*Chapter 16 Eqs. (12-17) and (12-18) respectively*
$\Omega_o Q_E$	E_m

1. Part III, Sec. 1. of the AISC Seismic Provisions is revised as follows:

1. SCOPE

These provisions are intended for the design and construction of structural steel members and connections in the Seismic Force Resisting Systems in buildings for which the design forces resulting from earthquake motions have been determined on the basis of various levels of energy dissipation in the inelastic range of response. These provisions shall apply to buildings in Seismic Zone 2 with an importance factor I greater than one, in Seismic Zones 3 and 4 or when required by the Engineer of Record.

These provisions shall be applied in conjunction with Chapter 22, Division III, hereinafter referred to as the ASD Specification. All members and connections in the Lateral Force Resisting System shall have a design strength as provided in the ASD Specification to resist load combinations 12-1 through 12-6 (in Chapter 16) and shall meet the requirements in these provisions.

Part I includes a Glossary, which is specifically applicable to this Part, and Appendix S.

2. Part III, Sec. 4.1. of the AISC Seismic Provisions is deleted and replaced as follows:

2.1 Loads and Load Combinations

The loads and load combinations shall be those in Section 1612.2 except as modified throughout these provisions.

E_h is the horizontal component of earthquake load E required in Chapter 16. Where required in these provisions, an amplified horizontal earthquake load $\Omega_o E_h$ shall be used in lieu of E_h as given in the load combinations below. The term Ω_o is the system overstrength factor as defined in chapter 16. The additional load combinations using amplified horizontal earthquake load are:

$$1.2\,D + 0.5\,L + 0.2S + \Omega_o E_h \qquad (4\text{-}1)$$

$$0.9\,D + \Omega_o E_h \qquad (4\text{-}2)$$

EXCEPTION: *the load factor on L in load combination 4-1 shall be equal to 1.0 for garages, areas occupied as places of public assembly and all areas where the live load is greater than 100 psf.*

Division VI—LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR COLD-FORMED STEEL STRUCTURAL MEMBERS

See Chapter 22, STEEL, Division VI for content of this division.

Division VII—SPECIFICATION FOR DESIGN OF COLD-FORMED STEEL STRUCTURAL MEMBERS

See Chapter 22, STEEL, Division VII for content of this division.

Division VIII—LATERAL RESISTANCE FOR STEEL STUD WALL SYSTEMS

See Chapter 22, STEEL, Division VIII for content of this division.

Division IX—OPEN WEB STEEL JOISTS

See Chapter 22, STEEL, Division IX for content of this division.

Division X—DESIGN STANDARD FOR STEEL STORAGE RACKS

See Chapter 22, STEEL, Division X for content of this division.

Division XI—DESIGN STANDARD FOR STRUCTURAL APPLICATIONS OF STEEL CABLES FOR BUILDINGS

See Chapter 22, STEEL, Division XI for content of this division.

Division III—DESIGN SPECIFICATIONS FOR ALLOWABLE STRESS DESIGN OF WOOD BUILDINGS

Part I—ALLOWABLE STRESS DESIGN OF WOOD

This standard, with certain exceptions, is the ANSI/NFoPA-91 National Design Specification for Wood Construction of the American Forest and Paper Association, Revised 1991 Edition, and the Supplement to the 1991 Edition, National Design Specification, adopted by reference.

The National Design Specification for Wood Construction, Revised 1991 Edition, and supplement are available from the American Forest and Paper Association, 1111 19th Street, NW, Eighth Floor, Washington, DC, 20036.

[For BSC, HCD 1 & HCD 2 & OSHPD 2] For applications regulated by the Building Standards Commission as referenced in Section 101.17.3; applications regulated by the Department of Housing and Community Development as referenced in Sections 101.17.9 and 101.17.10; and applications regulated by the Office of Statewide Health Planning and Development in Section 101.17.13, Item 2, this standard, with certain exceptions, is the ANSI/AF&PA NDS-01 National Design Specification for Wood Construction of the American Forest and Paper Association, 2001 Edition, and the Supplement to the 2001 Edition, National Design Specification, adopted by reference.

SECTION 2316 — DESIGN SPECIFICATIONS

2316.1 Adoption and Scope.

2316.1.1 The National Design Specification for Wood Construction, Revised 1991 Edition (NDS), which is hereby adopted as a part of this code, shall apply to the design and construction of wood structures using visually graded lumber, mechanically graded lumber, structural glued laminated timber, and timber piles. National Design Specification Appendix Section F, Design for Creep and Critical Deflection Applications, Appendix Section G, Effective Column Length, and Appendix Section J, Solution of Hankinson Formula are specifically adopted and made a part of this standard. The Supplement to the 1991 Edition National Design Specification, Tables 2A, 4A, 4B, 4C, 4D, 4E, 5A, 5B and 5C are specifically adopted and made a part of this standard.

Other codes, standards or specifications referred to in this standard are to be considered as only an indication of an acceptable method or material that can be used with the approval of the building official, except where such other codes, standards or specifications are specifically adopted by this code as primary standards.

2316.1.2 [For BSC, HCD 1 & HCD 2, OSHPD 2] The National Design Specification for Wood Construction, *2001* Edition (NDS), *as amended by Section 2316.3,* which is hereby adopted as a part of this code, shall apply to the *allowable stress* design and construction of wood structures. The Supplement to the *2001* Edition National Design Specification is specifically adopted and made a part of this standard.

Where a code, standard or specification referred to in this code conflicts with a code, standard or specification referenced in the NDS-01 for allowable stress design of wood building, the NDS-01 shall prevail.

2316.2 Amendments.

NOTE: *The provisions of this section shall not apply to applications regulated by the Building Standards Commission as referenced in Section 101.17.3; applications regulated by the Department of Housing and Community Development as referenced in Sections 101.17.9 and 101.17.10; and applications regulated by the Office of Statewide Health Planning and Development as referenced in Section 101.17.13, Item 2.*

1. Sec. 1.1. Delete and substitute the following:

The design of structures using visually graded lumber, mechanically graded lumber, structural glued laminated timber, timber piles, and design of their connections shall be in accordance with Chapter 23, Division III, Part 1.

2. Secs. 1.2 through 1.5. Delete.

3. Sec. 2.2. Delete first sentence and substitute the following:

Allowable stress design values for visually graded structural lumber, mechanically graded structural lumber and structural glued laminated timber shall be in accordance with NDS Supplement Tables 2A, 4A, 4B, 4C, 4D, 5A, 5B and 5C. Values for species and grades not tabulated shall be submitted to the building official for approval.

4. Sec. 2.3.2.1. In fourth sentence, delete "or Figure B1 (see Appendix B)."

5. Sec. 2.3.2.3. Delete and substitute the following:

2.3.2.3 When using Section 1612.3.1 basic load combinations, the Load Duration Factor, C_D, noted in Table 2.3.2 shall be permitted to be used. When using Section 1612.3.2 alternate load combinations, the one-third increase shall not be used concurrently with the Load Duration Factor, C_D.

6. Table 2.3.2. Delete and substitute as follows:

TABLE 2.3.2—LOAD DURATION FACTORS, C_D

DESIGN LOAD	LOAD DURATION	C_D
Dead Load	Permanent	0.9
Floor, Occupancy Live Load	Ten Years	1.0
Snow Load	Two Months	1.15
Roof Live Load	Seven Days	1.25
Earthquake Load[1]	—	1.33
Wind Load[2]	—	1.33
Impact	—	2.0

[1]1.60 may be used for nailed and bolted connections exhibiting Mode III or IV behavior, except that the increases for earthquake are not combined with the increase allowed in Section 1612.3. The 60-percent increase for nailed and bolted connections exhibiting Mode III or IV behavior for earthquake shall not be applicable to joist hangers, framing anchors, and other mechanical fastenings, including straps and hold-down anchors. The 60-percent increase shall not apply to the allowable shear values in Tables 23-II-H, 23-II-I-1, 23-II-I-2, 23-II-J or in Section 2315.3.

[2]1.60 may be used for members and nailed and bolted connections exhibiting Mode III or IV behavior, except that the increases for wind are not combined with the increase allowed in Section 1612.3. The 60-percent increase shall not apply to the allowable shear values in Tables 23-II-H, 23-II-I-1, 23-II-I-2, 23-II-J or in Section 2315.3.

7. Sec. 2.3.4. Add a second paragraph following Table 2.3.4:

The allowable unit stresses for fire-retardant-treated solid-sawn lumber and plywood, including fastener values, subject to prolonged elevated temperatures from manufacturing or equipment processes, but not exceeding 150°F (66°C), shall be developed from approved test methods that properly consider potential strength-reduction characteristics, including effects of heat and moisture.

8. Sec. 2.3.6. Add second, third and fourth paragraphs as follows:

The values for lumber and plywood impregnated with approved fire-retardant chemicals, including fastener values, shall be submitted to the building official for approval. Submittal to the building official shall include all substantiating data. Such values shall be developed from approved test methods and procedures that consider potential strength-reduction characteristics, including the effects of elevated temperatures and moisture. Other adjustments are applicable, except that the impact load-duration factor shall not apply.

Values for glued-laminated timber, including fastener design values, shall be recommended by the treater and submitted to the building official for approval. Submittal to the building official shall include all substantiating data.

In addition to the requirements specified in Section 207, fire-retardant lumber having structural applications shall be tested and identified by an approved inspection agency in accordance with UBC Standard 23-5.

9. Sec. 2.3.8. Add new second and third paragraphs following Table 2.3.8:

For lumber I beams and box beams, the form factor, C_f, shall be calculated as:

$$C_f = \left[1 + \left(\frac{d^2 + 143}{d^2 + 88} - 1\right) C_g\right]$$

For **SI:**

$$C_f = \left[1 + \left(\frac{\left(\frac{d}{25.4}\right)^2 + 143}{\left(\frac{d}{25.4}\right)^2 + 88} - 1\right) C_g\right]$$

WHERE:

C_f = form factor.

C_g = support factor = $p^2(6 - 8p + 3p^2)(1 - q) + q$.

d = depth of I or box beam.

p = ratio of depth of compression flange to full depth of beam.

q = ratio of thickness of web or webs to full width of beam.

10. Sec. 2.3.10. Add a paragraph at end of section as follows:

In joists supported on a ribbon or ledger board and spiked to the studding, the allowable stress in compression perpendicular to grain may be increased 50 percent.

11. Sec. 3.2.1. Add a second sentence as follows:

For continuous beams, the span shall be taken as the distance between centers of bearings on supports over which the beam is continuous.

12. Sec. 3.2.3.2. Add to end of paragraph as follows:

Cantilevered portions of beams less than 4 inches (102 mm) in nominal thickness shall not be notched unless the reduced section properties and lumber defects are considered in the design. For effects of notch on shear strength, see Section 3.4.4.

13. Sec. 3.3.2. Add a last paragraph as follows:

A beam of circular cross section may be assumed to have the same strength as a square beam having the same cross-sectional area. If a circular beam is tapered, it shall be considered a beam of variable cross section.

14. Sec. 3.4.4. Add a section as follows:

3.4.4.5 When girders, beams or joists are notched at points of support on the compression side, they shall meet design requirements for the net section in bending and in shear. The actual shear stress as such point shall be calculated as follows:

$$f_v = 3V / 2b\,[d - ((d - d') / d')\,e]$$

WHERE:

d = total depth.

d' = actual depth of beam at notch.

e = distance notch extends inside the inner edge of support.

V = shear force.

Where e exceeds d', the actual shear stress for the notch on the compression side shall be calculated as follows:

$$F_v = 3V / 2bd'$$

15. Sec. 3.7.1.4. Delete and substitute as follows:

The slenderness ratio for solid columns, le/d shall not exceed 50.

16. Sec. 3.8.2. Delete and substitute as follows:

Where designs that induce tension stresses perpendicular to grain cannot be avoided, mechanical reinforcement sufficient to resist such forces shall be provided.

17. Sec. 4.2.5.5. Delete.

18. Sec. 4.4.1.1. Delete and substitute as follows:

Rectangular sawn lumber beams, rafters, joists or other bending members shall be supported laterally to prevent rotation or lateral displacement in accordance with Section 4.4.1.2, or shall be designed in accordance with the lateral stability provisions in Section 3.3.3.

19. Sec. 4.4.1.2. Delete first sentence.

20. Sec. 5.4.1. Delete second paragraph and substitute as follows:

For curved bending members having a varying cross section, the maximum actual radial stress induced, f_r, is given by:

$$f_r = K_r \frac{6M}{bd^2}$$

WHERE:

b = width of cross section, inches (mm).

d = depth of cross section at the apex in inches (mm).

K_r = radial stress factor determined from the following relationship:

$$K_r = A + B\left(\frac{d}{Rm}\right) + C\left(\frac{d}{Rm}\right)^2$$

M = bending moment at midspan in inch-pounds (N·mm).

WHERE:

Rm = radius of curvature at the center line of the member at midspan in inches (mm).

A, B and C = constants as follows:

β (1)	A (2)	B (3)	C (4)
(0.0)	(0.0)	(0.2500)	(0.0)
2.5°	0.0079	0.1747	0.1284
5.0°	0.0174	0.1251	0.1939
7.5°	0.0279	0.0937	0.2162
10.0°	0.0391	0.0754	0.2119
15.0°	0.0629	0.0619	0.1722
20.0°	0.0893	0.0608	0.1393
25.0°	0.1214	0.0605	0.1238
30.0°	0.1649	0.0603	0.1115

and β = angle between the upper edge of the member and the horizontal in degrees. Values of K_r for intermediate values of β may be interpolated linearly.

When the beam is loaded with a uniform load, K_r may be modified by multiplying by the reduction factor C_r as calculated by the following formula:

$$C_r = A + B\left(\frac{L}{L_t}\right) + C\left(\frac{d_c}{R_m}\right) + D\left(\frac{L}{L_t}\right)^2 + E\left(\frac{d_c}{R_m}\right)^2 + F\left(\frac{d_c}{R_m}\right)\left(\frac{L}{L_t}\right) + G\left(\frac{L}{L_t}\right)^3 + H\left(\frac{d_c}{R_m}\right)^3$$

WHERE:

C_r = reduction factor.

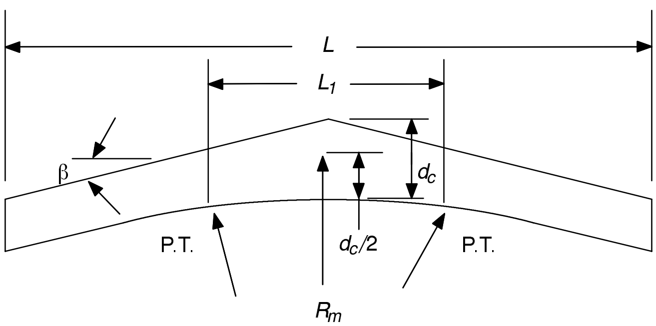
L = span of beam.

L_t = length of beam between tangent points.

A, B
$\ldots H$ = constants for a given β as follows:

β	A	B	C	D	E	F	G	H
2.3°	-0.142	0.418	-2.358	-0.053	—	—	0.002	—
9.7°	0.143	0.376	-0.541	-0.060	—	—	0.003	—
14.9°	0.406	0.293	-0.927	-0.041	—	—	0.002	—
20.0°	0.423	0.364	-1.022	-0.067	—	0.146	—	—
25.2°	0.540	0.360	-1.061	-0.070	—	0.156	—	—
29.8°	0.502	0.372	—	-0.076	-3.712	0.138	0.004	4.336

and β = angle between the upper edge of the member and the horizontal in degrees. Values of C_r for intermediate values may be interpolated linearly.



PITCHED AND TAPERED CURVED BEAM

21. Sec. 5.4.1.2. Add a second paragraph:

These values are subject to modification for duration of load. If these values are exceeded, mechanical reinforcing sufficient to resist all radial tension shall be provided, but in no case shall the calculated radial tension stress exceed one third the allowable unit stress in horizontal shear. When mechanical reinforcing is used, the maximum moisture content of the laminations at the time of manufacture shall not exceed 12 percent for dry conditions of use.

22. Sec. 5.4.4. Add a section as follows:

5.4.4 Ponding. Roof-framing members shall be designed for the deflection and drainage or ponding requirements specified in Section 1506 and Chapter 16. In glued-laminated timbers, the minimum slope for roof drainage required by Section 1506 shall be in addition to a camber of one and one-half times the calculated dead load deflection. The calculation of the required slope shall not include any vertical displacement created by short taper cuts. In no case shall the deflection of glued-laminated timber roof members exceed $^1/_2$-inch (13 mm) for a 5 pound-per-square-foot (239 Pa) uniform load.

23. Sec. 5.4.5. Add a new section as follows:

5.4.5 Tapered Faces. Sawn tapered cuts shall not be permitted on the tension face of any beam. Pitched or curved beams shall be so fabricated that the laminations are parallel to the tension face. Straight, pitched or curved beams may have sawn tapered cuts on the compression face.

For other members subject to bending, the slope of tapered faces, measured from the tangent to the lamination of the section under consideration, shall not be steeper than 1 unit vertical in 24 units horizontal (4% slope) on the tension side.

EXCEPTIONS: 1. This requirement does not apply to arches.

2. Taper may be steeper at sections increased in size beyond design requirements for architectural projections.

24. Sec. 8.3. Add a section as follows:

8.3 Allowable shear values for bolts used to connect a wood member to concrete or masonry are permitted to be determined as one half the tabulated double shear value for a wood member twice the thickness of the member attached to the concrete or masonry.

25. Sec. 12.4.1. Delete and substitute as follows:

12.4.1 For wood-to-wood joints, the spacing center to center of nails in the direction of stress shall not be less than the required penetration. Edge or end distances in the direction of stress shall not be less than one-half of the required penetration. All spacing and edge and end distances shall be such as to avoid splitting of the wood.

2316.3 [For BSC, HCD 1 & HCD 2, OSHPD 2] Amendments.

1. [For BSC, HCD 1 & HCD 2, OSHPD 2] Sec. 2.2. Add a fifth sentence as follows:

Values for species and grades not tabulated shall be submitted to the enforcing agency for approval.

2. [For BSC, HCD 1 & HCD 2, OSHPD 2] Sec. 2.3.2.1. In fourth sentence, delete "or Figure B1 (see Appendix B)."

3. [For BSC, HCD 1 & HCD 2, OSHPD 2] Sec. 2.3.2.3. Delete and substitute the following:

2.3.2.3 *When using Section 1612.3.1 basic load combinations, the Load Duration Factor, C_D, noted in Table 2.3.2 shall be permitted to be used. When using Section 1612.3.2 alternate load combinations, the one-third increase shall not be used concurrently with the Load Duration Factor, C_D.*

4. [For BSC, HCD 1 & HCD 2, OSHPD 2] Table 2.3.2. Delete and substitute as follows:

TABLE 2.3.2—LOAD DURATION FACTORS, C_D

DESIGN LOAD	*LOAD DURATION*	C_D
Dead Load	*Permanent*	*0.9*
Floor, Occupancy Live Load	*Ten Years*	*1.0*
Snow Load	*Two Months*	*1.15*
Roof Live Load	*Seven Days*	*1.25*
Earthquake Load[1]	—	*1.33*
Wind Load[2]	—	*1.33*
Impact	—	*2.0*

[1] *1.60 may be used for nailed and bolted connections exhibiting Mode III or IV behavior, except that the increases for earthquake are not combined with the increase allowed in Section 1612.3. The 60-percent increase for nailed and bolted connections exhibiting Mode III or IV behavior for earthquake shall not be applicable to joist hangers, framing anchors and other mechanical fastenings, including straps and hold-down anchors. The 60-percent increase shall not apply to the allowable shear values in Tables 23-II-H, 23-II-I-1, 23-II-I-2, 23-II-J or in Section 2315.3.*

[2] *1.60 may be used for members and nailed and bolted connections exhibiting Mode III or IV behavior, except that the increases for wind are not combined with the increase allowed in Section 1612.3. The 60-percent increase shall not apply to the allowable shear values in Tables 23-II-H, 23-II-I-1, 23-II-I-2, 23-II-J or in Section 2315.3.*

5. [For BSC, HCD 1 & HCD 2 & OSHPD 2] Sec. 2.3.3. Add a second paragraph following Table 2.3.3:

The allowable unit stresses for fire-retardant-treated solid-sawn lumber and plywood, including fastener values, subject to prolonged elevated temperatures from manufacturing or equipment processes, but not exceeding 150 °F (66 °C), shall be developed from approved test methods that properly consider potential strength-reduction characteristics, including effects of heat and moisture.

6. [For BSC, HCD 1 & HCD 2, OSHPD 2] Sec. 2.3.4. Add second, third and fourth paragraphs as follows:

The values for lumber and plywood impregnated with approved fire-retardant chemicals, including fastener values, shall be submitted to the building official for approval. Submittal to the building official shall include all substantiating data. Such values shall be developed from approved test methods and procedures that consider potential strength-reduction characteristics, including the effects of elevated temperatures and moisture. Other adjustments are applicable, except that the impact load-duration factor shall not apply.

Values for glued-laminated timber, including fastener design values, shall be recommended by the treater and submitted to the building official for approval. Submittal to the building official shall include all substantiating data.

In addition to the requirements specified in Section 207, fire retardant lumber having structural applications shall be tested and identified by an approved inspection agency in accordance with UBC Standard 23-5.

7. [For BSC, HCD 1 & HCD 2, OSHPD 2} Sec. 5.4. Add a section as follows:

***5.4.5 Ponding.** Roof-framing members shall be designed for the deflection and drainage or ponding requirements specified in Section 1506 and Chapter 16. In glued-laminated timbers, the minimum slope for roof drainage required by Section 1506 shall be in addition to a camber of one and one-half times the calculated dead load deflection. The calculation of the required slope shall not include any vertical displacement created by short taper cuts. In no case shall the deflection of glued-laminated timber roof members exceed 1/2-inch (13 mm) for a 5 pound-per-square-foot (239 Pa) uniform load.*

8. [For BSC, HCD 1 & HCD 2, OSHPD 2] Sec. 5.4. Add a new section as follows:

***5.4.6 Tapered Faces.** Sawn tapered cuts shall not be permitted on the tension face of any beam. Pitched or curved beams shall be so fabricated that the laminations are parallel to the tension face. Straight, pitched or curved beams may have sawn tapered cuts on the compression face.*

For other members subject to bending, the slope of tapered faces, measured from the tangent to the lamination of the section under consideration, shall not be steeper than 1 unit vertical in 24 units horizontal (4% slope) on the tension side.

> *EXCEPTION: 1. This requirement does not apply to arches.*
>
> *2. Taper may be steeper at sections increased in size beyond design requirements for architectural projections.*

9. [For BSC, HCD 1 & HCD 2, OSHPD 2] Sec. 11.1.5.6. Delete and substitute as follows:

11.1.5.6 For wood-to-wood joints, the spacing center to center of nails in the direction of stress shall not be less than the required penetration. Edge or end distances in the direction of stress shall not be less than one-half of the required penetration. All spacing and edge and end distances shall be such as to avoid splitting of the wood.

Part II—PLYWOOD STRUCTURAL PANELS

SECTION 2317 — PLYWOOD STRUCTURAL PANELS

Values for plywood structural panels shall be in accordance with Table 23-III-A.

Part III—FASTENINGS

SECTION 2318 — TIMBER CONNECTORS AND FASTENERS

2318.1 General. Timber connectors and fasteners may be used to transmit forces between wood members and between wood and metal members. Allowable design values, *Z* and *W,* shall be determined in accordance with Division III, Part I or this section. Modifications to allowable design values, and installation of timber connectors and fasteners shall be in accordance with the provisions set forth in Division III, Part I.

2318.2 Bolts. Allowable lateral design values, $Z_{\parallel}$, $Z_{m\perp}$ and $Z_{s\perp}$, in pounds for bolts in shear in seasoned lumber of Douglas fir-larch and Southern pine shall be as set forth in Tables 23-III-B-1 and 23-III-B-2.

2318.3 Nails and Spikes.

2318.3.1 Allowable lateral loads. Allowable lateral design values, *Z,* for common wire and box nails driven perpendicular to the grain of the wood, when used to fasten wood members together, shall be as set forth in Tables 23-III-C-1 and 23-III-C-2.

A wire nail driven parallel to the grain of the wood shall not be subjected to more than two thirds of the lateral load allowed when driven perpendicular to the grain. Toenails shall not be subjected to more than five sixths of the lateral load allowed for nails driven perpendicular to the grain.

In Seismic Zones 3 and 4, toenails shall not be used to transfer lateral forces in excess of 150 pounds per foot (2188 N/m) from diaphragms to shear walls, drag struts (collectors) or other elements, or from shear walls to other elements.

> **EXCEPTION:** Structures built in accordance with Section 2320.

2318.3.2 Allowable withdrawal loads. Allowable withdrawal design values, *W,* for wire nails driven perpendicular to the grain of the wood shall be as set forth in Table 23-III-D.

Nails driven parallel to the grain of the wood shall not be allowed for resisting withdrawal forces.

2318.3.3 Spacing and penetration. Common wire nails shall have penetration into the piece receiving the point as set forth in Tables 23-III-C-1 and 23-III-C-2. Nails or spikes for which the gages or lengths are not set forth in Tables 23-III-C-1 and 23-III-C-2 shall have a required penetration of not less than 11 diameters, and allowable loads may be interpolated. Allowable loads shall not be increased when the penetration of nails into the member holding the point is larger than required by this section.

2318.4 Joist Hangers and Framing Anchors. Connections depending on joist hangers or framing anchors, ties and other mechanical fastenings not otherwise covered may be used where approved.

2318.5 Miscellaneous Fasteners.

2318.5.1 Drift Bolts and Drift Pins.

2318.5.1.1 Withdrawal design values. Drift bolt and drift pin connections loaded in withdrawal shall be designed in accordance with good engineering practice.

2318.5.1.2 Lateral design values. Allowable lateral design values for drift bolts and drift pins driven in the side grain of wood shall not exceed 75 percent of the allowable lateral design values for common bolts of the same diameter and length in main member. Additional penetration of pin into members should be provided in lieu of the washer, head and nut on a common bolt.

2318.5.2 Spike Grids. Wood-to-wood connections involving spike grids for load transfer shall be designed in accordance with good engineering practice.

Part IV—ALLOWABLE STRESS DESIGN FOR WIND AND EARTHQUAKE LOADS

SECTION 2319 — WOOD SHEAR WALLS AND DIAPHRAGMS

2319.1 Conventional Lumber Diaphragms. Conventional lumber diaphragms of Douglas fir-larch or Southern pine, constructed in accordance with Section 2315.3.1, may be used to resist shear due to wind or seismic forces not exceeding 300 pounds per lineal foot (4.37 kN/m) of width. Where nails are used with sheathing and framing members with a specific gravity less than 0.49, the allowable unit shear strength of the diaphragm shall be multiplied by the following factors: 0.82 for species with specific gravity greater than or equal to 0.42 but less than 0.49, and 0.65 for species with a specific gravity less than 0.42.

2319.2 Special Lumber Diaphragms. Special diagonally sheathed diaphragms of Douglas fir-larch or Southern pine, constructed in accordance with Section 2315.3.2, may be used to resist shears due to wind or seismic loads, provided such shear do not stress the nails beyond their allowable safe lateral strength and do not exceed 600 pounds per lineal foot (8.75 kN/m) of width. Where nails are used with sheathing and framing members with a specific gravity less than 0.49, the allowable unit shear strength of the diaphragm shall be multiplied by the following factors: 0.82 for species with specific gravity greater than or equal to 0.42 but less than 0.49, and 0.65 for species with a specific gravity less than 0.42.

2319.3 Wood Structural Panel Diaphragms. Horizontal and vertical diaphragms sheathed with wood structural panels may be used to resist horizontal forces not exceeding those set forth in Table 23-II-H for horizontal diaphragms and Table 23-II-I-1 for vertical diaphragms.

Where the wood structural panel is applied to both faces of a shear wall in accordance with Table 23-II-I-1, allowable shear for the wall may be taken as twice the tabulated shear for one side, except that where the shear capacities are not equal, the allowable shear shall be either the shear for the side with the higher capacity or twice the shear for the side with the lower capacity, whichever is greater.

2319.4 Particleboard Diaphragms. Vertical diaphragms sheathed with particleboard may be used to resist horizontal forces not exceeding those set forth in Table 23-II-I-2.

2319.5 Fiberboard Sheathing Diaphragms. Wood stud walls sheathed with fiberboard sheathing may be used to resist horizontal forces not exceeding those set forth in Table 23-II-J.

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Chapter 23A *[For DSA/SS and OSHPD]*

WOOD

NOTE: This chapter has been revised in its entirety.

NOTES: [For DSA/SS] 1. This chapter is applicable to public schools, community colleges and state-owned or state-leased essential services buildings regulated by the Division of the State Architect, Structural Safety Section.

2. [For OSHPD 1 & 4] This chapter is applicable to hospitals, skilled nursing facilities, intermediate-care facilities and correctional treatment centers regulated by the Office of Statewide Health Planning and Development.

[For OSHPD 1 & 4] EXCEPTION: Single-story Type V skilled nursing or intermediate care facilities utilizing wood-frame or light-steel-frame construction as defined in Health and Safety Code Section 129725, which shall comply with UBC Chapter 23 and any applicable amendments therein.

Division I—GENERAL DESIGN REQUIREMENTS

SECTION 2301A — GENERAL

2301A.1 Scope. The quality and design of wood members and their fastenings shall conform to the provisions of this chapter.

2301A.2 Design Methods. Design shall be based on one of the following methods.

2301A.2.1 Allowable stress design. Design using allowable stress design methods shall resist the load combinations of Section 1612A.3, in accordance with the applicable requirements of Section 2305A.

2301A.2.2 Conventional light-frame construction. The design and construction of conventional light-frame wood structures shall be in accordance with the applicable requirements of Section 2305A.

SECTION 2302A — DEFINITIONS

2302A.1 Definitions. The following terms used in this chapter shall have the meanings indicated in this section:

AFPA is the American Forest and Paper Association, 1111 19th Street, N.W., Suite 800, Washington, D.C. 20036 (formerly NFoPA, National Forest Products Association).

AHA is the American Hardboard Association, Inc., 1210 W. Northwest Highway, Palatine, Illinois 60067.

AITC is the American Institute of Timber Construction, 7012 S. Revere Parkway, Suite 140, Englewood, Colorado 80112.

ALSC is the American Lumber Standard Committee, Post Office Box 210, Germantown, Maryland 20875-0210.

APA is the American Plywood Association, 7011 South 19th Street, Tacoma, Washington 98411.

AWPA is the American Wood Preservers Association, Post Office Box 286, Woodstock, Maryland 21163-0286.

BLOCKED DIAPHRAGM is a diaphragm in which all sheathing edges not occurring on framing members are supported on and connected to blocking.

BRACED WALL LINE is a series of braced wall panels in a single story that meets the requirements of Section 2320A.11.3.

BRACED WALL PANEL is a section of wall braced in accordance with Section 2320A.11.3.

CONVENTIONAL LIGHT-FRAME CONSTRUCTION is a type of construction whose primary structural elements are formed by a system of repetitive wood-framing members. Refer to Section 2320A for conventional light-frame construction provisions.

DIAPHRAGM is a horizontal or nearly horizontal system acting to transmit lateral forces to the vertical-resisting elements. When the term "diaphragm" is used, it includes horizontal bracing systems.

FIBERBOARD is a fibrous-felted, homogeneous panel made from lignocellulosic fibers (usually wood or cane) and having a density of less than 31 pounds per cubic foot (497 kg/m^3) but more than 10 pounds per cubic foot (160 kg/m^3).

GLUED BUILT-UP MEMBERS are structural elements, the sections of which are composed of built-up lumber, wood structural panels or wood structural panels in combination with lumber, all parts bonded together with adhesives.

GRADE (Lumber) is the classification of lumber in regard to strength and utility in accordance with UBC Standard 23-1 and the grading rules of an approved lumber grading agency.

HARDBOARD is a fibrous-felted, homogeneous panel made from lignocellulosic fibers consolidated under heat and pressure in a hot press to a density not less than 31 pounds per cubic foot (497 kg/m^3).

NELMA is the Northeastern Lumber Manufacturers Association, 272 Tuttle Road, Post Office Box 87 A, Cumberland Center, Maine 04021.

NLGA is the National Lumber Grades Authority, 103-4000 Dominion Street, Burnaby B.C., Canada V5G 4G3.

NSLB is the Northern Softwood Lumber Bureau (serviced by NELMA), 272 Tuttle Road, Post Office Box 87 A, Cumberland Center, Maine 04021.

NOMINAL LOADING is a design load that stresses a member of fastening to the full allowable stress tabulated in this chapter. This loading may be applied for approximately 10 years, either continuously or cumulatively, and 90 percent of this load may be applied for the remainder of the life of the member or fastening.

NOMINAL SIZE (Lumber) is the commercial size designation of width and depth, in standard sawn lumber and glued-laminated lumber grades; somewhat larger than the standard net size of dressed lumber, in accordance with UBC Standard 23-1 for sawn lumber.

PARTICLEBOARD is a manufactured panel product consisting of particles of wood or combinations of wood particles and wood fibers bonded together with synthetic resins or other suitable bonding system by a bonding process in accordance with approved nationally recognized standards.

PLYWOOD is a panel of laminated veneers conforming to UBC Standard 23-2. *

RIS is the Redwood Inspection Service, 405 Enfrente Drive, Suite 200, Novato, California 94949.

ROTATION is the torsional movement of a diaphragm about a vertical axis.

SPIB is the Southern Pine Inspection Bureau, 4709 Scenic Highway, Pensacola, Florida 32504.

STRUCTURAL GLUED-LAMINATED TIMBER is any member comprising an assembly of laminations of lumber in which the grain of all laminations is approximately parallel longitudinally, in which the laminations are bonded with adhesives.

SUBDIAPHRAGM is a portion of a larger wood diaphragm designed to anchor and transfer local forces to primary diaphragm struts and the main diaphragm.

TREATED WOOD is wood treated with an approved preservative under treating and quality control procedures.

WCLIB is the West Coast Lumber Inspection Bureau, 6980 S.W. Varnes Road, Post Office Box 23145, Portland, Oregon 97223.

WOOD OF NATURAL RESISTANCE TO DECAY OR TERMITES is the heartwood of the species set forth below. Corner sapwood is permitted on 5 percent of the pieces provided 90 percent or more of the width of each side on which it occurs is heartwood. Recognized species are:

Decay resistant: Redwood, Cedars, Black Locust

Termite resistant: Redwood, Eastern Red Cedar

WOOD STRUCTURAL PANEL is a *plywood* structural panel product composed * * * of wood and meeting the requirements of UBC Standard 23-2 * * *. Wood structural panels *are* all-veneer plywood.

WWPA is the Western Wood Products Association, Yeon Building, 522 S. W. Fifth Avenue, Portland, Oregon 97204-2122.

SECTION 2303A — STANDARDS OF QUALITY

The standards listed below labeled a "UBC Standard" are also listed in Chapter 35, Part II, and are part of this code. The other standards listed below are *also part of this code*. (See Sections 3503 and 3504.)

1. **Grading rules.**
   - 1.1 UBC Standard 23-1, Classification, Definition, Methods of Grading and Development of Design Values for All Species of Lumber
   - 1.2 Standard Grading Rules for Canadian Lumber, United States Edition, NLGA
   - 1.3 Standard Grading Rules No. 17, WCLIB
   - 1.4 Standard Grading Rules, WWPA
   - 1.5 *Not adopted by DSA.*
   - 1.6 Grading Rules, SPIB
   - 1.7 Standard Specifications for Grades of California Redwood Lumber, RIS
   - 1.8 *Not adopted by DSA.*
2. **Structural glued-laminated timber.**
   - 2.1 ANSI/AITC Standard A190.1 and ASTM D 3737, Design and Manufacture of Structural Glued-laminated Timber
   - 2.2 Standard Specifications for Structural Glued-laminated Timber of Softwood Species, AITC 117; Manufacturing, AITC 117; Design and Standard Specifications for Hardwood Glued-laminated Timber, AITC 119.
   - 2.3 Inspection Manual AITC 200 of the American Institute of Timber Construction, Tests for Structural Glued-laminated Timber.
   - 2.4 AITC 500, Determination of Design Values for Structural Glued-laminated Timber in accordance with ASTM D 3737, American Institute of Timber Construction.
3. **Preservative treatment by pressure process and quality control.**
   - 3.1 Standard Specifications C1, C2, C3, C4, C9, C14, C15, C16, C22, C23, C24, C28 and M4, AWPA
4. **Product standards.**
   - 4.1 UBC Standard 23-2, Construction and Industrial Plywood
   - 4.2 *Not adopted by DSA, OSHPD.*
   - 4.3 ANSI A208.1, Particleboard
   - 4.4 ASTM D 1037, Evaluating the Properties of Wood-based Fiber and Particle Panel Materials
   - 4.5 ASTM D 1333, Determining Formaldehyde Levels from Wood-based Products Under Defined Test Conditions Using a Large Chamber
   - 4.6 ANSI 05.1, Wood Poles—Specifications and Dimensions
   - 4.7 ASTM D 25, Round Timber Piles
   - 4.8 ANSI/AHA A194.1, Cellulosic Fiber Insulating Board (Fiberboard)
   - 4.9 ANSI/AHA 135.6, Hardboard Siding
5. **Design standards.**
   - 5.1 ASTM D 5055, Structural Capacities of Prefabricated Wood I-Joists
   - 5.2 ANSI/TPI 1 National Design Standard for Metal Plate Connected Wood Truss Construction
   - 5.3 ANSI/TPI 2 Standard for Testing Performance for Metal Plate Connected Wood Trusses
6. **Fire retardancy.**
   - 6.1 UBC Standard 23-4, Fire-retardant-treated Wood Tests on Durability and Hygroscopic Properties
   - 6.2 UBC Standard 23-5, Fire-retardant-treated Wood
7. **Adhesives and glues.**
   - 7.1 ASTM D 3024, Dry Use Adhesive with Protein Base, Casein Type
   - 7.2 ASTM D 2559, Wet Use Adhesives
   - 7.3 APA Specification AFG-01, Adhesives for Field Gluing Plywood to Wood Framing
   - 7.4 ASTM D 1101 and AITC 200 in Testing of Glue Joints in Laminated Wood Product
8. **Design values.**
   - 8.1 ASTM D 1990, Establishing Allowable Properties for Visually-Graded Dimension Lumber from In-Grade Tests of Full-Size Specimens
   - 8.2 ASTM D 245, Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber
   - 8.3 ASTM D 2555, Standard Test Methods for Establishing Clear Wood Strength Values

SECTION 2304A — MINIMUM QUALITY

2304A.1 Quality and Identification. All lumber, *plywood,* structural glued-laminated timber, end-jointed lumber, * * * piles and poles regulated by this chapter shall conform to the applicable standards and grading rules specified in this code and shall be so identified by the grade mark or certificate of inspection issued by an approved agency.

All preservatively treated wood required to be treated under Section 2306A shall be identified by the quality mark of an inspec-

Division III—DESIGN SPECIFICATIONS FOR ALLOWABLE STRESS DESIGN OF WOOD BUILDINGS

Part I—ALLOWABLE STRESS DESIGN OF WOOD

This standard, with certain exceptions, is the *ANSI/AF&PA NDS-2001* National Design Specification for Wood Construction of the American Forest and Paper Association and the Supplement to the *2001* Edition, National Design Specification, adopted by reference.

The National Design Specification for Wood Construction, *2001* Edition, and *2001* Supplement are available from the American Forest and Paper Association, 1111 19th Street, NW, Eighth Floor, Washington, DC, 20036.

SECTION 2316A — DESIGN SPECIFICATIONS

2316A.1 Adoption and Scope. The National Design Specification for Wood Construction, *2001* Edition (NDS), which is hereby adopted as a part of this code, shall apply to the *allowable stress* design and construction of wood structures using visually graded lumber, mechanically graded lumber, structural glued laminated timber, and timber piles.

Where a standard or specification referred to in this code conflicts with a standard or specification referenced in the 2001 NDS for wood construction, the 2001 NDS shall prevail.

2316A.2 Amendments.

***1.* Sec. 2.3.2.1. In fourth sentence, delete "or Figure B1 (see Appendix B)."**

***2.* Sec. 2.3.2.3. Delete and substitute the following:**

2.3.2.3 When using Section 1612A.3.1 basic load combinations, the Load Duration Factor, C_D, *except for dead load,* noted in Table 2.3.2 shall *NOT* be permitted to be used. When using Section 1612A.3.2 alternate load combinations, the one-third increase shall not be used concurrently with the Load Duration Factor, C_D.

***3.* Table 2.3.2. Delete and substitute as follows:**

TABLE 2.3.2—LOAD DURATION FACTORS, C_D

DESIGN LOAD	LOAD DURATION	C_D
Dead Load	Permanent	0.9
Floor, Occupancy Live Load	Ten Years	1.0
Snow Load	Two Months	1.15
Roof Live Load[3]	Seven Days	1.25
Earthquake Load[1]	—	1.33
Wind Load[2]	—	1.33
Impact	—	2.0

[1]1.60 may be used for nailed and bolted connections exhibiting Mode III or IV behavior, except that the increases for earthquake are not combined with the increase allowed in Section 1612A.3. The 60-percent increase for nailed and bolted connections exhibiting Mode III or IV behavior for earthquake shall not be applicable to joist hangers, framing anchors, and other mechanical fastenings, including straps and hold-down anchors. The 60-percent increase shall not apply to the allowable shear values in Tables 23A-II-H, 23A-II-I-1, 23A-II-I-2, 23A-II-J or in Section 2315A.3.

[2]1.60 may be used for members and nailed and bolted connections exhibiting Mode III or IV behavior, except that the increases for wind are not combined with the increase allowed in Section 1612A.3. The 60-percent increase shall not apply to the allowable shear values in Tables 23A-II-H, 23A-II-I-1, 23A-II-I-2, 23A-II-J or in Section 2315A.3.

[3]*Provided the dead load includes the weight of at least one reroofing.*

***4.* Sec. 2.3.3. Add a second paragraph following Table 2.3.3:**

The allowable unit stresses for fire-retardant-treated solid-sawn lumber and plywood, including fastener values, subject to prolonged elevated temperatures from manufacturing or equipment processes, but not exceeding 150°F (66°C), shall be developed from approved test methods that properly consider potential strength-reduction characteristics, including effects of heat and moisture.

***5.* Sec. 2.3.4. Add second, third, *fourth and fifth* paragraphs as follows:**

The values for lumber and plywood impregnated with approved fire-retardant chemicals, including fastener values, shall be submitted to the *enforcement agency* for approval. Submittal to the *enforcement agency* shall include all substantiating data. Such values shall be developed from approved test methods and procedures that consider potential strength-reduction characteristics, including the effects of elevated temperatures and moisture. Other adjustments are applicable, except that the impact load-duration factor shall not apply.

The values for fasteners specified in Division III shall be reduced to 90 percent, except that values for light metal plate connectors shall be recommended by each truss plate manufacturer and approved by the enforcement agency.

Values for glued-laminated timber, including fastener design values, shall be recommended by the treater and submitted to the *enforcement agency* for approval. Submittal to the *enforcement agency* shall include all substantiating data.

In addition to the requirements specified in Section 207, fire-retardant lumber having structural applications shall be tested and identified by an approved inspection agency in accordance with UBC Standard 23-5.

6. Sec. 4.4.1. Add a section as follows:

4.4.1.4 Bridging for Floor Joists and Roof Joists or Rafters. *Roof joists or rafters of more than 8-inch (203 mm) depth and floor joists of more than 4-inch (102 mm) depth which are spaced 32 inches (813 mm) on center or less shall be provided with bridging to distribute superimposed loads. Floor joists shall be bridged every 8 feet (2438 mm) and roof joists or rafters every 10 feet (3048 mm) by solid blocking 2 inches (51 mm) thick and the full depth of the joist or rafter, or by wood cross bridging of not less than 1 inch by 3 inches (25 mm by 76 mm) or nailed metal cross bridging of equal strength. Where cross bridging is used, the lower ends of such cross bridging shall be driven up and nailed after the floor, subfloor or roof has been nailed.*

7. Sec. 5.4. Add a new section as follows:

***5.4.5* Ponding.** Roof-framing members shall be designed for the deflection and drainage or ponding requirements specified in Section 1506 and Chapter 16A. In glued-laminated timbers, the minimum slope for roof drainage required by Section 1506 shall be in addition to a camber of one and one-half times the calculated dead load deflection. The calculation of the required slope shall not include any vertical displacement created by short taper cuts. In no case shall the deflection of glued-laminated timber roof members exceed $^1/_2$-inch (13 mm) for a 5 pound-per-square-foot (239 Pa) uniform load.

***8. Sec. 5.4.* Add a new section as follows:**

***5.4.6* Tapered Faces.** Sawn tapered cuts shall not be permitted on the tension face of any beam. Pitched or curved beams shall be so fabricated that the laminations are parallel to the tension face. Straight, pitched or curved beams may have sawn tapered cuts on the compression face.

For other members subject to bending, the slope of tapered faces, measured from the tangent to the lamination of the section under consideration, shall not be steeper than 1 unit vertical in 24 units horizontal (4% slope) on the tension side.

EXCEPTIONS: 1. This requirement does not apply to arches.

2. Taper may be steeper at sections increased in size beyond design requirements for architectural projections.

9. Sec. 5.4. Add a new section as follows:

5.4.7 Manufacture and Fabrication. *The manufacture and fabrication of structural glued-laminated timber shall be in accordance with ANSI/AITC A 190.1 and the following requirements:*

*1. **Joints.** All portions of end joints in adjacent laminations shall be separated in accordance with ANSI/AITC A 190.1 and ASTM D 3737. The areas requiring 6-inch (152 mm) spacing shall be shown on the approved drawings or described in the specifications.*

Joints in adjacent laminations of arched members shall be separated as required for bending members.

*2. **Adhesives.** Dry-use adhesives shall not be used.*

*3. **Moisture content at the time of gluing.** The maximum moisture content of the laminating lumber at the time of gluing shall not exceed 16 percent for projects located in coastal areas, 12 percent for projects located in interior valleys or desert areas, with the geographical areas as determined by the enforcement agency. The moisture content of the wood for members that will be exposed to direct sunlight in the finished structure shall not exceed 12 percent at time of gluing:*

The range of moisture content of laminations assembled into a single member shall not exceed 5 percent at the time of gluing.

When mechanical reinforcing is used, such as radial tension reinforcement, the maximum moisture content of the laminations at time of manufacture shall not exceed 12 percent for dry conditions of use.

*4. **Inspection.** See Section 2337A for inspection requirements.*

10. Sec. 5.4. Add a new section as follows:

5.4.8 Specifications. *For structural glued-laminated timber, the following shall be shown on the plans and in the specifications:*

Whether for dry or wet conditions of use

Species and applicable standard

Stress requirements and combination symbol

If the temperature of the timber exceeds 150°F (66°C) in service

Tension zones for purposes of determining grades of laminations and location of spaced end joints for all members except simple beams supporting uniform loads.

Those portions of glued-laminated timbers which form the structural supports of a building or other structure and are exposed to weather and not properly protected by a roof, eave overhangs or similar covering shall be pressure treated with an approved preservative or be manufactured from wood of natural resistance to decay.

All weather-exposed surfaces of members shall be protected in an approved manner to prevent decay where they are located in a high-humidity environment such as in direct contact with soil or water and where portions extend beyond the walls and roof coverage in buildings. When the member is protected with an approved pressure treatment, the treatment process shall not impair the structural integrity of the member. When the member is protected by flashing or is encased, care must be taken to provide ventilation and prevent moisture entrapment on the member.

All members shall have appropriate weather protection during transit, storage and erection.

11. Sec. 11.5.4. Delete.

12. Sec. 11.1.5.6 **Delete and substitute as follows:**

11.1.5.6 For wood-to-wood joints, the spacing center to center of nails in the direction of stress shall not be less than the required penetration. Edge or end distances in the direction of stress shall not be less than one-half of the required penetration. All spacing and edge and end distances shall be such as to avoid splitting of the wood.

(Pages 2-378.21 and 2-378.22 have beeen deleted. Text continues on page 2-278.23.)

Part II—PLYWOOD STRUCTURAL PANELS

SECTION 2317A — PLYWOOD STRUCTURAL PANELS

Values for plywood structural panels shall be in accordance with Table 23A-III-A.

Part III—FASTENINGS

SECTION 2318A — TIMBER CONNECTORS AND FASTENERS

2318A.1 General. Timber connectors and fasteners may be used to transmit forces between wood members and between wood and metal members. Allowable design values, *Z* and *W,* shall be determined in accordance with Division III, Part I or this section. Modifications to allowable design values, and installation of timber connectors and fasteners shall be in accordance with the provisions set forth in Division III, Part I.

2318A.2 Bolts. Allowable lateral design values, $Z_{\|}$, $Z_{m\perp}$ and $Z_{s\perp}$, in pounds for bolts in shear in seasoned lumber of Douglas fir-larch and Southern pine shall be as set forth in Tables 23A-III-B-1 and 23A-III-B-2.

Carriage bolts without washers under heads may be used in shear for loads not exceeding two thirds the tabulated bolt values for standard machine bolts.

Wood-framing members with bolted connections shall be supported on adequate bearing surfaces to resist the applied loadings. Cross grain shrinkage must be considered in bolted connections with steel side plates.

2318A.3 Nails and Spikes.

2318A.3.1 Allowable lateral loads. Allowable lateral design values, *Z,* for common wire and box nails driven perpendicular to the grain of the wood, when used to fasten wood members together, shall be as set forth in Tables 23A-III-C-1 and 23A-III-C-2.

The allowable load on casing nails shall not exceed one half that allowed for common nails.

Common wire nails or spikes driven parallel to the grain of the wood or installed as toenails shall not be subjected to more than one half of the lateral load allowed when driven perpendicular to the grain. When toenails are driven in subdrilled pilot holes, a value of two thirds of the allowable lateral load allowed for nails driven perpendicular to the grain may be used. Pilot holes shall have a diameter approximately 90 percent of the nail shank diameter.

Toenails shall not be used to transfer lateral forces in excess of 150 pounds per foot (2190 N/m) from diaphragms to shear walls, drag struts (collectors) or other elements, nor from shear walls to diaphragms or other elements of public elementary and secondary schools, community college buildings, and state-owned or state-leased essential services buildings.

In Seismic Zones 3 and 4, toenails shall not be used to transfer lateral forces in excess of 150 pounds per foot (2188 N/m) from diaphragms to shear walls, drag struts (collectors) or other elements, or from shear walls to other elements.

> **EXCEPTION:** Structures built in accordance with Section 2320A.

2318A.3.2 Allowable withdrawal loads. Allowable withdrawal design values, *W,* for wire nails driven perpendicular to the grain of the wood shall be as set forth in Table 23A-III-D.

The use of nails driven perpendicular to the grain to resist loads in withdrawal shall be limited to connections using not more than four nails in a single connection.

Nails driven parallel to the grain of the wood shall not be allowed for resisting withdrawal forces.

Toenails shall not be permitted to resist loads in withdrawal.

2318A.3.3 Spacing and penetration. Common wire nails shall have penetration into the piece receiving the point as set forth in Tables 23A-III-C-1 and 23A-III-C-2. Nails or spikes for which the gages or lengths are not set forth in Tables 23A-III-C-1 and 23A-III-C-2 shall have a required penetration of not less than 11 diameters, and allowable loads may be interpolated. Allowable loads shall not be increased when the penetration of nails into the member holding the point is larger than required by this section. *Spacing shall be in accordance with Section 2316A.2, Item 12.*

Common wire 10d, 12d and 16 d nails may be used to join two members of 2-inch (51 mm) nominal thickness at the tabulated values indicated for these nails.

Nails in plywood shall not be overdriven such that the nail heads penetrate the face ply by more than the thickness of the nail head or break the face-ply wood fibers.

2318A.3.4 [For DSA/SS, OSHPD] Corrosion resistance. *Nails and spikes used in wet or exterior locations, such as exterior wall coverings of hospitals, public elementary and secondary schools, community college buildings, and state-owned or state-leased essential services buildings, shall be corrosion resistant and shall have a hot-dipped or tumbled galvanized coating of not less than 1.0 ounces of zinc per square foot (305 gm/m²) or be fabricated of copper, stainless steel or brass.*

2318A.4 Joist Hangers and Framing Anchors. Connections depending on joist hangers or framing anchors, ties and other mechanical fastenings not otherwise covered may be used where approved.

2318A.5 Miscellaneous Fasteners.

2318A.5.1 Drift Bolts and Drift Pins.

2318A.5.1.1 Withdrawal design values. Drift bolt and drift pin connections loaded in withdrawal shall be designed in accordance with good engineering practice.

2318A.5.1.2 Lateral design values. Allowable lateral design values for drift bolts and drift pins driven in the side grain of wood shall not exceed 75 percent of the allowable lateral design values for common bolts of the same diameter and length in main member. Additional penetration of pin into members should be provided in lieu of the washer, head and nut on a common bolt.

2318A.5.2 Spike Grids. Wood-to-wood connections involving spike grids for load transfer shall be designed in accordance with good engineering practice.

2318A.6 Wood Screws and Lag Screws.

2318A.6.1 *The use of wood screws and lag screws to resist continuously applied loads in withdrawal shall be limited to connections using not more than four screws in a single connection and at loads not exceeding 50 percent of the allowable values set forth in Division III.*

Washers shall be provided under heads of lag screws which bear on wood.

2318A.7 Metal Plate Connectors. *The design and fabrication of light metal plate connected light wood trusses shall be in accordance with Division III, Part III, and in accordance with the following requirements:*

*1. **Joint designs.** The effects of eccentric loading shall be considered in the design of all joints and complete calculations for each joint shall be submitted.*

Metal plate connector teeth located within $^1/_2$ inch (12.7 mm) of the edge or end of a wood member shall not be considered effective in carrying load. Teeth shall not be installed in knots or greatly distorted grain. An 8d by $1^1/_2$-inch-long (38 mm) ring-shanked nail shall be installed through each plate to each member.

*2. **Basic code value.** The basic normal load value per tooth, plug, nail or square inch of connector plate area shall be the least of the values determined by the following criteria:*

2.1 The average uncycled proportional limit slip divided by 2.0;

2.2 The average cycled proportional limit slip divided by 1.6;

2.3 The average uncycled ultimate load at failure divided by 5.0;

2.4 The minimum uncycled ultimate load at failure divided by 4.0;

2.5 The average cycled ultimate load at failure divided by 4.0; or

2.6 The minimum cycled ultimate load at failure divided by 3.2.

*3. **Inspection.** See Section 2337A.3 for inspection requirements.*

*4. **Truss loads.** The truss loads bearing on beams or top plates of walls shall not exceed the allowable bearing and bending stresses for those members.*

*5. **Complete truss drawings.** Complete drawings of truss configurations and all joint details, including the details for the support of the ends of the truss shall be shown on the drawings submitted for approval.*

*6. **Moisture content.** All lumber used in trusses shall have a moisture content of not less than 11 percent or more than 19 percent at the time of fabrication.*

Part IV—ALLOWABLE STRESS DESIGN FOR WIND AND EARTHQUAKE LOADS

SECTION 2319A — WOOD SHEAR WALLS AND DIAPHRAGMS

2319A.1 Conventional Lumber Diaphragms. Conventional lumber diaphragms *with common wire nails in* Douglas fir-larch or Southern pine, * * * may be used to resist shear due to wind or seismic forces not exceeding *200 pounds* per lineal foot *(2.92 kN/m) for horizontal diaphragms, 250 pounds per lineal foot (3.65 kN/m) for vertical diaphragms with a maximum height-to-width ratio of 1, and 300 pounds per lineal foot (4.37 kN/m) for vertical diaphragms with a maximum ratio of 1 to 1.5. Where box nails are used, one additional nail shall be used at each bearing and end connection.*

Wood diaphragms made up of 2-inch by 6-inch (51 mm by 152 mm) diagonal sheathing with 16d nails may be used at the same shear values and in the same locations as 1-inch (25 mm) sheathing, provided there are no splices in adjacent boards on the same support and supports are at least 4 inches (102 mm) in width and at least 3 inches (76 mm) thick.

Where nails are used with sheathing and framing members with a specific gravity less than 0.49, the allowable unit shear strength of the diaphragm shall be multiplied by the following factors: 0.82 for species with specific gravity greater than or equal to 0.42 but less than 0.49, and 0.65 for species with a specific gravity less than 0.42.

2319A.2 Special Lumber Diaphragms. Special diagonally sheathed diaphragms of Douglas fir-larch or Southern pine, constructed in accordance with Section 2315A.3.2, may be used to resist shears due to wind or seismic loads, provided such shear do not stress the nails beyond their allowable safe lateral strength and do not exceed 600 pounds per lineal foot (8.75 kN/m) of width. Where nails are used with sheathing and framing members with a specific gravity less than 0.49, the allowable unit shear strength of the diaphragm shall be multiplied by the following factors: 0.82 for species with specific gravity greater than or equal to 0.42 but less than 0.49, and 0.65 for species with a specific gravity less than 0.42.

2319A.3 *Plywood* Diaphragms. Horizontal and vertical diaphragms sheathed with wood structural panels may be used to resist horizontal forces not exceeding those set forth in Table 23A-II-H for horizontal diaphragms and Table 23A-II-I-1 for vertical diaphragms, *and in accordance with Section 2315A.3.3.*

Where the wood structural panel is applied to both faces of a shear wall in accordance with Table 23A-II-I-1, allowable shear for the wall may be taken as twice the tabulated shear for one side, except that where the shear capacities are not equal, the allowable shear shall be either the shear for the side with the higher capacity or twice the shear for the side with the lower capacity, whichever is greater.

2319A.4 ***Not adopted by DSA, OSHPD.***

2319A.5 ***Not adopted by DSA, OSHPD.***

Division IV—CONVENTIONAL LIGHT-FRAME CONSTRUCTION

SECTION 2320A — CONVENTIONAL LIGHT-FRAME CONSTRUCTION DESIGN PROVISIONS

2320A.1 General. The requirements *contained* in this section are intended for conventional light-frame construction ∗ ∗ ∗ *and are in addition to the requirements contained in other sections of this chapter.*

2320A.2 ***Not adopted by DSA, OSHPD.***

2320A.3 ***Not adopted by DSA, OSHPD.***

2320A.4 ***Not adopted by DSA, OSHPD.***

2320A.5 ***Not adopted by DSA, OSHPD.***

2320A.5.1 ***Not adopted by DSA, OSHPD.***

2320A.5.2 ***Not adopted by DSA, OSHPD.***

2320A.5.3 ***Not adopted by DSA, OSHPD.***

2320A.5.4 ***Not adopted by DSA, OSHPD.***

2320A.5.5 ***Not adopted by DSA, OSHPD.***

2320A.5.6 Interior braced wall support. In one-story buildings, interior braced wall lines shall be supported on continuous foundations at intervals not exceeding 50 feet (15 240 mm). In buildings more than one story in height, all interior braced wall panels shall be supported on continuous foundations.

2320A.6 Foundation Plates or Sills. *Sills under bearing, exterior or shear walls shall be bolted to the masonry or concrete with not smaller than $^5/_8$-inch by 12-inch (16 mm by 305 mm) bolts spaced not more than 4 feet (1219 mm) on center with a minimum of two bolts for each piece of sill plate. There shall be a bolt within 9 inches (229 mm) of each end of each piece of sill. The effective embedment length in concrete of bolts designed to resist overturning or uplift forces shall not include the embedment in 6-inch (152 mm) or narrower concrete curbs. Where sills are bored or notched exceeding one third the sill width, extra bolts shall be required as given for ends of sill pieces. All sills shall bear on a level and competent concrete or masonry supporting surface.*

Sills under nonbearing interior partitions on concrete floor slabs shall be anchored at not more than 4 feet (1219 mm) on center to resist a shear of not less than 50 pounds per linear foot (0.7 kN/m) acting either parallel or normal to the wall.

The nominal thickness of the sills shall not be less than 3 inches (76 mm) when the wall is diagonally sheathed, and not less than 2 inches (51 mm) in any event.

Treated wood sills where cut, drilled or notched shall be treated with a preservative and approved by the architect and the enforcement agency on all exposed surfaces from which preservative treatment has been removed.

2320A.7 Girders. Girders for single-story construction or girders supporting loads from a single floor shall not be less than 4 inches by 6 inches (102 mm by 153 mm) for spans 6 feet (1829 mm) or less, provided that girders are spaced not more than 8 feet (2438 mm) on center. Other girders shall be designed to support the loads specified in this code. Girder end joints shall occur over supports. When a girder is spliced over a support, an adequate tie shall be provided. The end of beams or girders supported on masonry or concrete shall not have less than 3 inches (76 mm) of bearing.

2320A.8 Floor Joists.

2320A.8.1 General. *Floor joists shall be designed in accordance with the general provisions of these rules and regulations.*

2320A.8.2 Bearing. Except where supported on a 1-inch by 4-inch (25 mm by 102 mm) ribbon strip and nailed to the adjoining stud, the ends of each joist shall not have less than $1^1/_2$ inches (38 mm) of bearing on wood or metal, or less than 3 inches (76 mm) on masonry.

2320A.8.3 Framing details. Joists shall be supported laterally at the ends and at each support by solid blocking except where the ends of joists are nailed to a header, band or rim joist or to an adjoining stud or by other approved means. Solid blocking shall not be less than 2 inches (51 mm) in thickness and the full depth of joist.

Notches on the ends of joists shall not exceed one fourth the joist depth. Holes bored in joists shall not be within 2 inches (51 mm) of the top or bottom of the joist, and the diameter of any such hole shall not exceed one third the depth of the joist. Notches in the top or bottom of joists shall not exceed one *sixth* the depth and shall not be located in the middle third of the span. *Notches or holes shall not be placed in joists unless fully detailed in the approved plans.*

Joist framing from opposite sides of a beam, girder or partition shall be lapped at least 3 inches (76 mm) or the opposing joists shall be tied together in an approved manner.

Ledger strips applied to the sides of girders for support of joists shall not be less than 2 inches by 4 inches (51 mm by 102 mm).

2320A.8.4 Framing around openings. Trimmer and header joists shall be doubled, or of lumber of equivalent cross section, when the span of the header exceeds 4 feet (1219 mm). The ends of header joists more than 6 feet (1829 mm) long shall be supported by framing anchors or joist hangers unless bearing on a beam, partition or wall. Tail joists over 12 feet (3658 mm) long shall be supported at header by framing anchors or on ledger strips not less than 2 inches by 2 inches (51 mm by 51 mm).

2320A.8.5 Supporting bearing partitions. Bearing partitions perpendicular to joists shall not be offset from supporting girders, walls or partitions more than the joist depth.

Joists under and parallel to bearing partitions shall be doubled *and nailed or bolted as set forth for built-up beams in Table 23A-II-B-1, or may be separated by solid blocking of 2-inch-minimum (51 mm) nominal thickness spaced at not more than 4-foot (1219 mm) intervals.*

2320A.8.6 Blocking. Floor joists shall be blocked when required by the provisions of Division III, Part I or Section 2320A.8.3.

2320A.8.7 Bridging. *Floor joists more than 4 inches (102 mm) in depth shall be provided with bridging in accordance with the provisions of Section 2316A.2, Item 6.*

2320A.9 Subflooring.

2320A.9.1 ***Not adopted by DSA, OSHPD.***

2320A.9.2 ***Plywood.*** Where used as structural subflooring, wood structural panels shall be as set forth in Table 23A-II-E-1. Wood structural panel combination subfloor underlayment shall have maximum spans as set forth in Table 23A-II-F-1.

When wood structural panel floors are glued to joists with an adhesive in accordance with the adhesive manufacturer's directions, fasteners may be spaced a maximum of 12 inches (305 mm) on center at all supports.

2320A.9.3 Plank flooring. Plank flooring shall be designed in accordance with the general provisions of this code.

2320A.9.4 *Not adopted by DSA, OSHPD.*

2320A.10 Particleboard Underlayment. In accordance with approved recognized standards, particleboard floor underlayment shall conform to Type PBU. Underlayment shall not be less than $^{1}/_{4}$ inch (6.4 mm) in thickness and shall be identified by the grade mark of an approved inspection agency. Underlayment shall be installed in accordance with this code and as recommended by the manufacturer.

2320A.11 Wall Framing.

2320A.11.1 Size, height and spacing. * * *

2320A.11.1.1 Size. *Studs in exterior walls and interior bearing walls of buildings not more than two stories in height shall be not less than 2 inches by 4 inches (51 mm by 102 mm) in size. See Section 2308A. Interior nonbearing partitions may be framed with 2-inch by 3-inch (51 mm by 76 mm) studs.*

2320A.11.1.2 Height. *Unless supported laterally by adequate framing, the maximum allowable height for studs shall be 10 feet (3048 mm) for 2-inch by 3-inch (51 mm by 76 mm) studs, 14 feet (4267 mm) for 2-inch by 4-inch (51 mm by 102 mm) and 3-inch by 4-inch (76 mm by 102 mm) studs, and 20 feet (6096 mm) for 2-inch by 6-inch (51 mm by 152 mm) studs.*

When approved for use by the enforcement agency, the maximum allowable height for utility studs shall be 8 feet (2438 mm) for load-bearing and for exterior wall studs, and 10 feet (3048 mm) for interior nonload-bearing studs. When used in bearing walls, utility studs shall support not more than a roof and ceiling load.

2320A.11.1.3 Spacing. *Studs supporting floors shall be spaced not more than 16 inches (406 mm) on center.*

2320A.11.2 Framing details. Studs shall be placed with their wide dimension perpendicular to the wall. Not less than three studs shall be installed at each corner of an exterior wall.

Openings in stud walls or partitions shall have headers and a minimum of two studs at jambs, one stud of which may be cut to support the header in bearing.

NOTE: *See Section 2320A.11.6.*

Where wood and masonry or concrete walls intersect, the end stud shall be fastened with bolts or other devices at top, bottom and midheight, with at least the equivalent of one bolt of $^{1}/_{2}$-inch (13 mm) diameter passing through the end stud and embedded in the masonry or concrete a minimum of 4 inches (102 mm). If the wall intersection is calculated to transfer loads greater than the capacity of the above requirement, the fastening shall be completely detailed on the approved drawings. All studs shall be capped with double top plates, except at interior nonstructural partitions. End joints in double top plates shall be offset at least 48 inches (1219 mm).

When bearing studs are spaced at 24-inch (610 mm) intervals and top plates are less than two 2-inch by 6-inch (51 mm by 152 mm) or two 3-inch by 4-inch (76 mm by 102 mm) members and when the * * * roof trusses which they support are spaced at more than 16-inch (406 mm) intervals, such joists or trusses shall bear within 5 inches (127 mm) of the studs beneath or a third plate shall be installed.

Interior nonbearing partitions *not designed to resist shear* may be capped with a single top plate installed to provide overlapping at corners and at intersections with other walls and partitions. The plate shall be continuously tied at joints by solid blocking at least 16 inches (406 mm) in length and equal in size to the plate or by $^{1}/_{8}$-inch by $1^{1}/_{2}$-inch (3.2 mm by 38 mm) metal ties with spliced sections fastened with two 16d nails on each side of the joint.

Studs shall have full bearing on a plate or sill not less than 2 inches (51 mm) in thickness having a width not less than that of the wall studs.

2320A.11.3 Bracing. *Bracing for walls which are part of the lateral load-resisting system shall be designed in accordance with the general provisions of these rules and regulations.*

All exterior walls and main cross-stud partitions which are not part of the lateral load-resisting system shall be effectively and thoroughly braced at each end, or as near thereto as possible, and as otherwise necessary to resist wind and seismic forces by one of the following methods:

1. Nominal 1-inch by 4-inch (25 mm by 102 mm) continuous diagonal braces let into top and bottom plates and intervening studs, placed at an angle not more than 60 degrees or less than 45 degrees from the horizontal, and attached to the framing in conformance with Table 23A-II-B-1.

2. Wood boards of $^{5}/_{8}$-inch (16 mm) net minimum thickness applied diagonally on studs spaced not over 24 inches (610 mm) on center.

3. Wood structural panel sheathing with a thickness not less than $^{5}/_{16}$ inch (7.9 mm) for 16-inch (406 mm) stud spacing and not less than $^{3}/_{8}$ inch (9.5 mm) for 24-inch (610 mm) stud spacing in accordance with Tables 23A-II-A-1 and 23A-IV-D-1.

4. Fiberboard sheathing 4-foot by 8-foot (1219 mm by 2438 mm) panels not less than $^{1}/_{2}$ inch (13 mm) thick applied vertically on studs spaced not over 16 inches (406 mm) on center. * * *

5. Gypsum board [sheathing $^{1}/_{2}$ inch (13 mm) thick by 4 feet (1219 mm) wide, wallboard or veneer base] on studs spaced not over 24 inches (610 mm) on center and nailed at 7 inches (178 mm) on center with nails as required by Table 25-I.

6. Particleboard wall sheathing panels where installed in accordance with Table 23A-IV-D-2.

7. Portland cement plaster on studs spaced 16 inches (406 mm) on center installed in accordance with Table 25-I.

8. Hardboard panel siding when installed in accordance with Section 2310A.6 and Table 23A-II-C.

Method 1 is not permitted in Seismic Zones 2B, 3 and 4. For cripple wall bracing, see Section 2320A.11.5. For Methods 2, 3, * * * 6 * * * and 8, each braced panel must be at least 48 inches (1219 mm) in length, covering three stud spaces where studs are spaced 16 inches (406 mm) apart and covering two stud spaces where studs are spaced 24 inches (610 mm) apart.

For Methods *4, 5 and 7*, each braced wall panel must be at least 96 inches (2438 mm) in length when applied to one face of a braced wall panel and 48 inches (1219 mm) when applied to both faces.

All vertical joints of panel sheathing shall occur over studs. Horizontal joints shall occur over blocking equal in size to the studding except where waived by the installation requirements for the specific sheathing materials.

Braced wall panel sole plates shall be nailed to the floor framing and top plates shall be connected to the framing above in accordance with Table 23A-II-B-1. Sills shall be bolted to the foundation or slab in accordance with Section 1806A.6. Where joists are perpendicular to braced wall lines above, blocking shall be provided under and in line with the braced wall panels.

2320A.11.4 *Not adopted by DSA, OSHPD.*

2320A.11.5 Cripple walls. Foundation cripple walls shall be framed of studs not less in size than the studding above with a minimum length of 14 inches (356 mm), or shall be framed of solid blocking. When exceeding 4 feet (1219 mm) in height, such walls shall be framed of studs having the size required for an additional story.

Cripple walls having a stud height exceeding 14 inches (356 mm) shall be *considered to be first-story walls for the purpose of determining the bracing required by Section 2320A.11.3*. Solid blocking or wood structural panel sheathing may be used to brace cripple walls having a stud height of 14 inches (356 mm) or less. In Seismic Zone 4, Method 7 is not permitted for bracing any cripple wall studs.

Spacing of boundary nailing for required wall bracing shall not exceed 6 inches (152 mm) on center along the foundation plate and the top plate of the cripple wall. Nail size, nail spacing for field nailing and more restrictive boundary nailing requirements shall be as required elsewhere in the code for the specific bracing material used.

2320A.11.6 Headers. *In all cases, the construction at heads and jambs of openings shall be structurally adequate for the design loads prescribed in these regulations.*

2320A.11.7 Pipes in walls. Stud partitions containing plumbing, heating, or other pipes shall be so framed and the joists underneath so spaced as to give proper clearance for the piping. Where a partition containing such piping runs parallel to the floor joists, the joists underneath such partitions shall be doubled and spaced to permit the passage of such pipes and shall be bridged. *Notches shall not be placed in studs unless fully detailed on the approved plans to restore the structural resistance of the member.*

> ***NOTE:** See Section 2320A.11.10 for bored holes.*

All splices in studs or plates which are required to resist loads shall be designed and fully detailed on the approved drawings.

2320A.11.8 ***Not adopted by DSA, OSHPD.***

2320A.11.9 Cutting and notching. *Any cutting and notching shall be detailed on the approved plans.*

2320A.11.10 Bored holes. *Holes exceeding one third of the width of the member being penetrated shall not be placed in studs unless fully detailed on the approved plans. Holes not exceeding one third of the stud width shall be neatly bored and shall be located in the center of the member being penetrated.*

2320A.12 Roof and Ceiling Framing.

2320A.12.1 General. *Roof and ceiling framing shall be designed in accordance with the general provisions of these regulations.*

2320A.12.2 ***Not adopted by DSA, OSHPD.***

2320A.12.3 ***Not adopted by DSA, OSHPD.***

2320A.12.4 ***Not adopted by DSA, OSHPD.***

2320A.12.5 ***Not adopted by DSA, OSHPD.***

2320A.12.6 ***Not adopted by DSA, OSHPD.***

2320A.12.7 Purlins. Purlins *shall be designed in accordance with the general provisions of these regulations.*

2320A.12.8 Blocking. Roof rafters and ceiling joists shall be supported laterally to prevent rotation and lateral displacement when required by Division III, Part I, Section 4.4.1.2. *In addition, rafters of more than 8 inches (203 mm) in depth shall be provided with bridging in accordance with the provisions of Section 2316A.2, Item 6.*

2320A.12.9 Roof sheathing. Roof sheathing shall be in accordance with Tables 23A-II-E-1 and 23A-II-E-2 for wood structural panels. * * *

Wood structural panels used for roof sheathing shall be bonded by intermediate or exterior glue. Wood structural panel roof sheathing exposed on the underside shall be bonded with exterior glue.

2320A.12.10 Roof planking. Planking shall be designed in accordance with the general provisions of this code.

2320A.13 Exit Facilities. In Seismic Zones 3 and 4, exterior exit balconies, stairs and similar exit facilities shall be positively anchored to the primary structure at not over 8 feet (2438 mm) on center or shall be designed for lateral forces. Such attachment shall not be accomplished by use of toenails or nails subject to withdrawal.

TABLE 23A-IV-A—*NOT ADOPTED BY DSA, OSHPD.*

TABLE 23A-IV-B—*NOT ADOPTED BY DSA, OSHPD.*

TABLE 23A-IV-C-1—BRACED WALL PANELS[1]

SEISMIC ZONE	CONDITION	CONSTRUCTION METHOD[2,3]								BRACED PANEL LOCATION AND LENGTH[4]
		1	2	3	4	5	6	7	8	
0, 1 and 2A	One story, top of two or three story	X	X	X	X	X	X	X	X	Each end and not more than 25 feet (7620 mm) on center
	First story of two story or second story of three story	X	X	X	X	X	X	X	X	
	First story of three story		X	X	X	X[5]	X	X	X	
2B, 3 and 4	One story, top of two story or three story		X	X	X	X	X	X[6]	X	Each end and not more than 25 feet (7620 mm) on center
	First story of two story or second of three story		X	X	X	X[5]	X	X[6]	X	Each end and not more than 25 feet (7620 mm) on center but not less than 25% of building length[7]
	First story of three story		X	X	X	X[5]	X	X[6]	X	Each end and not more than 25 feet (7620 mm) on center but not less than 40% of building length[7]

[1]This table specifies minimum requirements for braced panels which form interior or exterior braced wall lines.
[2]See Section 2320A.11.3 for full description.
[3]See Section 2320A.11.4 for alternate braced panel requirement.
[4]Building length is the dimension parallel to the braced wall length.
[5]Gypsum wallboard applied to supports at 16 inches (406 mm) on center.
[6]Not permitted for bracing cripple walls in Seismic Zone 4. See Section 2320A.11.5.
[7]The required lengths shall be doubled for gypsum board applied to only one face of a braced wall panel.

TABLE 23A-IV-C-2—CRIPPLE WALL BRACING

SEISMIC ZONE	CONDITION	AMOUNT OF CRIPPLE WALL BRACING[1,2] × 25.4 for mm
4	One story above cripple wall	$^3/_8''$ wood structural panel with 8d at $6''/12''$ nailing on 60 percent of wall length minimum
	Two story above cripple wall	$^3/_8''$ wood structural panel with 8d at $4''/12''$ nailing on 50 percent of wall length minimum or $^3/_8''$ wood structural panel with 8d at $6''/12''$ nailing on 75 percent of wall length minimum
3	One story above cripple wall	$^3/_8''$ wood structural panel with 8d at $6''/12''$ nailing on 40 percent of wall length minimum
0, 1 and 2	One story above cripple wall	$^3/_8''$ wood structural panel with 8d at $6''/12''$ nailing on 30 percent of wall length minimum
0, 1, 2 and 3	Two story above cripple wall	$^3/_8''$ wood structural panel with 8d at $4''/12''$ nailing on 40 percent of wall length minimum or $^3/_8''$ wood structural panel with 8d at $6''/12''$ nailing on 60 percent of wall length minimum

[1]Braced panel length shall be at least two times the height of the cripple wall, but not less than 48 inches (1219 mm).
[2]All panels along a wall shall be nearly equal in length and shall be nearly equally spaced along the length of the wall.

TABLE 23A-IV-D-1—WOOD STRUCTURAL PANEL WALL SHEATHING[1]
(Not exposed to the weather, strength axis parallel or perpendicular to studs)

MINIMUM THICKNESS (inch)	PANEL SPAN RATING	STUD SPACING (inches) × 25.4 for mm		
			Sheathing under Coverings Specified in Section 2310A.4	
× 25.4 for mm		Siding Nailed to Studs	Sheathing Parallel to Studs	Sheathing Perpendicular to Studs
$^5/_{16}$	12/0, 16/0, 20/0 Wall—16 o.c.	16	—	16
$^3/_8$, $^{15}/_{32}$, $^1/_2$	16/0, 20/0, 24/0, 32/16 Wall—24 o.c.	24	16	24
$^7/_{16}$, $^{15}/_{32}$, $^1/_2$	24/0, 24/16, 32/16 Wall—24 o.c.	24	24[2]	24

[1]In reference to Section 2320A.11.3, blocking of horizontal joints is not required.
[2]Plywood shall consist of four or more plies.

TABLE 23A-IV-D-2—*NOT ADOPTED BY DSA, OSHPD.*

portions thereof, which are below the isolation interface shall be designed and constructed in accordance with the requirements of Section 1632.

SECTION 1661 — DETAILED SYSTEMS REQUIREMENTS

1661.1 General. The isolation system and the structural system shall comply with the requirements of Section 1633 and the material requirements of Chapters 19 through 23. In addition, the isolation system shall comply with the detailed system requirements of this section and the structural system shall comply with the detailed system requirements of this section and the applicable portions of Section 1633.

1661.2 Isolation System.

1661.2.1 Environmental conditions. In addition to the requirements for vertical and lateral loads induced by wind and earthquake, the isolation system shall be designed with consideration given to other environmental conditions including aging effects, creep, fatigue, operating temperature and exposure to moisture or damaging substances.

1661.2.2 Wind forces. Isolated structures shall resist design wind loads at all levels above the isolation interface in accordance with the general wind design provisions. At the isolation interface, a wind restraint system shall be provided to limit lateral displacement in the isolation system to a value equal to that required between floors of the structure above the isolation interface.

1661.2.3 Fire resistance. Fire resistance for the isolation system shall meet that required for the building columns, walls or other structural elements in which it is installed.

Isolator systems required to have a fire-resistive rating shall be protected with approved materials or construction assemblies designed to provide the same degree of fire resistance as the structural element in which it is installed when tested in accordance with UBC Standard 7-1. See Section 703.2.

Such isolation system protection applied to isolator units shall be capable of retarding the transfer of heat to the isolator unit in such a manner that the required gravity load-carrying capacity of the isolator unit will not be impaired after exposure to the standard time-temperature curve fire test prescribed in UBC Standard 7-1 for a duration not less than that required for the fire-resistive rating of the structural element in which it is installed.

Such isolation system protection applied to isolator units shall be suitably designed and securely installed so as not to dislodge, loosen, sustain damage, or otherwise impair its ability to accommodate the seismic movements for which the isolator unit is designed and to maintain its integrity for the purpose of providing the required fire-resistive protection.

1661.2.4 Lateral restoring force. The isolation system shall be configured to produce a restoring force such that the lateral force at the total design displacement is at least $0.025W$ greater than the lateral force at 50 percent of the total design displacement.

> **EXCEPTION:** The isolation system need not be configured to produce a restoring force, as required above, provided the isolation system is capable of remaining stable under full vertical load and accommodating a total maximum displacement equal to the greater of either 3.0 times the total design displacement 36 C_{VM}, inches (For **SI:** 914.4 C_{VM}, mm).

1661.2.5 Displacement restraint. The isolation system may be configured to include a displacement restraint that limits lateral displacement due to the maximum capable earthquake to less than C_{VM}/C_{VD} times the total design displacement, provided that the seismic-isolated structure is designed in accordance with the following criteria when more stringent than the requirements of Section 1629.

1. Maximum capable earthquake response is calculated in accordance with the dynamic analysis requirements of Sections 1631 and 1659, explicitly considering the nonlinear characteristics of the isolation system and the structure above the isolation system.

2. The ultimate capacity of the isolation system and structural elements below the isolation system shall exceed the strength and displacement demands of the maximum capable earthquake.

3. The structure above the isolation system is checked for stability and ductility demand of the maximum capable earthquake.

4. The displacement restraint does not become effective at a displacement less than 0.75 times the total design displacement unless it is demonstrated by analysis that earlier engagement does not result in unsatisfactory performance.

1661.2.6 Vertical load stability. Each element of the isolation system shall be designed to be stable under the maximum vertical load, $1.2D + 1.0L + |E|_{max}$ and the minimum vertical load, $0.80\text{-}|E_{min}|$, at a horizontal displacement equal to the total maximum displacement. The vertical earthquake load on an individual isolation unit due to overturning, $|E|_{max}$ and $|E|_{min}$, shall be based on peak response due to the maximum capable earthquake.

1661.2.7 Overturning. The factor of safety against global structural overturning at the isolation interface shall not be less than 1.0 for required load combinations. All gravity and seismic loading conditions shall be investigated. Seismic forces for overturning calculations shall be based on the maximum capable earthquake and W shall be used for the vertical restoring force.

Local uplift of individual elements is permitted provided the resulting deflections do not cause overstress or instability of the isolator units or other building elements.

1661.2.8 Inspection and replacement.

1. Access for inspection and replacement of all components of the isolation system shall be provided.

2. The architect or engineer of record or a person designated by the architect or engineer of record shall complete a final series of inspections or observations of building separation areas and of components that cross the isolation interface prior to the issuance of the certificate of occupancy for the seismic-isolated building. Such inspections and observations shall indicate that as-built conditions allow for free and unhindered displacement of the structure to maximum design levels and that all components that cross the isolation interface as installed, are able to accommodate the stipulated displacements.

3. Seismic-isolated buildings shall have a periodic monitoring, inspection and maintenance program for the isolation system established by the architect or engineer responsible for the design of the system. The objective of such a program shall be to ensure that all elements of the isolation system are able to perform to minimum design levels at all times.

4. Remodeling, repair or retrofitting at the isolation system interface, including that of components that cross the isolation interface, shall be performed under the direction of an architect or engineer licensed in the appropriate disciplines and experienced in the design and construction of seismic-isolated structures.

5. Horizontal displacement recording devices shall be installed at the isolation interface in seismic-isolated buildings.

1661.2.9 Quality control. A quality control testing program for isolator units shall be established by the engineer responsible for the structural design.

1661.3 Structural System.

1661.3.1 Horizontal distribution of force. A horizontal diaphragm or other structural elements shall provide continuity above the isolation interface and shall have adequate strength and ductility to transmit forces (due to nonuniform ground motion) from one part of the building to another.

1661.3.2 Building separations. Minimum separations between the isolated building and surrounding retaining walls or other fixed obstructions shall not be less than the total maximum displacement.

SECTION 1662 — NONBUILDING STRUCTURES

Nonbuilding structures shall be designed in accordance with the requirements of Section 1634 using design displacements and forces calculated in accordance with Section 1658 or 1659.

SECTION 1663 — FOUNDATIONS

Foundations shall be designed and constructed in accordance with the requirements of Chapter 18 using design forces calculated in accordance with Section 1658 or 1659.

SECTION 1664 — DESIGN AND CONSTRUCTION REVIEW

1664.1 General. A design review of the isolation system and related test programs shall be performed by an independent engineering team including persons licensed in the appropriate disciplines, experienced in seismic analysis methods and the theory and application of seismic isolation.

1664.2 Isolation System. Isolation system design review shall include, but not be limited to, the following:

1. Review of site-specific seismic criteria, including the development of site-specific spectra and ground motion time histories, and all other design criteria developed specifically for the project.

2. Review of the preliminary design, including the determination of the total design displacement of the isolation system design displacement and lateral force design level.

3. Overview and observation of prototype testing (Section 1665).

4. Review of the final design of the entire structural system and all supporting analyses.

5. Review of the isolation system quality control testing program (Section 1661.2.9).

The engineer of record shall submit with the plans and calculations a statement by all members of the independent engineering team stating that the above has been completed.

SECTION 1665 — REQUIRED TESTS OF ISOLATION SYSTEM

1665.1 General. The deformation characteristics and damping values of the isolation system used in the design and analysis of seismic-isolated structures shall be based on the following tests of a selected sample of the components prior to construction.

The isolation system components to be tested shall include the wind restraint system if such systems are used in the design.

The tests specified in this section are for establishing and validating the design properties of the isolation system, and shall not be considered as satisfying the manufacturing quality control tests of Section 1661.2.9.

1665.2 Prototype Tests.

1665.2.1 General. Prototype tests shall be performed separately on two full-size specimens or sets of specimens, as appropriate, of each type and size of isolator unit of the isolation system. The test specimens shall include the wind restraint system, as well as individual isolator units, if such systems are used in the design. Specimens tested shall not be used for construction.

1665.2.2 Record. For each cycle of tests the force-deflection behavior of the test specimen shall be recorded.

1665.2.3 Sequence and cycles.

1665.2.3.1 The following sequence of tests shall be performed for the prescribed number of cycles at a vertical load equal to the average $D + 0.5L$ on all isolator units of a common type and size: CA

1. Twenty fully reversed cycles of loading at a lateral force corresponding to the wind design force.

2. Three fully reversed cycles of loading at each of the following increments of displacement: 0.2 D_D, 0.5 D_D and 1.0 D_D, 1.0 D_M.

3. Three fully reversed cycles at the total maximum displacement, $1.0D_{TM}$.

4. $(15C_{VD}/C_{VA}B_D)$, but not less than 10, fully reversed cycles of loading at 1.0 times the total design displacement, $1.0D_{TD}$.

1665.2.3.2 [For BSC] The following sequence of tests shall be performed for the prescribed number of cycles at a vertical load equal to the average $D + 0.5L$ on all isolator units of a common type and size:

1. Twenty fully reversed cycles of loading at a lateral force corresponding to the wind design force.

2. Three fully reversed cycles of loading at each of the following increments of displacement: 0.2 D_D, 0.5 D_D and 1.0 D_D, 1.0 D_M.

3. Three fully reversed cycles at the total maximum displacement, $1.0D_{TM}$.

4. *$(15C_{VD}/C_{AD}B_D)$*, but not less than 10, fully reversed cycles of loading at 1.0 times the total design displacement, 1.0DTD. CA

If an isolator unit is also a vertical load-carrying element, then Item 2 of the sequence of cyclic tests specified above shall be performed for two additional vertical load cases:

$$(1)\ \ 1.2D + 0.5L + |E|$$

$$(2)\ \ 0.8D - |E|$$

where D and L are defined in Chapter 16, Division III. The vertical test load on an individual isolator unit shall include the load increment due to earthquake overturning, $|E|$, and shall be equal to or greater than the peak earthquake vertical force response corresponding to the test displacement being evaluated. In these tests, the combined vertical load shall be taken as the typical or average downward force on all isolator units of a common type and size.

1665.2.4 Units dependent on loading rates. If the force-deflection properties of the isolator units are dependent on the rate of loading, then each set of tests specified in Section 1665.2.3 shall be performed dynamically at a frequency equal to the inverse of the effective period, T_D, of the isolated structure.

If reduced-scale prototype specimens are used to quantify rate-dependent properties of isolators, the reduced-scale prototype specimens shall be of the same type and material and be manufactured with the same processes and quality as full-scale prototypes, and shall be tested at a frequency that represents full-scale prototype loading rates.

The force-deflection properties of an isolator unit shall be considered to be dependent on the rate of loading if there is greater than a plus or minus 10 percent difference in the effective stiffness at the design displacement when tested at a frequency equal to the inverse of the effective period, T_D, of the isolated structure and when tested at any frequency in the range of 0.1 to 2.0 times the inverse of the effective period, T_D, of the isolated structure.

1665.2.5 Units dependent on bilateral load. If the force-deflection properties of the isolator units are dependent on bilateral load, then the tests specified in Sections 1665.2.3 and 1665.2.4 shall be augmented to include bilateral load at increments of the total design displacement 0.25 and 1.0, 0.50 and 1.0, 0.75 and 1.0, and 1.0 and 1.0.

EXCEPTION: If reduced-scale prototype specimens are used to quantify bilateral-load-dependent properties, then such scaled specimens shall be of the same type and material, and manufactured with the same processes and quality as full-scale prototypes.

The force-deflection properties of an isolator unit shall be considered to be dependent on bilateral load, if the bilateral and unilateral force-deflection properties have greater than a plus or minus 10 percent difference in effective stiffness at the design displacement.

1665.2.6 Maximum and minimum vertical load. Isolator units that carry vertical load shall be statically tested for the maximum and minimum vertical load, at the total maximum displacement. In these tests, the combined vertical loads of $1.2D + 1.0L + |E|_{max}$ shall be taken as the maximum vertical force, and the combined vertical load of $0.8D - |E|_{min}$ shall be taken as the minimum vertical force, on any one isolator unit of a common type and size. The vertical load on an individual isolator unit shall include the load increment due to earthquake overturning, $|E|_{max}$ and $|E|_{min}$, and shall be based on peak response due to the maximum capable earthquake.

1665.2.7 Sacrificial wind-restraint systems. If a sacrificial wind-restraint system is to be utilized, then the ultimate capacity shall be established by test.

1665.2.8 Testing similar units. The prototype tests are not required if an isolator unit is of similar dimensional characteristics and of the same type and material as the prototype isolator unit that has been previously tested using the specified sequence of tests.

1665.3 Determination of Force-deflection Characteristics. The force-deflection characteristics of the isolation system shall be based on the cyclic load tests of isolator prototypes specified in Section 1665.2.3.

As required, the effective stiffness of an isolator unit, k_{eff}, shall be calculated for each cycle of loading by the formula:

$$k_{eff} = \frac{F^+ - F^-}{\Delta^+ - \Delta^-} \qquad (65\text{-}1)$$

where $F+$ and $F-$ are the positive and negative forces at Δ^+ and Δ^-, respectively.

As required, the effective damping (β_{eff}) of an isolator unit shall be calculated for each cycle of loading by the formula:

$$\beta_{eff} = \frac{2}{\pi}\left[\frac{E_{Loop}}{k_{eff}(|\Delta^+| + |\Delta^-|)^2}\right] \qquad (65\text{-}2)$$

where the energy dissipated per cycle of loading, E_{Loop}, and the effective stiffness, k_{eff}, shall be based on test displacements of Δ^+ and Δ^-.

1665.4 System Adequacy.

1665.4.1 The performance of the test specimens shall be assessed as adequate if the following conditions are satisfied: CA

1. The force-deflection plots of all tests specified in Section 1665.2 have a positive incremental force-carrying capacity.

2. For each increment of test displacement specified in Section 1665.2.3, Item 2, and for each vertical load case specified in Section 1665.2.3:

   2.1 There is no greater than a plus or minus 10 percent difference between the effective stiffness at each of the three cycles of test and the average value of effective stiffness for each test specimen.

   2.2 There is no greater than a 10 percent difference in the average value of effective stiffness of the two test specimens of a common type and size of the isolator unit over the required three cycles of test.

3. For each specimen there is no greater than a plus or minus 20 percent change in the initial effective stiffness of each test specimen over the ($15C_{VD}/C_{VA}B_D$), but not less than 10, cycles of the test specified in Section 1665.2.3, Item 4.

4. For each specimen there is no greater than a 20 percent decrease in the initial effective damping over for the ($15C_{VD}/C_{VA}B_D$), but not less than 10, cycles of the test specified in Section 1665.2.3, Item 4.

5. All specimens of vertical load-carrying elements of the isolation system remain stable at the total maximum displacement for static load as prescribed in Section 1665.2.6.

1665.4.2 [For BSC] The performance of the test specimens shall be assessed as adequate if the following conditions are satisfied: CA

1. The force-deflection plots of all tests specified in Section 1665.2 have a positive incremental force-carrying capacity.

2. For each increment of test displacement specified in Section 1665.2.3, Item 2, and for each vertical load case specified in Section 1665.2.3:

   2.1 There is no greater than a plus or minus 10 percent difference between the effective stiffness at each of the three cycles of test and the average value of effective stiffness for each test specimen.

   2.2 There is no greater than a 10 percent difference in the average value of effective stiffness of the two test specimens of a common type and size of the isolator unit over the required three cycles of test.

3. For each specimen there is no greater than a plus or minus 20 percent change in the initial effective stiffness of each test specimen over the *($15C_{VD}/C_{AD}B_D$)*, but not less than 10, cycles of the test specified in Section 1665.2.3, Item 4. CA

4. For each specimen there is no greater than a 20 percent decrease in the initial effective damping over for the *($15C_{VD}/C_{AD}B_D$)*, but not less than 10, cycles of the test specified in Section 1665.2.3, Item 4. CAC

5. All specimens of vertical load-carrying elements of the isolation system remain stable at the total maximum displacement for static load as prescribed in Section 1665.2.6.

1665.5 Design Properties of the Isolation System.

1665.5.1 Maximum and minimum effective stiffness. At the design displacement, the maximum and minimum effective stiffnesses of the isolation system, k_{Dmax} and k_{Dmin}, shall be based on the cyclic tests of Section 1665.2.3 and calculated by the formulas:

$$k_{Dmax} = \frac{\sum |F_D^+|_{max} + \sum |F_D^-|_{max}}{2D_D} \quad (65\text{-}3)$$

$$k_{Dmin} = \frac{\sum |F_D^+|_{min} + \sum |F_D^-|_{min}}{2D_D} \quad (65\text{-}4)$$

At the maximum displacement, the maximum and minimum effective stiffness of the isolation system, k_{Mmax} and k_{Mmin}, shall be based on the cyclic tests of Section 1665.2.3 and calculated by the formulas:

$$k_{Mmax} = \frac{\sum |F_M^+|_{max} + \sum |F_M^-|_{max}}{2D_M} \quad (65\text{-}5)$$

$$k_{Mmin} = \frac{\sum |F_M^+|_{min} + \sum |F_M^-|_{min}}{2D_M} \quad (65\text{-}6)$$

For isolator units that are found by the tests of Sections 1665.2.3, 1665.2.4 and 1665.2.5 to have force-deflection characteristics which vary with vertical load, rate of loading or bilateral load, respectively, the values of k_{Dmax} and k_{Mmax} shall be increased and the values of k_{Dmin} and k_{Mmin} shall be decreased, as necessary, to bound the effects of measured variation in effective stiffness.

1665.5.2 Effective damping. At the design displacement, the effective damping of the isolation system, β_D, shall be based on the cyclic tests of Section 1665.2.3 and calculated by the formula:

$$\beta_D = \frac{1}{2\pi}\left[\frac{\sum E_D}{k_{Dmax} D_D^2}\right] \quad (65\text{-}7)$$

In Formula (65-7), the total energy dissipated in the isolation system per cycle of design displacement response, ΣE_D, shall be taken as the sum of the energy dissipated per cycle in all isolator units measured at test displacements, Δ^+ and Δ^-, that are equal in magnitude to the design displacement, D_D.

At the maximum displacement, the effective damping of the isolation system, β_M, shall be based on the cyclic tests of Section 1665.2.3 and calculated by the formula:

$$\beta_M = \frac{1}{2\pi}\left[\frac{\sum E_M}{k_{Mmax} D_M^2}\right] \quad (65\text{-}8)$$

In Formula (65-8), the total energy dissipated in the isolation system per cycle of response, E_M, shall be taken as the sum of the energy dissipated per cycle in all isolator units measured at test displacements, Δ^+ and Δ^-, that are equal in magnitude to the maximum displacement, D_M.

TABLE A-16-C—DAMPING COEFFICIENTS, B_D AND B_M

EFFECTIVE DAMPING, β_D or β_M (percentage of critical)[1,2]	B_D or B_M FACTOR
≤ 2	0.8
5	1.0
10	1.2
20	1.5
30	1.7
40	1.9
≥ 50	2.0

[1]The damping coefficient shall be based on the effective damping of the isolation system determined in accordance with the requirements of Section 1665.5.

[2]The damping coefficient shall be based on linear interpolation for effective damping values other than those given.

TABLE A-16-D—MAXIMUM CAPABLE EARTHQUAKE RESPONSE COEFFICIENT, M_M

DESIGN BASIS EARTHQUAKE SHAKING INTENSITY, ZN_v	MAXIMUM CAPABLE EARTHQUAKE RESPONSE COEFFICIENT, M_M
0.075	2.67
0.15	2.0
0.20	1.75
0.30	1.50
0.40	1.25
≥ 0.50	1.20

TABLE A-16-E—STRUCTURAL SYSTEMS ABOVE THE ISOLATION INTERFACE[1]

BASIC STRUCTURAL SYSTEM[2]	LATERAL-FORCE-RESISTING SYSTEM DESCRIPTION	R_I	HEIGHT LIMIT FOR SEISMIC ZONES 3 AND 4 (× 304.8 for mm)
1. Bearing wall system	1. Light-framed walls with shear panels		
	a. Wood structural panel walls for structures three stories or less	2.0	65
	b. All other light-framed walls	2.0	65
	2. Shear walls		
	a. Concrete	2.0	160
	b. Masonry	2.0	160
	3. Light steel-framed bearing walls with tension-only bracing	1.6	65
	4. Braced frames where bracing carries gravity load		
	a. Steel	1.6	160
	b. Concrete[3]	1.6	—
	c. Heavy timber	1.6	65
2. Building frame system	1. Steel eccentrically braced frame (EBF)	2.0	240
	2. Light-framed walls with shear panels		
	a. Wood structural panel walls for structures three stories or less	2.0	65
	b. All other light-framed walls	2.0	65
	3. Shear walls		
	a. Concrete	2.0	240
	b. Masonry	2.0	160
	4. Ordinary braced frames		
	a. Steel	1.6	160
	b. Concrete[3]	1.6	—
	c. Heavy timber	1.6	65
	5. Special concentrically braced frames		
	a. Steel	2.0	240

(Continued)

TABLE A-16-E—STRUCTURAL SYSTEMS ABOVE THE ISOLATION INTERFACE[1]

BASIC STRUCTURAL SYSTEM[2]	LATERAL-FORCE-RESISTING SYSTEM DESCRIPTION	R_I	HEIGHT LIMIT FOR SEISMIC ZONES 3 AND 4 (× 304.8 for mm)
3. Moment-resisting frame system	1. Special moment-resisting frame (SMRF)		
	a. Steel	2.0	N.L.
	b. Concrete	2.0	N.L.
	2. Masonry moment-resisting wall frame (MMRWF)	2.0	160
	3. Concrete intermediate moment-resisting frame (IMRF)[4]	2.0	—
	4. Ordinary moment-resisting frame (OMRF)		
	a. Steel[5]	2.0	160
	b. Concrete[6]	2.0	—
	5. Special truss moment frames of steel (STMF)	2.0	240
4. Dual systems	1. Shear walls		
	a. Concrete with SMRF	2.0	N.L.
	b. Concrete with steel OMRF	2.0	160
	c. Concrete with IMRF[4]	2.0	160
	d. Masonry with SMRF	2.0	160
	e. Masonry with steel OMRF	2.0	160
	f. Masonry with concrete IMRF[3]	2.0	—
	g. Masonry with masonry MMRWF	2.0	160
	2. Steel EBF		
	a. With steel SMRF	2.0	N.L.
	b. With steel OMRF	2.0	160
	3. Ordinary braced frames		
	a. Steel with steel SMRF	2.0	N.L.
	b. Steel with steel OMRF	2.0	160
	c. Concrete with concrete SMRF[3]	2.0	—
	d. Concrete with concrete IMRF[3]	2.0	—
	4. Specially concentrically braced frames		
	a. Steel with steel SMRF	2.0	N.L.
	b. Steel with steel OMRF	2.0	160
5. Cantilevered column building systems	1. Cantilevered column elements	1.4	35[7]
6. Shear wall-frame interaction systems	1. Concrete[6]	2.0	—
7. Undefined systems	See Sections 1629.6.7 and 1629.9.2		—

N.L.—no limit.

[1]See Section 1630.4 for combination of structural systems.

[2]Basic structural systems are defined in Section 1629.6.

[3]Prohibited in Seismic Zones 3 and 4.

[4]Prohibited in Seismic Zones 3 and 4, except as permitted in Section 1633.2.

[5]Ordinary moment-resisting frames in Seismic Zone 1 meeting the requirements of Section 2213.6 may use an R_I value of 2.0.

[6]Prohibited in Seismic Zones 2A, 2B, 3 and 4. See Section 1633.2.7.

[7]Total height of the building including cantilevered columns.

TABLE A-16-F—SEISMIC COEFFICIENT, C_{AM}[1]

SOIL PROFILE TYPE	MAXIMUM CAPABLE EARTHQUAKE SHAKING INTENSITY M_MZN_a				
	M_MZN_a = 0.075	M_MZN_a = 0.15	M_MZN_a = 0.2	M_MZN_a = 0.3	$M_MZN_a \geq$ 0.4
S_A	0.06	0.12	0.16	0.24	$0.8M_MZN_a$
S_B	0.08	0.15	0.20	0.30	$1.0M_MZN_a$
S_C	0.09	0.18	0.24	0.33	$1.0M_MZN_a$
S_D	0.12	0.22	0.28	0.36	$1.1M_MZN_a$
S_E	0.19	0.30	0.34	0.36	$0.9M_MZN_a$
S_F	See Footnote 2				

[1]Linear interpolation may be used to determine the value of C_{AM} for values of M_MZN_a for other than those shown in the table.

[2]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for soil.

TABLE A-16-G—SEISMIC COEFFICIENT, C_{VM}[1]

SOIL PROFILE TYPE	MAXIMUM CAPABLE EARTHQUAKE SHAKING INTENSITY M_MZN_v				
	M_MZN_v = 0.075	M_MZN_v = 0.15	M_MZN_v = 0.20	M_MZN_v = 0.30	$M_MZN_v \geq$ 0.40
S_A	0.06	0.12	0.16	0.24	$0.8M_MZN_v$
S_B	0.08	0.15	0.20	0.30	$1.0M_MZN_v$
S_C	0.13	0.25	0.32	0.45	$1.4M_MZN_v$
S_D	0.18	0.32	0.40	0.54	$1.6M_MZN_v$
S_E	0.26	0.50	0.64	0.84	$2.4M_MZN_v$
S_F	See Footnote 2				

[1]Linear Interpolation may be used to determine the value of C_{VM} for values of M_MZN_v for other than those shown in the table.

[2]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for soil.

DIVISION *VII*—EARTHQUAKE REGULATIONS FOR SEISMIC-ISOLATED STRUCTURES *[FOR OSHPD, DSA/SS]*

SECTION 1654A — GENERAL

Every seismic-isolated structure and every portion thereof shall be designed and constructed in accordance with the requirements of this division and the applicable requirements of Chapter *16A, Part VII.*

The lateral-force-resisting system and the isolation system shall be designed to resist the deformations and stresses produced by the effects of seismic ground motions as provided in this division.

Where wind forces prescribed by Chapter 16, Part III, produce greater deformations or stresses, such loads shall be used for design in lieu of the deformations and stresses resulting from earthquake forces.

SECTION 1655A — DEFINITIONS

The definitions of Section 1627A and the following apply to the provisions of this division:

DESIGN DISPLACEMENT is the design-basis earthquake lateral displacement, excluding additional displacement due to actual and accidental torsion, required for design of the isolation system.

DESIGN-BASIS EARTHQUAKE is defined in Section 1631A.2.

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

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

ISOLATION INTERFACE is 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, *and any other structure*.

ISOLATION SYSTEM is 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 if such a system is used to meet the design requirements of this section.

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

MAXIMUM CAPABLE EARTHQUAKE is the maximum level of earthquake ground shaking that may ever be expected at the building site within the known geological framework. In Seismic Zones 3 and 4, this intensity may be taken as the level of earthquake ground motion that has a 10 percent probability of being exceeded in a 100-year time period.

MAXIMUM DISPLACEMENT is the maximum capable earthquake lateral displacement, excluding additional displacement due to actual and accidental torsion, required for design of the isolation system.

TOTAL DESIGN DISPLACEMENT is the design-basis 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 is the maximum capable earthquake lateral displacement, including additional displacement due to actual and accidental torsion, required for verification of the stability of the isolation system, or elements thereof, design of building separations, and vertical load testing of isolator unit prototypes.

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

SECTION 1656A — SYMBOLS AND NOTATIONS

The symbols and notations of Section 1628A and the following provisions apply to the provisions of this division:

B_D = numerical coefficient related to the effective damping of the isolation system at the design displacement, β_D, as set forth in Table A-16-C.

B_M = numerical coefficient related to the effective damping of the isolation system at the maximum displacement, β_M, as set forth in Table A-16-C.

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

C_{AD} = the seismic coefficient, C_a, as set forth in Table 16A-Q.

C_{AM} = the seismic coefficient, C_a, as set forth in Table A-16-F for shaking intensity, $M_M ZN_a$.

C_{VD} = seismic coefficient, C_v, as set forth in Table 16A-R.

C_{VM} = seismic coefficient, C_v, as set forth in Table A-16-G for shaking intensity, $M_M ZN_v$.

D_D = design displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Formula (58-1).

D_D' = design displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Formula (59-1).

D_M = maximum displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Formula (58-3).

D_M' = maximum displacement, in inches (mm), at the center of rigidity of the isolation system in the direction under consideration, as prescribed by Formula (59-2).

D_{TD} = total design displacement, in inches (mm), of an element of the isolation system including both translational displacement at the center of rigidity, D_D, and the component of torsional displacement in the direction under consideration, as specified in Section 1658A.3.5.

D_{TM} = total maximum displacement, in inches (mm), of an element of the isolation system, including both translational displacement at the center of rigidity, D_M, and the component of torsional displacement in the direction under consideration, as specified by Section 1658A.3.3.

d = the longest plan dimension of the structure, in feet (mm).

E = *vertical force component due to an earthquake loading acting in the horizontal direction.*

E_{LOOP} = energy dissipated in kip-inches (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.

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

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

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

$F-$ = negative force, in kips (kN), in an isolator unit during a single cycle of prototype testing at a displacement amplitude of $\Delta-$.

$F+$ = positive force, in kips (kN), in an isolator unit during a single cycle of prototype testing at a displacement amplitude of $\Delta+$.

$\Sigma|F_D{}^+|_{max}$

= sum, for all isolator units, of the absolute values of the individual isolator unit's maximum positive force in kips (kN) at positive displacement D_D. For a given isolator unit, the maximum positive force at positive displacement, D_D, is determined by comparing each of the maximum positive forces that occurred during each cycle of the prototype test sequence associated with displacement increment D_D, and selecting the maximum positive value at positive displacement, D_D.

$\Sigma|F_D{}^+|_{min}$

= sum, for all isolator units, of the absolute values of the individual isolator unit's minimum positive force in kips (kN) at positive displacement D_D. For a given isolator unit, the minimum positive force at positive displacement, D_D, is determined by comparing each of the minimum positive forces that occurred during each cycle of the prototype test sequence associated with displacement increment D_D, and selecting the minimum positive value at positive displacement, D_D.

$\Sigma|F_D{}^-|_{max}$

= sum, for all isolator units, of the absolute values of the individual isolator unit's maximum negative force in kips (kN) at negative displacement D_D. For a given isolator unit, the maximum negative force at negative displacement, D_D, is determined by comparing each of the maximum negative forces that occurred during each cycle of the prototype test sequence associated with displacement increment D_D, and selecting the maximum negative value at negative displacement, D_D.

$\Sigma|F_D{}^-|_{min}$

= sum, for all isolator units, of the absolute values of the individual isolator unit's minimum negative force in kips (kN) at negative displacement D_D. For a given isolator unit, the minimum negative force at negative displacement, D_D, is determined by comparing each of the minimum negative forces that occurred during each cycle of the prototype test sequence associated with displacement increment D_D, and selecting the minimum negative value at negative displacement, D_D.

$\Sigma|F_M{}^+|_{max}$

= sum, for all isolator units, of the absolute values of the individual isolator unit's maximum positive force in kips (kN) at positive displacement D_M. For a given isolator unit, the maximum positive force at positive displacement, D_M, is determined by comparing each of the maximum positive forces that occurred during each cycle of the prototype test sequence associated with displacement increment D_M, and selecting the maximum positive value at positive displacement, D_M.

$\Sigma|F_M{}^+|_{min}$

= sum, for all isolator units, of the absolute values of the individual isolator unit's minimum positive force in kips (kN) at positive displacement, D_M. For a given isolator unit, the minimum positive force at positive displacement, D_M, is determined by comparing each of the minimum positive forces that occurred during each cycle of the prototype test sequence associated with displacement increment D_M and selecting the minimum positive value at positive displacement, D_M.

$\Sigma|F_M{}^-|_{max}$

= sum, for all isolator units, of the absolute values of the individual isolator unit's maximum negative force in kips (kN) at negative displacement D_M. For a given isolator unit, the maximum negative force at negative displacement, D_M, is determined by comparing each of the maximum negative forces that occurred during each cycle of the prototype test sequence associated with displacement increment D_M and selecting the maximum negative value at negative displacement, D_M.

$\Sigma|F_M{}^-|_{min}$

= sum, for all isolator units, of the absolute values of the individual isolator unit's minimum negative force in kips (kN) at negative displacement D_M. For a given isolator unit, the minimum negative force at negative displacement, D_M, is determined by comparing each of the minimum negative forces that occurred during each cycle of the prototype test sequence associated with displacement increment D_M and selecting the minimum negative value at negative displacement, D_M.

g = gravity constant (386.4 in/sec.^{2}, or 9,810 mm/sec.^{2}, for **SI**).

k_{eff} = effective stiffness of an isolator unit, in kips/inch as prescribed by Formula (65-1).

k_{Dmax} = maximum effective stiffness, in kips/inch (kN/mm), of the isolation system at the design displacement in the horizontal direction under consideration.

k_{Mmax} = maximum effective stiffness, in kips/inch (kN/mm), of the isolation system at the maximum displacement in the horizontal direction under consideration.

k_{Dmin} = minimum effective stiffness, in kips/inch (kN/mm), of the isolation system at the design displacement in the horizontal direction under consideration.

k_{Mmin} = minimum effective stiffness, in kips/inch (kN/mm), of the isolation system at the maximum displacement in the horizontal direction under consideration.

M_M = numerical coefficient related to maximum capable earthquake response as set forth in Table A-16-D.

N_a = near-source factor used in the determination of C_{AD} and C_{AM} related to both the proximity of the building or structure to known faults with magnitudes and slip rates as set forth in Tables 16A-S and 16A-U.

N_v = near-source factor used in the determination of C_{VD} and C_{VM} related to both the proximity of the building or structure to known faults with magnitudes and slip rates as set forth in Tables 16A-T and 16A-U.

R_I = numerical coefficient related to the type of lateral-force-resisting system above the isolation system as set forth in Table A-16-E for seismic-isolated structures.

T_D = effective period, in seconds, of seismic-isolated structure at the design displacement in the direction under consideration, as prescribed by Formula (58-2).

T_M = effective period, in seconds, of seismic-isolated structure at the maximum displacement in the direction under consideration, as prescribed by Formula (58-4).

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 Formula (58-5).

V_s = the total lateral seismic design force or shear on elements above the isolation system as prescribed by Formula (58-8) and the limits specified in Section 1658A.

W = the total seismic dead load defined in Section 1630A.1. For design of the isolation system, W is the total seismic dead load weight of the structure above the isolation interface.

y = the distance, in feet (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.

β_{eff} = effective damping of the isolation system and isolator unit, as prescribed by Formula (65-2).

β_D = effective damping of the isolation system at the design displacement, as prescribed by Formula (65-3).

β_M = effective damping of the isolation system at the maximum displacement, as prescribed by Formula (65-4).

$\Delta+$ = maximum positive displacement of an isolator unit during each cycle of prototype testing.

$\Delta-$ = maximum negative displacement of an isolator unit during each cycle of prototype testing.

SECTION 1657A — CRITERIA SELECTION

1657A.1 Basis for Design. The procedures and limitations for the design of seismic-isolated structures shall be determined considering zoning, site characteristics, vertical acceleration, cracked section properties of concrete and masonry members, occupancy, configuration, structural system and height in accordance with Section 1629A, except as noted below.

1657A.2 Stability of the Isolation System. The stability of the vertical load-carrying elements of the isolation system shall be verified by analysis and test, as required, for lateral seismic displacement equal to the total maximum displacement.

1657A.3 Occupancy Categories. The importance factor, I, for a seismic-isolated building shall be taken as 1.0 regardless of occupancy category. *[For OSHPD 1, DSA/SS] The importance factor, I_p, for parts and portions of a seismic-isolated building shall be the same as that required for a fixed-base building of the same occupancy category.*

1657A.4 Configuration Requirements. Each structure shall be designated as being regular or irregular on the basis of the structural configuration above the isolation system, in accordance with Section 1629A.5.

1657A.5 Selection of Lateral Response Procedure.

1657A.5.1 General. *Seismic-isolated structures shall be* designed using the dynamic lateral response procedure of Section 1659A.

1657A.5.1.1 Period separation. *In each principal direction, the fundamental period, T of the superstructure, computed in accordance with Formula (30-10), assuming that the structure is fixed at the isolation interface, shall not exceed the isolated-structure period, T_M.*

1657A.5.2 Static analysis. The static lateral response procedure of Section 1658A *shall* be used *to establish minimum criteria only, and not be used for design purposes unless these minimum requirements exceed calculated values from the dynamic analysis.*

1657A.5.3 Dynamic analysis. The dynamic lateral response procedure of Section 1659A shall be used for design of seismic-isolated structures as specified below:

1. **Response spectrum analysis.** Response spectrum analysis may be used for design of a seismic-isolated structure, provided:

   1.1 The structure is located on Soil Profile Type S_A, S_B, S_C or S_D.

   1.2 The isolation system is defined by all of the attributes:

   1.2.1 The effective stiffness of the isolation system at the design displacement is greater than one half of the effective stiffness at 20 percent of the design displacement.

   1.2.2 The isolation system is capable of producing a restoring force, as specified in Section 1661A.2.4.

   1.2.3 The isolation system has force-deflection properties that are independent of the rate of loading.

   1.2.4 The isolation system has force-deflection properties that are independent of vertical load and bilateral load.

   1.2.5 The isolation system does not limit the total maximum displacement to less than C_{VM}/C_{VD} times the total design displacement.

   1.3 The near source factor, N_a, is equal to 1.0.

2. **Time-history analysis.** Time-history analysis may be used for design of any seismic-isolated structure and shall be used for design of all seismic-isolated structures not meeting the criteria of Section 1657A.5.3, Item 1.

3. **Site-specific design spectra.** Site-specific ground motion spectra of the design-basis earthquake and the maximum capable earthquake, developed in accordance with Section 1631A.2 *and 1637A*, shall be used for design and analysis of all seismic-isolated structures.

SECTION 1658A — STATIC LATERAL RESPONSE PROCEDURE

1658A.1 General. Except as provided in Section 1659A, every seismic-isolated structure, or portion thereof, shall be designed and constructed to resist minimum earthquake displacements and forces as specified by this section and the applicable requirements of Section 1630A.

1658A.2 Deformation Characteristics of the Isolation System. Minimum lateral earthquake design displacements and forces on seismic-isolated structures shall be based on the deformation characteristics of the isolation system.

The deformation characteristics of the isolation system shall explicitly include the effects of the wind-restraint system if such a system is used to meet the design requirements of this document.

The deformation characteristics of the isolation system shall be based on properly substantiated tests performed in accordance with Section 1665A.

1658A.3 Minimum Lateral Displacements.

1658A.3.1 Design displacement. The isolation system shall be designed and constructed to withstand minimum lateral earthquake displacements which act in the direction of each of the main horizontal axes of the structure in accordance with the formula:

$$D_D = \frac{\left(\frac{g}{4\pi^2}\right) C_{VD} T_D}{B_D} \qquad (58\text{-}1)$$

1658A.3.2 Effective period at the design displacement. The effective period of the isolated structure at the design displacement, T_D, shall be determined using the deformational characteristics of the isolation system in accordance with the formula:

$$T_D = 2\pi \sqrt{\frac{W}{k_{Dmin} g}} \qquad (58\text{-}2)$$

1658A.3.3 Maximum displacement. The maximum displacement of the isolation system, D_M, in the most critical direction of horizontal response shall be calculated in accordance with the formula:

$$D_M = \frac{\left(\frac{g}{4\pi^2}\right) C_{VM} T_M}{B_M} \qquad (58\text{-}3)$$

1658A.3.4 Effective period at the maximum displacement. The effective period of the isolated structure at the maximum displacement, T_M, shall be determined using the deformational characteristics of the isolation system in accordance with the formula:

$$T_M = 2\pi \sqrt{\frac{W}{k_{Mmin} g}} \qquad (58\text{-}4)$$

1658A.3.5 Total displacement. The total design displacement, D_{TD}, and the total maximum displacement, D_{TM}, of elements of the isolation system shall include additional displacement due to actual and accidental torsion calculated considering the spatial distribution of the lateral stiffness of the isolation system and the most disadvantageous location of mass eccentricity.

The total design displacement, D_{TD}, and the of total maximum displacement D_{TM}, of elements of an isolation system with uniform spatial distribution of lateral stiffness shall not be taken as less than that prescribed by the formulas:

$$D_{TD} = D_D\left[1 + y\frac{12e}{b^2 + d^2}\right] \qquad (58\text{-}5)$$

$$D_{TM} = D_M\left[1 + y\frac{12e}{b^2 + d^2}\right] \qquad (58\text{-}6)$$

The total design displacement, D_{TD}, and the total maximum displacement, D_{TM}, may be taken as less than the value prescribed by Formulas (58-5) and (58-6), but not less than 1.1 times D_D and 1.1 times D_M, respectively, provided the isolation system is shown by calculation to be configured to resist torsion accordingly.

1658A.4 Minimum Lateral Forces.

1658A.4.1 Isolation system and structural elements at or below the isolation system. The isolation system, the foundation, and all structural elements below the isolation system shall be designed and constructed to withstand a minimum lateral seismic force, V_b, using all of the appropriate provisions for a nonisolated structure where:

$$V_b = k_{Dmax} D_D \qquad (58\text{-}7)$$

1658A.4.2 Structural elements above the isolation system. The structure above the isolation system shall be designed and constructed to withstand a minimum shear force, V_s, using all of the appropriate provisions for a nonisolated structure where:

$$V_s = \frac{k_{Dmax}\, D_D}{R_I} \qquad (58\text{-}8)$$

The R_I factor shall be based on the type of lateral-force-resisting system used for the structure above the isolation system.

1658A.4.3 Limits on V_s. The value of V_s shall not be taken as less than the following:

1. The lateral seismic force required by Chapter 16, Division III, for a fixed-base structure of the same weight, W, and a period equal to the isolated period, T_D, *using the importance factor, I, given in Table 16A-K.* CAC

2. The base shear corresponding to the design wind load.

3. The lateral seismic force required to fully activate the isolation system factored by 1.5 (e.g., one and one-half times the yield level of a softening system, the ultimate capacity of a sacrificial wind-restraint system or the static friction level of a sliding system).

1658A.5 Vertical Distribution of Force. The total force shall be distributed over the height of the structure above the isolation interface in accordance with the formula:

$$F_x = \frac{V_s w_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad (58\text{-}9)$$

At each level designated as x, the force F_x shall be applied over the area of the building in accordance with the mass distribution at the level. Stresses in each structural element shall be calculated as the effect of force, F_x, applied at the appropriate levels above the base.

1658A.6 Drift Limits. The maximum interstory drift ratio of the structure above the isolation system shall not exceed $0.010/R_I$.

SECTION 1659A — DYNAMIC LATERAL-RESPONSE PROCEDURE

1659A.1 General. As required by Section 1657A, every seismic-isolated structure, or portion thereof, shall be designed and constructed to resist earthquake displacements and forces as specified in this section and the applicable requirements of Section 1631A.

SECTION 1661A — DETAILED SYSTEMS REQUIREMENTS

1661A.1 General. The isolation system and the structural system shall comply with the requirements of Section 1633A and the material requirements of Chapters 19 through 23. In addition, the isolation system shall comply with the detailed system requirements of this section and the structural system shall comply with the detailed system requirements of this section and the applicable portions of Section 1633A.

1661A.2 Isolation System.

1661A.2.1 Environmental conditions. In addition to the requirements for vertical and lateral loads induced by wind and earthquake, the isolation system shall be designed with consideration given to other environmental conditions including aging effects, creep, fatigue, operating temperature and exposure to moisture or damaging substances.

1661A.2.2 Wind forces. Isolated structures shall resist design wind loads at all levels above the isolation interface in accordance with the general wind design provisions. At the isolation interface, a wind restraint system shall be provided to limit lateral displacement in the isolation system to a value equal to that required between floors of the structure above the isolation interface.

1661A.2.3 Fire resistance. Fire resistance for the isolation system shall meet that required for the building columns, walls or other structural elements in which it is installed.

Isolator systems required to have a fire-resistive rating shall be protected with approved materials or construction assemblies designed to provide the same degree of fire resistance as the structural element in which it is installed when tested in accordance with UBC Standard 7-1. See Section 703.2.

Such isolation system protection applied to isolator units shall be capable of retarding the transfer of heat to the isolator unit in such a manner that the required gravity load-carrying capacity of the isolator unit will not be impaired after exposure to the standard time-temperature curve fire test prescribed in UBC Standard 7-1 for a duration not less than that required for the fire-resistive rating of the structural element in which it is installed.

Such isolation system protection applied to isolator units shall be suitably designed and securely installed so as not to dislodge, loosen, sustain damage, or otherwise impair its ability to accommodate the seismic movements for which the isolator unit is designed and to maintain its integrity for the purpose of providing the required fire-resistive protection.

1661A.2.4 Lateral restoring force. The isolation system shall be configured to produce a restoring force such that the lateral force at the total design displacement is at least $0.025W$ greater than the lateral force at 50 percent of the total design displacement.

> **EXCEPTION:** The isolation system need not be configured to produce a restoring force, as required above, provided the isolation system is capable of remaining stable under full vertical load and accommodating a total maximum displacement equal to the greater of either 3.0 times the total design displacement 36 C_{VM}, inches (For **SI:** 914.4 C_{VM}, mm).

1661A.2.5 Displacement restraint. The isolation system may be configured to include a displacement restraint that limits lateral displacement due to the maximum capable earthquake to less than C_{VM}/C_{VD} times the total design displacement, provided that the seismic-isolated structure is designed in accordance with the following criteria when more stringent than the requirements of Section 1629A.

1. Maximum capable earthquake response is calculated in accordance with the dynamic analysis requirements of Sections 1631A and 1659A, explicitly considering the nonlinear characteristics of the isolation system and the structure above the isolation system.

2. The ultimate capacity of the isolation system and structural elements below the isolation system shall exceed the strength and displacement demands of the maximum capable earthquake.

3. The structure above the isolation system is checked for stability and ductility demand of the maximum capable earthquake.

4. The displacement restraint does not become effective at a displacement less than 0.75 times the total design displacement unless it is demonstrated by analysis that earlier engagement does not result in unsatisfactory performance.

1661A.2.6 Vertical load stability. Each element of the isolation system shall be designed to be stable under the maximum vertical load, $1.2D + 1.0L + |E|_{max}$ and the minimum vertical load, $0.80\text{-}|E_{min}|$, at a horizontal displacement equal to the total maximum displacement. The vertical earthquake load on an individual isolation unit due to overturning, $|E|_{max}$ and $|E|_{min}$, shall be based on peak response due to the maximum capable earthquake. *When considering the load combinations above, the load combinations of Section 1612A and the earthquake loads of Section 1630A.1.1 need not be considered.*

1661A.2.7 Overturning. The factor of safety against global structural overturning at the isolation interface shall not be less than 1.0 for required load combinations. All gravity and seismic loading conditions shall be investigated. Seismic forces for overturning calculations shall be based on the maximum capable earthquake and W shall be used for the vertical restoring force.

Local uplift of individual elements is permitted provided the resulting deflections do not cause overstress or instability of the isolator units or other building elements. *The effects of uplift and/or rocking shall be explicitly accounted for in the analysis and in the testing of the isolator units.*

1661A.2.8 *Instrumentation,* Inspection and replacement.

1. Access for inspection and replacement of all components of the isolation system shall be provided.

2. The architect or engineer of record or a person designated by the architect or engineer of record shall complete a final series of inspections or observations of building separation areas and of components that cross the isolation interface prior to the issuance of the certificate of occupancy for the seismic-isolated building. Such inspections and observations shall indicate that as-built conditions allow for free and unhindered displacement of the structure to maximum design levels and that all components that cross the isolation interface as installed, are able to accommodate the stipulated displacements.

3. Seismic-isolated buildings shall have a periodic monitoring, inspection and maintenance program for the isolation system established by the * * * engineer responsible for the design of the system. The objective of such a program shall be to ensure that all elements of the isolation system are able to perform to minimum design levels at all times. *These programs shall be submitted for approval with the plans and specifications and shall be a condition of occupancy for the structure.*

4. Remodeling, repair or retrofitting at the isolation system interface, including that of components that cross the isolation interface, shall be performed under the direction of an architect or engineer licensed in the appropriate disciplines and experienced in the design and construction of seismic-isolated structures.

5. *A proposal for instrumentation and equipment specifications shall be forwarded to the enforcement agency for approval.*

Motion measuring instruments shall be located within the building and at levels immediately above and below the isolators. The owner of the building is responsible for the implementation of the instrumentation program. Maintenance of the instrumentation and removal and processing of the records shall be the responsibility of the enforcement agency or its designated agent.

6. *After every significant seismic event, the owner shall retain a structural engineer to make an inspection of the structural system. The inspection shall consist of viewing the performance of the building, reviewing the strong motion records, and a visual examination of the isolators and their connections for deterioration, offset or physical damage. A report for each inspection, including conclusions on the continuing adequacy of the structural system, shall be submitted as required to the enforcement agency.*

1661A.2.9 Quality control. A quality control testing program for isolator units shall be established by the engineer responsible for the structural design *and approved by the enforcement agency. The quality control testing program shall include provisions for both prototype and production isolator units.*

1661A.3 Structural System.

1661A.3.1 Horizontal distribution of force. A horizontal diaphragm or other structural elements shall provide continuity above the isolation interface and shall have adequate strength and ductility to transmit forces (due to nonuniform ground motion) from one part of the building to another.

1661A.3.2 Building separations. Minimum separations between the isolated building and surrounding retaining walls or other fixed obstructions shall not be less than the total maximum displacement.

The separation requirements for the building above the isolation system and adjacent buildings shall be the sum of the factored displacements for each building. The factors to be used in determining separations shall be:

For elastic deformations resulting from the dynamic analysis using the Maximum Capable Earthquake unmodified by R_I or 0.7R times the elastic deformations of an adjacent fixed base structure resulting from an equivalent static analysis.

SECTION 1662A — NONBUILDING STRUCTURES

Nonbuilding structures shall be designed in accordance with the requirements of Section 1634A using design displacements and forces calculated in accordance with Section 1658A or 1659A.

SECTION 1663A — FOUNDATIONS

Foundations shall be designed and constructed in accordance with the requirements of Chapter 18A using design forces calculated in accordance with Section 1658A or 1659A.

SECTION 1664A — DESIGN AND CONSTRUCTION REVIEW

1664A.1 General. *The design review shall be the responsibility of the enforcement agency. The enforcement agency may at its discretion rquire the owner of the facility to retain an independent team to review and report on the isolation system design, site conditions and/or building configurations. The team shall serve in an advisory capacity to provide technical evaluations to the enforcement agency. The members of the independent team shall be approved by the enforcement agency.*

1664A.2 Isolation System. Isolation system design review shall include, but not be limited to, the following:

1. Review of site-specific seismic criteria, including the development of site-specific spectra and ground motion time histories, and all other design criteria developed specifically for the project.

2. Review of the preliminary design, including the determination of the total design displacement of the isolation system design displacement and lateral force design level.

3. Overview and observation of prototype testing (Section 1665A).

4. Review of the final design of the entire structural system and all supporting analyses.

5. Review of the isolation system quality control testing program (Section 1661A.2.9).

The engineer of record shall submit with the plans and calculations a statement by all members of the independent engineering team stating that the above has been completed.

1664A.3 Inspection. *A special inspector shall be hired by the owner and approved by the structural engineer of record and the enforcement agency to observe all prototype and production testing and file all reports required by Part 1 of the California Building Standards Administrative Code.*

SECTION 1665A — REQUIRED TESTS OF ISOLATION SYSTEM

1665A.1 General. The deformation characteristics and damping values of the isolation system used in the design and analysis of seismic-isolated structures shall be based on the following tests of a selected sample of the components prior to construction.

The isolation system components to be tested shall include the wind restraint system if such systems are used in the design.

The tests specified in this section are for establishing and validating the design properties of the isolation system, and shall not be considered as satisfying the manufacturing quality control tests of Section 1661A.2.9.

1665A.2 Prototype Tests.

1665A.2.1 General. Prototype tests shall be performed separately on two full-size specimens or sets of specimens, as appropriate, of each type and size of isolator unit of the isolation system. The test specimens shall include the wind restraint system, as well as individual isolator units, if such systems are used in the design. Specimens tested shall not be used for construction.

1665A.2.2 Record. For each cycle of tests the force-deflection behavior of the test specimen shall be recorded.

1665A.2.3 Sequence and cycles. The following sequence of tests shall be performed for the prescribed number of cycles at a vertical load equal to the average $D + 0.5L$ on all isolator units of a common type and size:

1. Twenty fully reversed cycles of loading at a lateral force corresponding to the wind design force.

2. Three fully reversed cycles of loading at each of the following increments of displacement: 0.2 D_D, 0.5 D_D and 1.0 D_D, 1.0 D_M.

3. Three fully reversed cycles at the total maximum displacement, $1.0D_{TM}$.

4. *($15C_{VD}/C_{AD}B_D$)*, but not less than 10, fully reversed cycles of loading at 1.0 times the total design displacement, $1.0D_{TD}$.

If an isolator unit is also a vertical load-carrying element, then Item 2 of the sequence of cyclic tests specified above shall be performed for two additional vertical load cases:

$$(1)\ 1.2D + 0.5L + |E|$$

$$(2)\ 0.8D - |E|$$

where D and L are defined in Chapter 16, Division III. The vertical test load on an individual isolator unit shall include the load increment due to earthquake overturning, $|E|$, and shall be equal to or greater than the peak earthquake vertical force response corresponding to the test displacement being evaluated. In these tests, the combined vertical load shall be taken as the typical or average downward force on all isolator units of a common type and size. *When considering the load combinations above, the load combinations of Section 1612A and the earthquake loads of Section 1630A.1.1 need not be considered.*

1665A.2.4 Units dependent on loading rates. If the force-deflection properties of the isolator units are dependent on the rate of loading, then each set of tests specified in Section 1665A.2.3 shall be performed dynamically at a frequency equal to the inverse of the effective period, T_D, of the isolated structure.

If reduced-scale prototype specimens are used to quantify rate-dependent properties of isolators, the reduced-scale prototype specimens shall be of the same type and material and be manufactured with the same processes and quality as full-scale prototypes, and shall be tested at a frequency that represents full-scale prototype loading rates.

The force-deflection properties of an isolator unit shall be considered to be dependent on the rate of loading if there is greater than a plus or minus 10 percent difference in the effective stiffness at the design displacement when tested at a frequency equal to the inverse of the effective period, T_D, of the isolated structure and when tested at any frequency in the range of 0.1 to 2.0 times the inverse of the effective period, T_D, of the isolated structure.

1665A.2.5 Units dependent on bilateral load. If the force-deflection properties of the isolator units are dependent on bilateral load, then the tests specified in Sections 1665A.2.3 and 1665A.2.4 shall be augmented to include bilateral load at increments of the total design displacement 0.25 and 1.0, 0.50 and 1.0, 0.75 and 1.0, and 1.0 and 1.0.

> **EXCEPTION:** If reduced-scale prototype specimens are used to quantify bilateral-load-dependent properties, then such scaled specimens shall be of the same type and material, and manufactured with the same processes and quality as full-scale prototypes.

The force-deflection properties of an isolator unit shall be considered to be dependent on bilateral load, if the bilateral and unilateral force-deflection properties have greater than a plus or minus 10 percent difference in effective stiffness at the design displacement.

1665A.2.6 Maximum and minimum vertical load. Isolator units that carry vertical load shall be statically tested for the maximum and minimum vertical load, at the total maximum displacement. In these tests, the combined vertical loads of $1.2D + 1.0L + |E|_{max}$ shall be taken as the maximum vertical force, and the combined vertical load of $0.8D - |E|_{min}$ shall be taken as the minimum vertical force, on any one isolator unit of a common type and size. The vertical load on an individual isolator unit shall include the load increment due to earthquake overturning, $|E|_{max}$ and $|E|_{min}$, and shall be based on peak response due to the maximum capable earthquake.

1665A.2.7 Sacrificial wind-restraint systems. If a sacrificial wind-restraint system is to be utilized, then the ultimate capacity shall be established by test.

1665A.2.8 Testing similar units. The prototype tests are not required if an isolator unit is of similar dimensional characteristics and of the same type and material as the prototype isolator unit that has been previously tested using the specified sequence of tests.

1665A.3 Determination of Force-deflection Characteristics. The force-deflection characteristics of the isolation system shall be based on the cyclic load tests of isolator prototypes specified in Section 1665A.2.3.

As required, the effective stiffness of an isolator unit, k_{eff}, shall be calculated for each cycle of loading by the formula:

$$k_{eff} = \frac{F^+ - F^-}{\Delta^+ - \Delta^-} \qquad (65\text{-}1)$$

where $F+$ and $F-$ are the positive and negative forces at Δ^+ and Δ^-, respectively.

As required, the effective damping (β_{eff}) of an isolator unit shall be calculated for each cycle of loading by the formula:

$$\beta_{eff} = \frac{2}{\pi}\left[\frac{E_{Loop}}{k_{eff}(|\Delta^+| + |\Delta^-|)^2}\right] \qquad (65\text{-}2)$$

where the energy dissipated per cycle of loading, E_{Loop}, and the effective stiffness, k_{eff}, shall be based on test displacements of Δ^+ and Δ^-.

1665A.4 System Adequacy. The performance of the test specimens shall be assessed as adequate if the following conditions are satisfied:

1. The force-deflection plots of all tests specified in Section 1665A.2 have a positive incremental force-carrying capacity.

2. For each increment of test displacement specified in Section 1665A.2.3, Item 2, and for each vertical load case specified in Section 1665A.2.3:

   2.1 There is no greater than a plus or minus 10 percent difference between the effective stiffness at each of the three cycles of test and the average value of effective stiffness for each test specimen.

   2.2 There is no greater than a 10 percent difference in the average value of effective stiffness of the two test specimens of a common type and size of the isolator unit over the required three cycles of test.

3. For each specimen there is no greater than a plus or minus 20 percent change in the initial effective stiffness of each test specimen over the *$(15C_{VD}/C_{AD}B_D)$*, but not less than 10, cycles of the test specified in Section 1665A.2.3, Item 4.

4. For each specimen there is no greater than a 20 percent decrease in the initial effective damping over for the *$(15C_{VD}/C_{AD}B_D)$*, but not less than 10, cycles of the test specified in Section 1665A.2.3, Item 4.

5. All specimens of vertical load-carrying elements of the isolation system remain stable at the total maximum displacement for static load as prescribed in Section 1665A.2.6.

1665A.5 Design Properties of the Isolation System.

1665A.5.1 Maximum and minimum effective stiffness. At the design displacement, the maximum and minimum effective stiffnesses of the isolation system, k_{Dmax} and k_{Dmin}, shall be based on the cyclic tests of Section 1665A.2.3 and calculated by the formulas:

$$k_{Dmax} = \frac{\sum |F_D^+|_{\max} + \sum |F_D^-|_{\max}}{2D_D} \quad (65\text{-}3)$$

$$k_{Dmin} = \frac{\sum |F_D^+|_{\min} + \sum |F_D^-|_{\min}}{2D_D} \quad (65\text{-}4)$$

At the maximum displacement, the maximum and minimum effective stiffness of the isolation system, k_{Mmax} and k_{Mmin}, shall be based on the cyclic tests of Section 1665A.2.3 and calculated by the formulas:

$$k_{Mmax} = \frac{\sum |F_M^+|_{\max} + \sum |F_M^-|_{\max}}{2D_M} \quad (65\text{-}5)$$

$$k_{Mmin} = \frac{\sum |F_M^+|_{\min} + \sum |F_M^-|_{\min}}{2D_M} \quad (65\text{-}6)$$

For isolator units that are found by the tests of Sections 1665A.2.3, 1665A.2.4 and 1665A.2.5 to have force-deflection characteristics which vary with vertical load, rate of loading or bilateral load, respectively, the values of k_{Dmax} and k_{Mmax} shall be increased and the values of k_{Dmin} and k_{Mmin} shall be decreased, as necessary, to bound the effects of measured variation in effective stiffness.

1665A.5.2 Effective damping. At the design displacement, the effective damping of the isolation system, β_D, shall be based on the cyclic tests of Section 1665A.2.3 and calculated by the formula:

$$\beta_D = \frac{1}{2\pi}\left[\frac{\sum E_D}{k_{Dmax} D_D^2}\right] \quad (65\text{-}7)$$

In Formula (65-7), the total energy dissipated in the isolation system per cycle of design displacement response, ΣE_D, shall be taken as the sum of the energy dissipated per cycle in all isolator units measured at test displacements, Δ^+ and Δ^-, that are equal in magnitude to the design displacement, D_D.

At the maximum displacement, the effective damping of the isolation system, β_M, shall be based on the cyclic tests of Section 1665A.2.3 and calculated by the formula:

$$\beta_M = \frac{1}{2\pi}\left[\frac{\sum E_M}{k_{Mmax} D_M^2}\right] \quad (65\text{-}8)$$

In Formula (65-8), the total energy dissipated in the isolation system per cycle of response, E_M, shall be taken as the sum of the energy dissipated per cycle in all isolator units measured at test displacements, Δ^+ and Δ^-, that are equal in magnitude to the maximum displacement, D_M.

TABLE A-16-C—DAMPING COEFFICIENTS, B_D AND B_M

EFFECTIVE DAMPING, β_D or β_M (percentage of critical)[1,2]	B_D or B_M FACTOR
≤ 2	0.8
5	1.0
10	1.2
20	1.5
30	1.7
40	1.9
≥ 50	2.0

[1]The damping coefficient shall be based on the effective damping of the isolation system determined in accordance with the requirements of Section 1665A.5.

[2]The damping coefficient shall be based on linear interpolation for effective damping values other than those given.

TABLE A-16-D—MAXIMUM CAPABLE EARTHQUAKE RESPONSE COEFFICIENT, M_M

DESIGN BASIS EARTHQUAKE SHAKING INTENSITY, ZN_v	MAXIMUM CAPABLE EARTHQUAKE RESPONSE COEFFICIENT, M_M
0.075	2.67
0.15	2.0
0.20	1.75
0.30	1.50
0.40	1.25
≥ 0.50	1.20

HISTORY NOTE APPENDIX

CALIFORNIA BUILDING CODE
(Title 24, Part 2, California Code of Regulations)

For prior history, see the History Note Appendix to the *California Building Code,* 1998 Triennial Edition published in December 1998 and effective July 1, 1999.

1. (DSA/SS 2/01) Adoption of necessary structural safety amendments to the 1998 California Building Code (CCR Title 24, Part 2) for public schools, community colleges and state-owned or state-leased essential service buildings. Approved by the Building Standards Commission on September 25, 2001 and effective on November 1, 2002.

2. (OSHPD 2/01) Adoption of the material and structural standards of the 1997 Uniform Building Code with necessary amendments (CCR, Title 24, Part 2) for hospital buildings and correctional treatment centers. Approved by the Building Standards Commission on September 25, 2001 and effective on November 1, 2002.

3. (HCD 1/01) Adoption of amendments to the California Building Code (CCR, Title 24, Part 2) for hotels, motels, lodging houses, apartment houses, dwellings, employee housing, factory-built housing, and permanent building and accessory buildings in mobile home parks and special occupancy parks. Approved by the Building Standards Commission on November 28, 2001 and effective on November 1, 2002.

4. (SFM 1/01) Adoption of various amendments to the fire and panic safety standards in the California Building Code (CCR, Title 24, Part 2) for State Fire Marshal regulated occupancies. Approved by the Building Standards Commission on November 28, 2001 and effective on November 1, 2002.

5. **Errata October 1, 2002:**

Page 2-1: Delete the words "**Note: This chapter has been revised in its entirety**" from the heading.

Page 2-18: In the last paragraph of **Section 1632.1** revise "[For OSHPD 1]" to "[For OSHPD 2]".

Page 2-38.12: In **Section 1627A**, under ***APPROVED EXISTING BUILDING***, revise "***[For OSHPD 1, 2 and 4]***" to "***[For OSHPD 1 and 4***]".

Page 2-38.23: Revise language in **Section 1632A.6**.

Page 2-38.29: Revise **Section 1637A** title to "***SITE DATA FOR HOSPITALS AND STATE OWNED OR STATE-LEASED ESSENTIAL SERVICES BUILDINGS***".

Page 2-38.45: Revise description to read "This map delineates the boundaries of the seismic hazard zones as given in Section 1629A.4.1 for hospitals and public schools in California".

Page 2-38.52: Revise item 1. in **Section 1644A.9.2.3.2** to read as follows: "*Chapter 19A, Section 1921A.4, for concrete, and Chapter 22A, Section 2210A, 2211A, items 4 and 5, for steel in structures in .....*".

Page 2-38.57: Revise **Section 1645A.7.1.3** Item 2. to read "*Nonstructural components, as listed in the 1995 California Building Code, Part 2, Title 24,...*" Revise Item 3. to read "*Equipment listed in the 1995 California Building Code, Part 2, Title 24,...*"

Page 2-38.66: In Table 16A-R-3, for *Site Class E*, in the right column replace the "0" with an "*". For *Site Class F*, in the left column replace the "0" with an "*". In Table 16AR-4, for *Site Class E*, in the right column replace the "0" with an "*".

Page 2-39: Revise title of **Section 1701.4** Item 3. to "**Spray-applied fire-resistive materials.**" Revise title of **Section 1701.5** Item 1.1 to "***[For OSHPD 2] Placing record.***"

Page 2-41: Revise title of **Section 1704.1.2.1** to "***[For HCD 1] Factory-built housing.***"

Page 2-42.2: Revise title of **Section 1704.6.4** Item 17. to "***Glued-laminated timber.***" Revise Title of Item 18 to "***Post installed anchors.***"

Page 2-96.6: In Section 1809A.5.1, replace "... *Type S3 or S4 soils, ...* " with "... Type S_D, S_E or S_F soils, ..."

Page 2-184.74: In the last line of Section 1923A, replace "*Section 1916A.4.2.*" with "*Section l916A.7.1.*"

Page 2-236.11: Revise the title of **Section 2106A.1.12.4** Item 2. to "***Shear walls.***" Revise the title of **Section 2106A.2.3.3** to "***Walls and piers.***" and the heading "***Thickness of Walls.***" to "***Thickness of walls.***"

5. (DSA/SS EF 01/03) Emergency adoption/approval of technical design and construction building standards for the adaptive reuse of existing building public school use; CCR, Title 24, Part 2. Approved by the California Building Standards Commission on May 14, 2003 and filed with Secretary of State on May 15, 2003. Effective May 15, 2003.

6. (DSA/SS EF 03/03) Emergency re-adoption/re-approval of technical design and construction building standards for the adaptive reuse of existing building public school use; CCR, Title 24, Part 2. Approved by the California Building Standards Commission on July 16, 2003 and filed with Secretary of State on May 15, 2003. Effective September 10, 2003.

7. (BSC EF 1/03) Amend Title 24, Part 2, Vol. 2, Chapters 2, 16, 17, 19, 22B and 23. Various sections. Filed with the Secretary of State on July 18, 2003. July 18, 2003.

8. (DSA/SS 3/02) Adoption of various amendments to the California Building Code (CCR, Title 24, Part 2) for seismic design of irregular structures. Approved by the Building Standards Commission on May 14, 2003 and effective July 30, 2004.

9. (OSHPD 3/02) Adoption of various amendments to the California Building Code (CCR, Title 24, Part 2) for seismic design of irregular structures. Approved by the Building Standards Commission on May 14, 2003 and effective July 30, 2004.

10. (DSA/SS EF 03/03) Emergency re-adoption/re-approval of technical design and construction building standards for the adaptive reuse of existing building public school use; CCR, Title 24, Part 2, Vol. 2. Approved as permanent by the California Building Standards Commission on January 7, 2004 and filed with the Secretary of State on January 8, 2004. Effective January 8, 2004.

11. (BSC EF 1/03) Amend Title 24, Part 2, Vol. 2, Chapters 2, 16, 17, 19, 22, 22B and 23, various sections; filed as permanent adoption with the Secretary of State on September 20, 2004; effective date September 20, 2004.

12. (OSHPD EF 01/05) Amend Title 24, Part 2, Vol. 2, Chapter 16A, Div. VI-R. Approved by the California Building Standards Commission on December 13, 2005. Filed with the Secretary of State on December 14, 2005 with an effective date of December 14, 2005.

13. (OSHPD EF 01/05) Amend Title 24, Part 2, Vol. 2, Chapter 16A, Div. VI-R. Re-adopted/approved by the California Building

Standards Commission on March 22, 2006. Filed with the Secretary of State on March 30, 2006 with an effective date of March 30, 2006.

14. (BSC 01/04, HCD 03/04, and OSHPD 02/04) Amend Title 24, Part 2, Vol. 2, Chapters 16, 18, 19, 21, 22, 22B, 23 and Appendix Chapter 16 with editorial corrections, updated seismic provision for allowable stress design for steel buildings, and adoption of the more recent edition of the national design standard for wood design and construction. Approved by the California Building Standards Commission on May 17, 2006. Filed with Secretary of State on May 23, 2006 with an effective date 180 days after publication.

15. (OSHPD 02/04 and DSA/SS 01/04) Amend Title 24, Part 2, Vol. 2, Chapters 16, 16A, 18A, 19, 19A, 21A, 22, 22A, 23A and Appendix Chapter 16A with editorial corrections, updated seismic design procedures for nonstructural building components, and adoption of the more recent edition of the national design standard for wood design and construction. Approved by the California Building Standards Commission on May 17, 2006. Filed with the Secretary of State on May 23, 1006 with an effective date 180 days after publication.