REVISION RECORD FOR THE STATE OF CALIFORNIA

EMERGENCY SUPPLEMENT

November 7, 2003

2001 Title 24, Part 2, California Building Code

PLEASE NOTE: The date of this Emergency 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suggested that the section number as well as the page number be checked when inserting this material and removing the superseded material.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It is further suggested that the superseded material be retained with this revision record sheet so that the prior wording of any section can be easily ascertained.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 <u>strongly recommended</u> that the removed pages be retained for historical reference.

VOLUME 2

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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*.

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

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

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

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

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

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

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.

1612.3.2.1 [For BSC] Alternate basic load combinations. *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.*

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

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

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

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

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

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

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:

$$1.2D + f_1 L + 1.0E_m \quad (12\text{-}17)$$

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

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 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.

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. 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. ρ = 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.

1629.4.2.1 [For BSC] Seismic Zone 4 near-source factor. *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.

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2. ρ = 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 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.

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.

> **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, 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, 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 $P\Delta$ 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 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:

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

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 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.

1630.2.3.4 [For BSC] Horizontal Distribution. *Diaphragms constructed of untopped steel decking or wood structural panels* C A C

CAC *or similar light-frame construction are permitted to be considered flexible.*

1630.2.3.5 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.

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.

1630.3.2 Determination of *R*. The notation *R* shall be taken from Table 16-N.

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. 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.

CACACACACACAC ***1630.4.2.1 [For BSC] 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. 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.

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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. 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.

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.

__1630.8.2.1.1 [For BSC] General.__ 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.

***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.

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1630.8.2.2 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 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.

***__1630.8.2.2.1 [For BSC] Detailing requirements in Seismic Zones 3 and 4.__** In Seismic Zones 3 and 4, elements supporting dis-* CAC

continuous 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. 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.

6. 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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TABLE 16.1-N—[For BSC] 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	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. Intermediate moment-resisting frame (IMRF)	4.5	2.8	35[6]
	a. Steel[6]	5.5	2.8	—
	b. Concrete[5]			
	4. Ordinary moment-resisting frame (OMRF)	3.5	2.8	—
	a. Steel[6]	3.5	2.8	—
	b. Concrete[8]	6.5	2.8	240
	5. Special truss moment frames of steel (STMF)			
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 $O_o = 2.2$.

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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Division VI-R —EARTHQUAKE EVALUATION AND DESIGN FOR RETROFIT OF [FOR BSC, DSA] EXISTING STATE-OWNED BUILDINGS [FOR OSHPD] EXISTING HOSPITAL BUILDINGS [FOR DSA/SS] EARTHQUAKE EVALUATION AND DESIGN FOR REHABILITATION OF EXISTING BUILDINGS FOR USE AS A PUBLIC SCHOOL BUILDING

SECTION 1640A — GENERAL

1640A.1 Purpose. *All modifications, alterations, and/or repairs to existing structures or portions thereof shall, at a minimum, be designed and constructed to resist the effects of seismic ground motions as provided in this division. When applicable, the structural system shall be evaluated by the design professional of record and, if not meeting or exceeding the minimum seismic design purpose of this division, shall be retrofitted in compliance with these requirements.*

[For DSA/SS] For rehabilitation of any existing buildings, the structural system shall be evaluated by the design professional in responsible charge of design and, if not meeting or exceeding the minimum seismic design purpose of this division, shall be designed and retrofitted in compliance with these requirements. Elements of structures, nonstructural components and equipment shall be evaluated and retrofitted to the seismic requirements of this division.

1640A.1.1 Minimum seismic design. *The purpose of this division is to provide a minimum level of seismic performance. At this essential life-safety level (seismic performance category SPC-2), in general, persons in and around the building will be able to safely exit or be evacuated from the building or its vicinity following an earthquake. It does not mean that persons will not be injured or not be in need of medical attention. This level of seismic performance is presumed to be achieved when a) the building has some margin against either total or partial collapse of the structural system even though significant damage may have occurred that may not be economical to repair; b) major structural elements have not fallen or been dislodged so as to pose a life-safety threat; and c) nonstructural systems or elements that are heavy enough to cause severe injuries either within or outside the building have not been dislodged so as to pose a life-safety threat. [For OSHPD 1 & 4] For buildings in seismic performance categories SPC-3 through 5, the purpose of the division is to provide the immediate occupancy level of seismic performance. At this level, the building and essential nonstructural systems will be reasonably capable of functioning following an earthquake.*

[For DSA/SS] For rehabilitation of existing buildings for use as school buildings, the performance objective of this division is to provide for Protection of Life and Property as defined in Section 1641A.

1640A.2 Applicability*. For all state-owned structures, including all buildings owned by the University of California and California State University: The requirements of this division apply wherever the structure is to be retrofitted, repaired, or modified and 1) total construction cost, not including cost of furnishings, fixtures and equipment, or normal maintenance, for the building exceeds 25 percent of the construction cost for the replacement of the existing building; or 2) changes in occupancy category; or 3) changes to structural elements reduce the lateral load capacity by more than 5 percent at any story; or 4) structural elements need repair where the damage has reduced the lateral load capacity by more than 10 percent at any story; or 5) changes in live or dead load increase the story shear by more than 5 percent. The changes in Item 1 are cumulative for past alterations to the building that occurred after adoption of this division and did not require the application of this division.*

[For DSA/SS] The requirements of this division apply to the rehabilitation of existing buildings for public school use as required per Section 4-307, Article 1, Group 1, Chapter 4, Part 1, Title 24.

1640A.2.1 [For OSHPD 1 & 4] *The requirements of this division apply to hospital buildings where Chapter 6, Part 1, Title 24, Building Standards Administrative Code, so requires, wherever the structure is to be retrofitted, repaired, or modified and: 1) there is change in occupancy; or 2) changes to structural elements that reduce the lateral load capacity by more than 5% at any story; or 3) repair of structural elements where the damage has reduced the lateral load capacity by more than 10% at any story; or 4) changes in live or dead load that increase the story shear by more than 5%; or 5) where required by Section 1638A or Chapter 6, Part 1, Title 24, Building Standards Administrative Code. The changes in Items 2), 3), and 4) are cumulative for past alterations to the building.*

1640A.2.2 [Not adopted by DSA/SS] Evaluation required. *If the criteria in Section 1640A.2 apply to the project under consideration, the design professional of record shall provide an evaluation in accordance with Section 1643A to determine the seismic performance of the building in its current configuration and condition. If the structure's seismic performance is evaluated as satisfactory and the peer reviewer(s), when Method B of Section 1648A is used, concur, then no structural retrofit is required.*

> ***EXCEPTION:*** *In some cases a technical review and evaluation may be waived under the exception of Section 1648A.1, where the life-safety threat posed by the building is clearly minimal.*

1640A.2.3 [For OSHPD 1] Retrofit required. *Where the evaluation indicates the building does not meet the SPC performance objective of this division, the owner shall take appropriate steps to ensure that the building's structural system is retrofitted in accordance with the provisions of this division. Appropriate steps are either 1) undertake the seismic retrofit as part of the modifications, alterations and/or repairs; or 2) provide a plan, acceptable to the enforcement agent, to complete the seismic retrofit in a timely manner.*

1640A.2.4 [For DSA/SS] Required evaluation and retrofit. *Where evaluation per Section 1643A indicates the building or an element of the building does not meet the performance objective of this division, the retrofit shall be designed to meet the provisions of the same methodology used in the evaluation per Sections 1640A.3, 1640A.6 or 1640A.7. Retrofit to existing construction shall comply with the detailing requirements for new construction of Part 2, Title 24, currently effective edition.*

1640A.3 [Not adopted by DSA/SS] *The modification to any existing building may be prepared in accordance with the requirements for a new building, Chapter 16A, Division VI, Part 2, Title 24, California Code of Regulations, 2001 edition.*

1640A.3.1 [For DSA/SS] *The rehabilitation of any existing building may be evaluated and designed in accordance with the requirements for a new public school building in accordance with Part 2, Title 24, currently effective edition. Evaluation and design, material testing, condition assessment and determination of equivalency with code standards for nonconforming construction shall be approved in accordance with the procedures defined in Section 1640.A.8. Equivalency with code standards for non-conforming construction shall be determined by rational analysis or testing.*

1640A.4 *The requirements of the UBC Appendix Chapter 16, Sections 1654-1665, are to apply to the use of seismic isolation for*

the repair, modification or retrofit of an existing structure. When seismic isolation or passive energy dissipation is used, the project must have project peer review as prescribed in Section 1649A.

EXCEPTION: *For hospital buildings [DSA/SS: and public school buildings] the requirements of Appendix Chapter 16A, Section 1654A through 1665A apply in lieu of those of the UBC for repair, modification or retrofit to existing hospital buildings.*

1640A.5 [Not adopted by DSA/SS] *Any construction required by this division shall include structural observation by the licensed structural engineer, civil engineer or architect of record who is responsible for the structural design in accordance with Section 1643A.12.*

1640A.5.1 [For DSA/SS] *Construction testing, inspection and observation requirements shall be as set forth in Section 4-333(a), Article 5, Group 1, Chapter 4, Part 1, Title 24, Building Standards Administrative Code, Chapter 17A, and the testing and inspection requirements of Chapters 18A through 23A, Part 2, Title 24.*

1640A.6 *Where Method B of Section 1648A is used or is required by Section 1643A.7 the proposed method of building evaluation and design procedures must be accepted by the enforcement agent prior to the commencement of the work.*

[For DSA/SS] *The rehabilitation of any existing building may be evaluated and designed in accordance with the requirements for Method B of this division, provided the methodologies for evaluation and design, and for determination of acceptance criteria for existing construction are approved in accordance with the procedures defined in Section 1640.A.8. Procedures for material testing and condition assessment shall also be approved in accordance with Section 1640.A.8. When Method B is used, the owner shall retain an independent peer review consultant to provide expertise and recommendations appropriate to the design, analysis and performance issues associated with the project in accordance with Sections 1640A.8.3 and 1648A.2.*

1640A.6.1 *The structural system allowances of Chapter 34 do not apply to any building to which Division VI-R applies.*

1640A.7 [For DSA/SS] *The rehabilitation of any existing building, unless otherwise limited per Section 1643A.7, may be evaluated and designed in accordance with the requirements for Method A of this division. Methodologies for evaluation and design, and determination of β-factors or alternate β-factors per Section 1645A.2.2 shall be approved in accordance with the procedures defined in Section 1640.A.8. Procedures for material testing and condition assessment shall also be approved in accordance with Section 1640.A.8.*

1640A.8 [For DSA/SS] Procedures for DSA Approval of the Evaluation and Retrofit Design. *During the schematic phase of the project, the owner or the design professional in responsible charge of design shall perform initial data collection and assessment of the building (Section 1640A.8.1) and prepare and sign an Evaluation and Design Criteria Report (Section 1640A.8.2). The report shall propose the methodologies for evaluation and design, and determination of acceptance criteria for nonconforming construction; and shall propose the material testing and condition assessment requirements for the rehabilitation. Two copies of the report shall be submitted to the DSA for review and approval prior to proceeding with design development of the rehabilitation. The DSA shall review the report to determine that each item per Section 1640A.8.2 has been satisfactorily addressed. If DSA determines that one or more items are not satisfactorily addressed or DSA does not concur with any of the proposals, the report shall be returned to the design professional for correction. Upon concurrence that all items have been satisfactorily addressed by the proposals in the report, DSA shall approve, sign and return a copy of the signed report to the owner.*

The approved Evaluation and Design Criteria Report establishes: 1) the criteria for the evaluation and design to be used by the project design professionals, and 2) the material testing and condition assessment requirements.

If changes to the approved criteria are determined to be necessary during design development and completion of construction documents, the project design professional shall submit an amendment to the Evaluation and Design Criteria Report to the DSA for approval. When Method B is used, the peer reviewer shall review the amendment and provide a written report to the owner and DSA in accordance with Section 1649A.

Upon completion of the design, the plans and specifications shall be submitted to the DSA for approval per the provisions of Part 1,Title 24.

1640A.8.1 **Initial data collection and assessment.** Initial data collection and assessment shall include:

1. Site visit(s) of structure.

2. Data collection of existing site conditions and building construction in accordance with Section 1643A.2.

3. Review of original plans, specifications and associated construction documents, including material test reports, geohazard and geotechnical reports. Where original building plans and specifications are not available, "as-built" plans shall be prepared that accurately depict the existing vertical and lateral structural systems, exterior elements (cladding) and nonstructural systems. 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.

4. Preliminary analysis of the lateral force resisting system that provides the basis for the proposed evaluation and design method.

1640A.8.2 Evaluation and Design Criteria Report. *The Evaluation and Design Criteria Report shall be signed by the design professional in responsible charge of the design and the project structural engineer and shall:*

1. Describe the building(s) configuration and type of construction.

2. Identify the building gravity and lateral load resisting systems, and nonstructural systems that may affect the stiffness or strength of the lateral system during a seismic event.

3. Describe the project site and identify any potential hazards from adjacent or adjoining structures or site conditions.

4. Identify potential geological hazards.

5. Describe the physical condition and known material properties of existing gravity and lateral load resisting elements or components and of exterior elements of the structure based on the data collection processes of 1650A.

6. Based on data collection and review of original construction documents and preliminary analysis, identify potential deficiencies in the gravity and lateral load resisting systems.

7. Propose the methodology for evaluation and design of the structure. Include methodology to establish modeling parameters to be used in the analysis and to establish Beta-factors or acceptance criteria for noncode-compliant construction.

8. Provide the justification for the proposed methodology. Include any preliminary calculations.

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. A [for DSA/SS: 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:*

[For DSA/SS] *C_n represents the nominal strength of a material for a given state of stress.*

ϕC_n = *Usable strength or capacity of an element as determined in the materials chapters where ϕ is the strength reduction factor.*

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 Δ_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 VI 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, I=1.0 for category SPC-2 and I=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.*

2. Where approved by the DSA, the use of nonductile or limited ductile elements may be allowed if the particular material provides the only means of ensuring compatible performance without detrimental interaction effects on the existing element material.

1643A.11 Deformation Compatibility. *The compatibility of the deformation characteristics of all elements activated in the response shall be considered, as well as the configuration of the structural and nonstructural systems; the continuity, or lack thereof, of load paths; the redundancy, if any, of these load paths; and the physical condition of the materials and elements. [For DSA/SS: The gravity load resisting members and exterior elements shall be evaluated and retrofitted to resist gravity loads combined with seismic-induced drift associated with the design ground motion.]*

1643A.12 [Not adopted by DSA/SS] Structural Observation.

1643A.12.1 *Structural observation as used in this division shall mean visits to the project site by the responsible design professional to observe existing conditions and to review the construction work for general compliance with approved plans, specifications and applicable structural regulations. Such visits shall occur at significant construction stages and at the completion of the structural retrofit. Structural observation shall be provided in Seismic Zones 3 and 4 for all structures regulated by this division. High-rise construction requires an interim progress report each month in addition to observation reports for the significant construction stages.*

The owner shall directly employ the engineer or architect, or their designee, responsible for the structural design to perform structural observation.

After each visit, the structural observer shall report in writing on the general conformity of the work to the approved plans and note any observed deficiencies to the owner's representative, project inspector, contractor and the enforcement agent. The structural observer shall notify the enforcement agent in writing in a timely manner how the structural deficiencies are to be corrected. If satisfactory resolution of the deficiency is not obtained, the enforcement agent shall be notified for any necessary action.

At the conclusion of construction, the structural observer shall submit to the enforcement agent and the owner a final written statement that the required site visits have been made, that the work, to the best of the structural observer's knowledge and belief, is or is not in general conformity to the approved plans and that the observed structural deficiencies have been resolved and/or listing those that, to the best of the structural observer's knowledge and belief, have not been satisfactorily corrected.

1643A.12.1.1 *The requirement for structural observation shall be noted and prominently displayed on the front sheet of the approved plans and incorporated into the general notes on the approved plans.*

1643A.12.1.2 Preconstruction meeting. *A preconstruction meeting is mandatory for all projects which require structural observation. The meeting shall include, but is not limited to, the design engineer or architect, structural observer, general constructor, affected subcontractors, the project inspector and a representative of the enforcement agency (designated alternates may attend if approved by the structural observer). The structural observer will schedule and coordinate this meeting.*

The purpose of the meeting is to identify and clarify all essential structural elements and connections that affect the lateral and vertical load systems and to review scheduling of the required observations for the project's structural system retrofit.

1643A.12.2 [For OSHPD] Structural observation, testing and inspections. *Construction testing, inspection and observation requirements shall be as set forth in Chapter 7, Article 4, Part 1, Title 24, Building Standards Administrative Code, Chapter 17A, and the testing and inspection requirements of Chapters 18A through 24A.*

1643A.13 Temporary Actions. *When compatible with the building use, and the time phasing for both use and the retrofit program, temporary shoring or other structural support may be considered. Temporary bracing, shoring and prevention of falling hazards can offer an affordable means of qualifying for the exception in Section 1644A.4.1.1 that allows inadequate capability in some existing elements as long as life safety can be provided.*

1643A.14 [For DSA/SS] Unreinforced Masonry. *Unreinforced masonry shall not be used to resist in-plane or out-of-plane seismic forces or superimposed gravity loads.*

EXCEPTION: *Masonry may be used to resist gravity loads when justified by rational analysis and approved by the DSA.*

1643A.15 [For DSA/SS] Wood Frame Buildings. *Horizontal diaphragms and vertical shear walls shall consist of either diagonal lumber sheathing or structural panel sheathing. Braced horizontal diaphragms may be acceptable when approved by DSA. Straight lumber sheathing may be used in combination with diagonal or structural panel sheathing as diaphragms or shear walls. Let-in bracing, plaster (stucco), gypsum wallboard and particleboard sheathing shall not be allowed to resist seismic forces.*

SECTION *1644A* — METHOD A

1644A.1 General. *Structures shall be designed for seismic forces coming from any horizontal direction. The design seismic forces may be assumed to act nonconcurrently in the direction of each principal axis of the structure, except as required by Section 1646A.1.4. Seismic dead load, W, is the total dead load and applicable portions of other loads listed below.*

1644A.1.1 *In storage and warehouse occupancies, a minimum of 25 percent of the floor live load shall be applicable.*

1644A.1.2 *Where a partition load is required in the floor design, a load of not less than 10 pounds per square foot (psf) (0.48 kN/m^2) shall be included.*

1644A.1.3 *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.*

1644A.1.4 *Total weight of permanent equipment shall be included.*

1644A.2 *Determine the most applicable complying or essentially complying structural system as described in Section 1629A.6. All elements that are capable of providing significant resistance to the actions of lateral forces shall be included in the system.*

EXCEPTION: *Elements made of noncomplying materials and/or details, and nonstructural components may be omitted from the system provided that their rigidity, capacity and load-deformation behavior are established for use in the investigation of the effects of these elements on the structural system as required by Sections 1646A.2.4 and 1646A.2.4.1.*

1644A.2.1 *Classify each element included in the assigned structural system and foundation as being either "ductile," "limited-ductile," or "nonductile" according to its relative compliance with required provisions and/or its ability to deform beyond the nominal strength level without an abrupt or significant loss of resistance.*

All elements shall be considered nonductile if they do not comply or do not essentially comply with the requirements for ductile elements. The limited-ductile classification must be established by related empirical data and analysis, or by meeting the requirements given in Section 1645A.

Section 1645A provides a listing of code dates and extra provisions that apply for given elements and materials to qualify for the "code-complying or ductile" classification. Section 1645A also provides the procedures and criteria that apply for the "limited-ductile" and "nonductile" classification.

The stiffness and nominal strength or capacity C_n of each element shall be determined for each possible mode of failure of the element.

1644A.2.2 *Evaluate the uplift and/or sliding resistance of joints and connections at all levels including the diaphragm-to-wall or frame connection and collectors, and including the foundation soil-structure interface along with the soil compressive resistance to seismic forces; the contribution of existing piles and caissons shall be considered where they occur.*

1644A.2.3 Modeling requirements. *The mathematical model of the physical structure shall comply with Section 1630A.1.2.*

1644A.3 General. *Structural systems shall be classified with the requirements of Section 1629A.6 as one of the types listed in Table 16A-N and defined in this section. The system selected for an existing building to be most appropriate for a given existing building may contain noncomplying elements and/or elements that essentially comply to the required provisions and details for that system provided that all the noncomplying and essentially complying elements have been properly classified as "nonductile," "limited-ductile," or "ductile" and the corresponding β values are applied to their seismic load.*

1644A.3.1 *The system R-value shall be taken as 4.5 for all existing structural systems except for the following conditions. [For DSA/SS: For systems in which the R-value in Table 16A-N is lower than 4.5, the lower R-value shall be used.]*

1644A.3.1.1 *R may be taken as 5.5 if the system constructed meets the requirements for a Building Frame System as defined in Section 1629A.6.3. [For DSA/SS: For systems in which the R-value in Table 16A-N is lower than 5.5, the lower R-value shall be used.]*

1644A.3.1.2 *For structural systems designed to meet all of the seismic provisions of the 1976 or later editions of the UBC, R may be taken as appropriate R value given in Table 16A-N for the corresponding basic structural system.*

1644A.4 Static Force Procedures.

1644A.4.1 Design base shear. *The total design base shear in a given direction shall be determined from the following formula:*

$$V = \frac{H\,C_v\,I\,W}{RT} \qquad (44A\text{-}1)$$

The total design base shear need not exceed the following:

$$V = \frac{2.5\,H\,C_a\,I\,W}{R} \qquad (44A\text{-}2)$$

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

$$V = 0.11\,H\,C_a\,I\,W \qquad (44A\text{-}3)$$

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

$$V = \frac{0.8\,H\,Z\,N_v\,I\,W}{R} \qquad (44A\text{-}4)$$

1644A.4.1.1 Strength basis for evaluation and design. *Elements subject to seismic load E due to the specified base shear V shall have the usable strength capacity ϕC_n to resist the following load combinations:*

1. For the case where the actions D, L and E are all in the same sense,

$$\phi C_n = 1.05D + 0.25L + \beta E \qquad (44A\text{-}5)$$

where the live load L is the realistic live load, but shall not be less than the design load specified for the occupancy.

2. For the case where the action E is opposite to the sense of D,

$$\phi C_n = \beta E - 0.9D \qquad (44A\text{-}6)$$

In the load combinations (44A-5) and (44A-6), the seismic load penalty factor β represents the limited inelastic deformation capability of nonductile and limited-ductile elements for an associated mode of failure. Values of β for specific types of elements and modes of failure are given in Section 1645A.

EXCEPTION: *See Exceptions 1 and 2 in Section 1643A.9.*

1644A.4.1.2 Allowable or working stress basis for evaluation and design. *Allowable or working stress method along with the one-third allowable stress increase as permitted by Section 1612A.3.2 may be used to establish the allowable or working stress capacity C_w of an element. The capacity C_w shall meet the following load combination requirements:*

3. For the case where the actions D, L, and E are all in the same sense,

$$C_w = D + L + \frac{\beta E}{1.4} \qquad (44A\text{-}7)$$

4. For the case where the action E is opposite to the sense of D,

$$C_w = \frac{\beta E}{1.4} - 0.9D \qquad (44A\text{-}8)$$

EXCEPTION: *Section 1644A.4.1.2 may not be used for reinforced concrete.*

1644A.4.2 Structure period. *The value of T shall be determined in the same manner as for a new building contained in Section 1630A.2.2.*

1644A.5 Combinations of Structural Systems—General. *Where combinations of structural systems are incorporated into the same structure, the same requirements shall be satisfied as for a new building of Section 1630A.4 shall be satisfied.*

1644A.6 Vertical Distribution of Force. *The total force shall be distributed over the height of the structure in conformance with the requirements of Section 1630A.5 for new buildings.*

1644A.7 Horizontal Distribution of Shear. *The design story shear shall be distributed over the height of the structure in conformance with the requirements of Section 1630A.6 for new buildings.*

1644A.8 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 in conformance with the requirements of Section 1630A.7 for new buildings.*

1644A.9 Overturning.

1644A.9.1 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, 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, 2211.4, Items 4 and 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, 2211.4, 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. PΔ 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, PΔ 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:

$$F_g \text{ shall not be less than } 0.7\, H\, C_a\, I_p\, W_p \text{ and}$$
$$\text{Need not be more than } 4\, H\, C_a\, I_g\, W_g \qquad (44A\text{-}16)$$

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.*

[For OSHPD] *The total design lateral force , F_p, shall be determined from the following formula:*

$$F_p = \beta H C_a I_p W_p \qquad (44A\text{-}14)$$

Alternatively, F_p may be calculated using the following formula:

$$F_p = \beta a_p H C_a (1 + 3h_x/h_r) W_p / R_p \qquad (44A\text{-}15)$$

Except that:

$$F_p \text{ shall not be less than } 0.7\beta H C_a I_p W_p \text{ and}$$
$$\text{need not be more than } 4\beta H C_a I_p W_p. \qquad (44A\text{-}16)$$

Where:

β is the value for the connection, not the element to which it is attached. The values of β for connections, bracing and materials shall be as prescribed in Section 1645A.7.2.

I_p is the value used for the structure selected from Table 16A-K.

h_x is the elemement or component attachment elevation with respect to grade. h_x shall not be taken less than 0.0.

h_r is the structure roof elevation with respect to grade. The value of h_x/h_r need not exceed 1.0.

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.

R_p is 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 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.

> ***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 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.

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:

$$F_p = \beta H C_a I_p W_p \qquad \text{(44A-14)}$$

Alternatively, Fp may be calculated using the following formula:

$$F_p = \beta \frac{a_p H C_a I_p}{R_p} \left(1 + 3\frac{h_x}{h_r}\right) w_p \qquad \text{(44A-15)}$$

Except that:

F_p *shall not be less than* $0.7\beta H C_a I_p W_p$ *and need not be more than* $4\beta H C_a I_p W_p$. *(44A-16)*

WHERE:

- β = *the value for the connection, not the element to which it is attached. The values of β for connections, bracing and materials shall be as prescribed in Section 1645A.7.2.*
- 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.

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.

2. Boundary element or boundary zone confinement ties.

3. Horizontal shear steel and its anchorage in boundary element or boundary zone.

4. Location and characteristics of construction joints.

5. Relative stiffness and friction resistance of soil-footing interface to determine if the effects of foundation rotation and/or horizontal slip need to be included in the analytical model (Section 1646A.1.3.4)

6. Diaphragm drag or collector elements and connection of diaphragm to wall or braced frame (Sections 1646A.1.3.3 and 1646A.1.3.4).

7. Spandrel capacity to resist flexure and vertical shear.

8. Pile-to-footing connection resistance to tension due to overturning moment (Section 1644A.9.3).

1645A.3.2.4 *Any diaphragm 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. Thickness of slab and positioning of reinforcing.

2. Shear connection to walls.

3. Shear reinforcing.

4. Reinforcing around openings.

5. Chord element.

6. Drag or collector elements.

1645A.4 Masonry.

1645A.4.1 Ductile or code-complying. *Any element in essential conformance with the seismic requirements of Chapter 21A, Sections 2106A.1.12.4 [for DSA/SS: 2107A.1.3] and 2108A.2.3.8, may be classified as ductile and the β value taken as 1.0.*

> ***EXCEPTION:*** *Any shear wall pier and spandrel element having height or clear span to depth ratios greater than 2 shall comply with Section 2108A.2.6 (wall frames) to be classified as ductile; otherwise, it shall be classified as a limited-ductile element with $\beta = 2.5$ or greater.*

1645A.4.2 Limited-ductile. *Any masonry element in essential conformance with the 1994 UBC Sections 2106.1.12.3 (special provisions for Seismic Zone 2), and 2108.2.3.8 (seismic design provisions), shall be classified as limited ductile and assigned a β value equal to or greater than 2.5 for all modes of failure.*

1645A.4.3 Nonductile. *Systems and elements that do not comply with Section 1645A.4.1 or 1645A.4.2 shall be classified as nonductile, with a corresponding β value equal to or greater than 4.5 for all modes of failure, except where Section 1645A.2 allows use of another value. Section 1645A.2.2 analysis shall consider at a minimum:*

Wall elevation:

1. Horizontal and vertical reinforcing.

2. Reinforcing at edges of wall and openings.

3. Slenderness proportions of wall piers and spandrels.

4. Height-to-thickness ratio of wall.

5. Special reinforcing for slender piers.

6. Spandrels and openings.

7. Diaphragm connections.

8. Quality of dry-pack mortar joints and grouting of shear friction dowels at the horizontal joint between the top of masonry walls and adjoining reinforced concrete beams or slabs.

Grouting:

1. Grouting of cells, particularly those containing reinforcing steel.

2. Potential for incomplete grouting because of large or pairs of reinforcing bars in one cell or in bond beams.

3. Bond beams at required spacing and location.

4. Splice lengths for vertical and horizontal reinforcing.

5. Quality of construction joint at base of wall and vertical control joints.

Wall and diaphragm connections:

1. Wall joints and separations for pounding or hard-spot effects.

2. Wall-reinforcing ties at wall intersections and corners.

3. Wall-to-diaphragm connections.

1645A.4.4 [Not adopted by DSA/SS] *Where an element is unreinforced masonry, then the seismic capacities shall be determined in the manner consistent with the testing requirements specified in the Uniform Code for Building Conservation (UCBC). [For OSHPD 1] For hospital buildings, the use of unreinforced bearing wall masonry elements for seismic resistance is not allowed.*

1645A.4.5 [Not adopted by DSA/SS] *For masonry buildings with wood diaphragms, the requirements for Flexible Diaphragm-Rigid Wall Buildings of Uniform Code for Building Conservation, Appendix Chapter 5, shall apply. [For OSHPD 1] For masonry hospital buildings with wood diaphragms, the requirements of these regulations for new construction shall apply. The procedures for Flexible Diaphragm-Rigid Wall Buildings of the UCBC. Appendix Chapter 5, may be used, subject to the limitations of these regulations for new construction.*

1645A.4.6 [Not adopted by DSA/SS] Inspections required. *Unless inspection reports from the original construction are available and acceptable, then appropriate destructive testing and inspections shall be performed, including core testing and removing masonry. For each wall that is part of the lateral-resisting system, at least one of each of the following tests shall be done:*

1. Core test to determine the strength of the masonry, the bond between the grout and the masonry units, and the placement and size of reinforcing steel in the walls.

2. At sections of the construction joints where masonry adjoins concrete at slab, concrete framing or foundations, determine the value of shear transfer.

1645A.5 Structural Steel.

1645A.5.1 Welded steel moment frame elements. *The SAC references in this section are to the SAC Interim Guidelines for the Evaluation, Repair, Modification, and Design of Welded Steel Moment Frame Structures, FEMA 267, August, 1995.*

[For DSA/SS] *The references in this section are to the following SAC Joint Venture recommendations and AISC provisions:*

1. FEMA 350, Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings (July 2000).

2. FEMA 351, Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel-Moment Frame Buildings (July 2000).

3. FEMA 352, Recommended Post Earthquake Evaluation and Repair Criteria for Welded Moment-Frame Construction for Seismic Applications (July 2000).

4. AISC Seismic Provisions for Structural Steel Buildings, April 1997, with Supplement 2.

1645A.5.1.1 [Not adopted by DSA/SS] *Any frame element in conformance with the requirements of Chapter 7 of the FEMA 267 requirements for new construction or which has had its connections repaired and modified in accordance with the recommendations of Chapter 6 may be classified as ductile and the β value taken as 1.0.*

1645A.5.1.1.1 [For DSA/SS] *Any frame, including its connections, in conformance with the requirements of Part 2, Title 24 for new school buildings may be classified as ductile and the β-value taken as 1.0.*

1645A.5.1.2 [Not adopted by DSA/SS] *For any frame element in essential conformance with the requirements of 1976 UBC Section 2722 for Seismic Zones 3 and 4 or later editions of the UBC, where the structure:*

1. Has not experienced potentially damaging ground motions in an earthquake that by the recommendations of Chapter 4 of FEMA 267 require inspection may be classified as limited-ductile and the β value taken as 1.5; or

2. Has been repaired and evaluated in conformance with the recommendations of Chapters 4 and 6 of FEMA 267 may be classified as limited-ductile and the β value taken as 1.5 or greater; or

3. Has been repaired in conformance with the requirements of Chapter 6 of FEMA 267 requirements for the repair may be classified as limited-ductile and the β value taken as 2.0 or greater [for OSHPD]; or

4. Has been inspected in accordance with the requirements of Chapters 3 and 4 of FEMA 267,

4.1 Connections that have been inspected but not repaired or modified may be classified as limited-ductile and the β value taken as $1.5 + 0.5d_i$, where d_i is the damage index for the inspected connections.

4.2 Connections that have not been inspected may be classified as limited-ductile and the β value taken as 1.5 + 0.5DA, where DA is the average damage index for the inspected connections.

4.3 Connections that have been modified in accordance with the recommendations of Chapters 4 and 6 of FEMA 267 may be classified as ductile and the β value taken as 1.0.

4.4 Connections that have been repaired in accordance with the recommendations of Chapters 4 and 6 of FEMA 267 may be classified as limited-ductile and the β value taken as 1.5; or

5. Has not been inspected in accordance with the requirements of Chapters 3 and 4 of FEMA 267, the connections of the structure may be classified as limited-ductile and the β value taken as 3.0 or higher.

1645A.5.1.2.1 [For DSA/SS] *For any frame element, including connections, in essential conformance with the requirements of 1976 UBC Section 2722 for Seismic Zones 3 and 4 or later editions of the UBC, where the structure:*

1. Has been inspected and evaluated in accordance with Chapter 4 of FEMA 352 and repaired or retrofitted in accordance with the recommendations of Chapter 6 of FEMA 352 may be classified as limited-ductile and the β-value taken as 1.5; or

2. Has been repaired or retrofitted in accordance with the recommendations of Chapter 6 of FEMA 352 but not inspected and evaluated in accordance with Chapter 4 of FEMA 352, may be classified as limited-ductile and the β-value taken as 2.0 or greater; or

3. Has been inspected in accordance with the requirements of Chapters 3 and 4 of FEMA 352:

3.1 Connections within a connection group in which all connections in the group have been inspected (Method 1, Level 1 Evaluation) but not repaired or retrofitted may be classified as limited-ductile and the β value for connections may be taken as $1.5 + 1.5\ d_i$, where d_i is the damage index for the connection group at floor "i". Damaged connections, $d_j = 1$ where d_j is the damage index for the j^{th} connection in the group, shall be repaired or retrofitted in accordance with Section 1643A.10.1.

3.2 For buildings that may have experienced potentially damaging ground motions, connections within a connection group in which inspections have been conducted on a sample of connections (Method 2, Level 1 Evaluation) but not repaired or retrofitted may be classified as limited-ductile and the β value taken as $1.5 + 1.5d_{avg}$, where d_{avg} is the average damage index for the evaluated connections within the group at that floor. The use of this approach shall be approved by the DSA in the Evaluation and Design Criteria Report. When damage is found in any connection, $D_j = 1$, every connection shall be inspected. Damaged connections shall be retrofitted in accordance with Section 1643A.10.1; or

4. Has not been evaluated, repaired nor retrofitted in accordance with the requirements of FEMA 351 or 352, the connections of the structure may be classified as limited-ductile and the β value taken as 3.0 or higher.

1645A.5.1.3 *Any bolted frame element in conformance with the requirements of the 1997 UBC for bolted connections may be classified as ductile and the β value taken as 1.0. Where the frame element at least meets the requirements of 1976 UBC but not the 1997 requirements, then the element may be classified as limited-ductile and the β value taken as 1.5 or higher.*

1645A.5.1.4 *Any structural element having moment capacity but not qualifying as ductile under any UBC code provisions since 1976 may be classified as limited-ductile and the β value taken as 3.0 or higher.*

1645A.5.1.5 *Any truss girder or knee brace frame element may be classified as limited-ductile and the β value taken as 2.0 or higher.*

1645A.5.1.6 *Elements of frames with lateral girder buckling and/or noncompact column sections may be classified as limited-ductile and the β value taken as 2.0 or higher.*

1645A.5.2 Braced steel frame elements.

1645A.5.2.1 *Any braced frame element in conformance with the requirements of 1997 UBC for braced frames may be classified as ductile and the β value taken as 1.0.*

1645A.5.2.2 *Any braced frame element in conformance with the requirements of 1997 UBC, except that the b/t ratio exceeds the 1997 requirements for special braced frames may be classified as limited-ductile and the β value taken as 1.5 for a special and 2.5 for ordinary braced frames.*

1645A.5.2.3 *Any braced frame element where the connection gusset plate is subject to buckling may be classified as limited-ductile and the β value taken as 2.0 or greater.*

1645A.5.2.4 *Any braced frame element with tension-only bracing, with rods or angles, may be classified as limited-ductile and the β value taken as 3.0 or greater.*

1645A.6 Wood and Other Sheathing Materials.

1645A.6.1 *Wood elements and other sheathing materials that essentially comply with the 1976 UBC Chapter 25, Wood, and Chapter 47, Installation of Wall and Ceiling Coverings, or the equivalent sections of later editions may be classified as ductile and assigned a β value of 1 as given in Table 16A-R-2.*

EXCEPTION: *[Not adopted by DSA/SS] Let-in bracing, plaster (stucco), gypsum wallboard and particle board sheathing shall be classified as limited-ductile or nonductile and assigned a β value given in Table 16A-R-2.*

1645A.6.2 *Any element not meeting the requirements of Section 1645A.6.1 shall be classified as nonductile, with a corresponding β value equal to or greater than that given in Table 16A-R-2, except where Section 1645A.2 allows use of another value. The Section 1645A.2.2 analysis shall consider at a minimum:*

1. Anchoring attachment of tile or other heavy roofing elements, and chimneys.

2. In-plane and out-of-plane bracing of roof framing and trusses.

3. Wall-to-diaphragm connection for framing perpendicular to wall.

3.1 Indirect shear path.

4. Wall-to-diaphragm connection for framing parallel to wall.

5. Shear transfer connection from shear panels or walls to framing and/or collector elements at top and bottom of shear walls.

6. Wall hold-down details between floors and a positive load path to foundation at base of wall.

7. Attachment of sheathing and stucco to transfer shear from wall to foundation.

8. Sill bolts to transfer from wall framing to foundation.

9. Scabs and blocking and connections needed to transfer shear through floor framing.

1645A.7 [OSHPD 1: Nonstructural Components and Systems Critical to Patient Care]

1645A.7.1 *The requirements of Section 1643A.9 applies to the following systems for the indicated nonstructural performance levels NPC-1 through NPC-5, as defined in Chapter 6, Part 1, Title 24, Building Standards Administrative Code:*

EXCEPTION: *All exterior nonbearing, nonshear wall panels or elements that are not considered as part of the structural system shall be assessed using the requirements of Section 1646A.2.4.2, not Section 1645A.7.*

1645A.7.1.1 *For the NPC-1 performance level the requirements of Section 1643A.9 for nonstructural elements and systems do not apply.*

1645A.7.1.2 *For the NPC-2 performance level the requirements of Section 1643A.9 must be met by the following systems:*

1. Communications systems;

2. Emergency power systems;

3. Bulk medical gas systems;

4. Fire alarm systems; and

5. Emergency lighting equipment and signs in the means of egress.

1645A.7.1.3 *For the NPC-3 performance level the requirements of Section 1643A.9 must be met by the following systems in critical care areas, clinical laboratory, service spaces, pharmaceutical service spaces, radiological service spaces, and central and sterile supply areas:*

1. Those required by Section 1645A.7.1.2;

2. Nonstructural components, as listed in the 1995 California Building Code, Title 24, Part 2, Table 16A-O; and

3. Equipment listed in the 1995 California Building Code, Part 2, Title 24, California Code of Regulations, Table 16A-O "Equipment" including equipment in the physical plant that services these areas.

EXCEPTIONS: *For Section 1645A.7.1.3, seismic restraints need not be provided for cable trays, conduit and HVAC ducting. Seismic restraints may be omitted from piping systems, provided that an approved method of preventing release of the contents of the piping system in the event of a break is provided.*

2. For Section 1645A.7.1.3, only elevator(s) selected to provide patient, surgical, obstetrical and ground floors during interruption of normal power need meet the structural requirements of Part 2, Title 24.

4. Fire sprinkler systems must comply with the bracing and anchorage requirements of NFPA-13, 1994 edition, or subsequent applicable standards.

EXCEPTION: *Acute care hospital facilities in both a rural area as defined by Section 70059.1, Division 5 of Title 22 and Seismic Zone 3 shall comply with the bracing and anchorage requirements of NFPA 13, 1994 edition or subsequent applicable standards as specified in Article 11, Chapter 6, Part 1, Title 24, Building Standards Administrative Code.*

1645A.7.1.4 *For the NPC-4 performance level the requirements of Section 1643A.9 must be met by the following systems:*

1. Those required by Section 1645A.7.1.3; and

2. All architectural, mechanical and electrical systems, components and equipment and hospital equipment bracing and anchorages.

1645A.7.1.5 *For the NPC-5 performance level, the requirements of Section 1643A.9 must be met by the following systems:*

1. Those required by Section 1645A.7.1.4;

2. On-site supplies of water and holding tanks for waste water, sufficient for 72 hours of emergency operations, that are integrated into the building plumbing system, including any alternative hook-ups to allow the use of transportable water and sanitary waste water disposal; and

3. On-site emergency system as defined within Part 3, Title 24; this includes task lighting, selected outlets and ventilation systems, radiological service, and on-site fuel supply for 72 hours of acute care operation.

1645A.7.2 *The β values to be used in Section 1644A.13.1 and Formula 44A-11 for the connection and bracing of nonstructural elements, equipment and systems shall be determined as follows:*

1645A.7.2.1 Ductile or Code Complying: *Any element constructed under a permit issued by OSHPD may be classified as ductile and the β value taken as 1.0.*

1645A.7.2.2 Nonductile: *Any element whose construction was completed before 1973 shall be classified as nonductile and β taken as 4.0, except where Section 1645A.2 or 1646A.2.4.2 allow use of another value. The Section 1645A.2.2 analysis shall consider at a minimum:*

1. The anchorage of the element to the structural system.

2. The yielding and post yielding, buckling, and/or failure behavior of the connection and/or bracing system.

3. The attachment of supported equipment to the brace and bracing system and the ability to reliably develop yielding in the connection and/or brace.

4. Stability of the bracing system under both in-plane and out-of-plane displacements of the supported equipment.

1645A.7.2.3 Limited-Ductile: *Systems and elements that do not comply with Section 1645A.7.2.1 or 1645A.7.2.2 shall be classified as limited-ductile, with corresponding β value equal to or greater than 2.5 for all modes of failure, except where Section 1645A.2 allows use of another value. The Section 1645A.2.2 analysis shall consider at a minimum the items 1 through 4 listed in Section 1645A.7.2.2.*

> ***EXCEPTION:*** *All drilled mechanical anchors subject to tension loads shall be classified as nonductile, except that they may be classified as ductile where tension testing, consistent with OSHPD, DSA or comparable procedures, has been completed for the anchors and the results of testing are evaluated as acceptable.*

1645A.8 [For DSA/SS] Nonstructural Components and Systems

1645A.8.1 *The requirements of Section 1643A.9 apply to the nonstructural components and systems for public schools:*

> ***EXCEPTION:*** *All exterior nonbearing, nonshear wall panels or elements that are not considered part of the structural system shall be assessed using the requirements of Section 1646A.2.4.2, not Section 1645A.7.*

1645A.8.2 *The β values to be used in Section 1644A.13.1 and Formulas 44A-14, 44A-15 and 44A-16 for the connection and bracing of nonstructural elements, equipment and systems shall be determined as follows:*

1645A.8.2.1 Ductile or Code Complying: *Any element or connection satisfying the cyclical requirements to qualify as a "ductile element" per Section 1641A may be classified as ductile and the β-value taken as 1.0. Elements constructed to the requirements of prior editions of Part 2, Title 24, deemed to be equivalent to the current code may be classified as ductile and the β-value taken as 1.0 when approved by the DSA.*

1645A.8.2.2 Nonductile: *Any element or connection may be classified as nonductile and β taken as 4.0.*

1645A.8.2.3 Limited-Ductile: *Systems and elements that qualify use of another β value in accordance with Section 1645A.2 may be classified as limited-ductile. The Section 1645A.2.2 analysis shall consider at a minimum:*

1. The anchorage of the element to the structural system.

2. The yielding and post yielding, buckling and/or failure behavior of the connection and/or bracing system.

3. The attachment of supported equipment to the brace and bracing system and the ability to reliably develop yielding in the connection and/or brace.

4. Stability of the bracing system under both in-plane and out-of-plane displacements of the supported equipment.

SECTION 1646A — DETAILED SYSTEMS DESIGN REQUIREMENTS

1646A.1 General. *All structural framing systems shall comply with the requirements of Section 1643A.9. The individual elements shall have the usable strength capacity ϕC_n or the allowable capacity C_w to resist the prescribed seismic load combinations. In addition, such framing systems and elements shall comply with the detailed system design requirements contained in Section 1646A.*

1646A.1.1 *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.*

1646A.1.2 *Consideration shall be given at each story level to the effects of uplift, reversed moment and/or sliding, caused by seismic loads, as prescribed in Sections 1646A.1.3 and 1646A.2.4.2.*

1646A.1.3 *The following provisions apply for all levels of the superstructure and its connection to the foundation structure.*

1646A.1.3.1 [Not adopted by DSA/SS] *Overturning moment tension resistance for elements and connections: If the tension action due to $\Omega_o E$ - 0.9 D > 0, then the usable tensile strength ϕC_n shall equal or exceed the greater of the tension due to $\Omega_o E$ − 0.9 D or E/14 for semiductile and brittle elements; and E − 0.9 D or E/14 for ductile elements.*

1646A.1.3.1.1 [For DSA/SS] *Overturning moment tension resistance for elements and connections: If the tension action due to Ω_o E - 0.9 D > 0, then the usable tensile strength ϕC_n shall equal or exceed the greater of the tension due to Ω_o E-0.9 D or E/14 for limited ductile and nonductile elements; and E-0.9 D or E/14 for ductile elements.*

1646A.1.3.2 [Not adopted by DSA/SS] *Reversed moment opposite to that caused by gravity loads in beams, slabs and spandrels: If the flexural action due to $\Omega_o E$ − 0.9 D > 0, then the usable flexural strength ϕC_n shall equal or exceed the greater of the moment due to $\Omega_o E$ − 0.9 D or E/14 for semiductile and brittle elements; and E − 0.9 D or E/14 for ductile elements.*

1646A.1.3.2.1 [For DSA/SS] *Reversed moment opposite to that caused by gravity loads in beams, slabs and spandrels: If the flexural action due to Ω_o E - 0.9 D > 0, then the usable flexural strength ϕC_n shall equal or exceed the greater of the moment due to Ω_o E-0.9 D or E/14 for limited ductile and nonductile elements; and E-0.9 D or E/14 for ductile elements.*

1646A.1.3.3 *Resistance to sliding or slip of horizontal joints and/or the in-plane joints between diaphragms and walls or frames shall be such that the usable horizontal shear strength ϕC_n equals or exceeds the shear on the joint due to E.*

1646A.1.3.4 *For the following conditions:*

1. Foundations at the soil-structure interface;

2. Horizontal construction joints in shear walls; or

3. Diaphragm collectors, joints or connections of diaphragms to shear walls or frames.

If the strength capacity to resist overturning and/or sliding is exceeded by the application of a load combination of

$$\Omega_o E \pm 0.9D \qquad (46A\text{-}1)$$

then the deformations to be used in the investigation required by Section 1646A.2.4 shall be two times the displacement prescribed by Section 1646A.2.4.

1646A.1.4 *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:*

1. The structure has plan irregularity Type E as given in Table 16A-M.

[For DSA/SS] *The structure has plan structural irregularity Type 5, Nonparallel systems, as given in Table 16A-M.*

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

[For DSA/SS] *The structure has plan structural irregularity Type 1, Torsional irregularity, as given in Table 16A-M for both major axes.*

3. 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 allowable axial load.*

The requirement that orthogonal effects be considered may be satisfied by designing such elements for 100 percent of the prescribed seismic forces in one direction plus 30 percent of the prescribed 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.

1646A.2 Structural Framing Systems.

1646A.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.*

1646A.2.2 Detailing for combinations of systems. *For components common to different structural systems, the more restrictive detailing requirements shall be used.*

1646A.2.3 Connections. *Connections that resist seismic forces shall be designed and detailed on the drawings.*

1646A.2.4 Deformation compatibility. *All vertical load-bearing elements not included as a part of the lateral-force-resisting system shall be investigated and shown to be adequate for vertical load-carrying capacity when displaced (0.7)R times the displacements resulting from the required design lateral forces given in Section 1644A.4. A representation of cracked section stiffness properties for reinforced concrete and masonry elements [for DSA/SS:, as approved by the DSA in the Evaluation and Design Criteria Report,] shall be used in the calculation of the displacements. The displacements shall include diaphragm deformation.*

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. The effects of adjoining rigid and exterior elements shall be considered as follows.

1646A.2.4.1 Adjoining rigid elements. *Any framing elements, including those of the lateral-force-resisting system, may be enclosed by or adjoined by more rigid elements, which would tend to limit the frame from resisting lateral forces, where it can be shown that the action or failure of the more rigid elements will not impair the vertical and lateral-load-resisting ability of the frame. Where failure of the more rigid elements is indicated, then the life-safety consequences due to debris and other falling hazards shall be investigated and mitigated where appropriate.*

1646A.2.4.2 Exterior elements. *Exterior nonbearing, nonshear wall panels or elements that are attached to or enclose the exterior of the structure shall be designed to resist the forces per Formula (44A-14) or (44A-15) and shall accommodate movements of the structure resulting from lateral forces or temperature changes. In order to qualify for the "code-complying or ductile" classification, 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 or the story drift corresponding to the (0.7) R factored displacements given in Section 1646A.2.4, (0.015 h) or 0.5 inch (13 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 so as to preclude fracture of the concrete or brittle failures at or near welds.

4. The body of the connection shall be designed for one and one-third times the force determined by Formula (44A-14) or (44A-15) 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 four times the force determined by Formula (44A-14) or (44A-15) where $R_p = 3.0$ and $a_p = 1.0$.

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

1646A.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. At 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 H C_a I 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 H C_a I times the dead plus live load.

1646A.2.6 Collector elements. *Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the building to the element providing the resistance to those forces. These elements shall be classified as "ductile," "limited-ductile," or "nonductile" and assigned the corresponding β factor for the seismic load. Unless an element can qualify for a β value given in Section 1645A, β shall be 1.00 for code-complying or ductile elements, and 4.00 for nonductile elements.*

1646A.2.7 Concrete frames. *In order to qualify for the "code-complying or ductile" classification and use of an R greater than 5.5, concrete frames that are part of the lateral-force-resisting system shall conform to the requirements of Division VI [for DSA/SS: of Part 2, Title 24] for special moment-resisting frames in Seismic Zones 3 and 4.*

1646A.2.8 Anchorage of concrete or masonry walls. *Concrete or masonry walls shall be anchored to all floors and roofs that provide lateral support for the wall. The anchorage shall provide a positive direct connection between the wall and floor or roof construction capable of resisting the horizontal forces specified in Section 1611A or 1644A.13.1. Requirements for developing anchorage forces in diaphragms are given in Section 1646A.2.9. Diaphragm deformation shall be considered in the design of the supported walls.*

1646A.2.9 Diaphragms.

1646A.2.9.1 *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 in-*

dividual loading and continue to support the prescribed loads. For the purpose of this evaluation, the deflection of the diaphragm shall be (0.7)R_w times the deflection Δ_S due to F_{px} with $\beta = 1.00$ in Formula (46A-2).

1646A.2.9.2 *Floor and roof diaphragms shall be designed to resist the forces determined in accordance with the following formula:*

$$F_{px} = \beta \frac{F_t + \sum_{i=x}^{n} F_i}{\sum_{i=1}^{n} w_i} w_{px} \qquad (46A\text{-}2)$$

The force F_{px} determined from Formula (46A-2) need not exceed $1.0 \beta H C_A I w_{px}$, but shall not be less than $0.5 \beta H I w_{px}$. The β value to be used in the capacity analysis is the factor appropriate to the element and condition of loading.

The actions on an element due to the force F_{px} are the seismic load E. The value of β shall be 1.00 for code-complying or essentially complying elements and 4.00 for nonductile elements, unless the element qualifies for a lower value as given in Table 16A-R-1 or 16A-R-2.

1646A.2.9.3 *When the diaphragm is required to transfer lateral 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 (46A-2).*

1646A.2.9.4 *Design forces for flexible diaphragms and their connections providing lateral supports for walls or frames of masonry or concrete shall be calculated using an R not to exceed 4.*

1646A.2.9.5 *Diaphragms supporting concrete or masonry walls shall have continuous ties or struts between diaphragm chords to distribute the anchorage forces specified in Section 1644A.13.1. Added chords may be used to form subdiaphragms to transmit the anchorage forces to the main cross ties.*

1646A.2.9.6 *Where wood diaphragms are used to laterally support concrete or masonry walls, the anchorage shall conform to Section 1644A.13.1. Anchorage shall not be accomplished by use of toenails or nails subject to withdrawal, nor shall wood ledgers or framing be used in cross-grain bending or cross-grain tension, and the continuous ties required by Section 1646A.2.9.5 shall be in addition to the diaphragm sheathing.*

> ***EXCEPTION:*** *The prohibited details may be used if an appropriate β factor is assigned to allow for nonductile behavior.*

1646A.2.10 Framing below the base. *Elements of the lateral-force-resisting system and all framing elements between the base and the foundation are subject to the same provisions as required for the superstructure. [For DSA/SS: Retrofit of any elements shall comply with the provisions for new buildings in accordance with Section 1633A.2.10.]*

1646A.2.11 [Not adopted by DSA/SS] Building separations. *When the gap separating the building from adjacent structures is less than 0.7R times the displacement due to seismic forces of the building Δ_s, then the effects of pounding shall be investigated and the structure modified so that pounding or interaction does not pose a life-safety threat to the building.*

> ***EXCEPTION:*** *Smaller separations may be permitted when justified by rational analyses based on maximum expected ground motions. Under this exception, as a minimum, building separations shall not be less than R/5.5 times the displacements due to specified seismic forces.*

1646A.2.11.1 [For DSA/SS] Building separations. *When the gap separating the building from adjacent structures or other portions of the building is less than Δ_{MT} as calculated per the provisions of Section 1633A.2.11, then the effects of pounding shall be investigated and the structure modified so that pounding or interaction does not pose a life-safety threat to the building.*

SECTION 1647A — NONBUILDING STRUCTURES

1647A.1 [Not adopted by DSA/SS] General. *Nonbuilding existing structures include all self-supporting structures other than buildings that carry gravity loads and resist the effects of earthquake. Nonbuilding existing structures shall be designed to resist the minimum lateral forces specified in this division. Design shall conform to the applicable provisions of Section 1634A for new structures except as modified by the provisions contained in this division.*

1647A.1.1 [For DSA/SS] General. *For all rehabilitations of existing nonconforming nonbuilding structures as defined in Article 1, Group 1, Chapter 4, Part 1, Title 24, the structural system shall be evaluated by the design professional in responsible charge of design and, if not meeting or exceeding the minimum seismic design purpose of this division, shall be retrofitted in compliance with these requirements.*

SECTION 1648A — METHOD B

1648A.1 [Not adopted by DSA/SS] *The existing or retrofitted structure shall be demonstrated to have the capability to sustain the deformation response due to the specified earthquake ground motions. The engineer shall provide an evaluation of the response of the existing structure in its current configuration and condition to the ground motions specified. If the building's seismic performance is evaluated as satisfactory and the peer reviewer(s) [For OSHPD 1: and the enforcement agent] concurs, then no further engineering work is required. When the evaluation indicates the building does not meet the objective of the safety goals of this division, [For OSHPD 1: and the applicable structural seismic performance (SPC) and nonstructural performance (NPC) requirements] then a retrofit and/or repair design shall be prepared that yields a structure that meets the life-safety [For OSHPD 1: and operational] performance objectives of Section 1640A of this division and reflects the appropriate consideration of existing conditions. Any approach to analysis and design may be used that yields a building of reliable stability in the prescribed design earthquake loads and conditions. The approach shall be rational, shall be consistent with the established principals of mechanics, and shall use the known performance characteristics of materials and assemblages under reversing loads typical of severe earthquake ground motions.*

> ***EXCEPTION:*** *Further consideration of the structure's seismic performance can be waived by the enforcement agent if both the engineer-of-record and peer reviewer(s) [OSHPD 1: and/or Enforcement Agent] conclude that the structural system can be expected to perform at least as well as required by the provisions of this division without completing an analysis of the structure's conformance to these requirements. A detailed report shall be submitted to the responsible enforcement agent that presents the reasons and basis for this conclusion. This report shall be prepared by the engineer of record. The peer reviewer(s) [OSHPD 1: and/or Enforcement Agent] shall concur in this conclusion and affirm to it in writing.*

1648A.1.1 [For DSA/SS] *The existing or retrofitted structure shall be demonstrated to have the capability to sustain the deformation response due to the specified earthquake ground motions and yield a building of reliable stability when subjected to the prescribed design earthquake loads and conditions. When the evaluation indicates the structural elements of the building do not meet the objective of the safety goals of this division, then a retrofit and/or repair design shall be prepared that yields a structure that*

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_w, 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 those required by Section 1643A.8.*

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.2.2.2.1 or the Site-Specific Procedure of Section 1648A.2.2.3.

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 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).*

Class C: *Very dense soil and soft rock with 1,200 ft/sec (366 m/s) < $\bar{v}_s$ ≤ 2,500 ft/sec (762 m/s) or with either standard blow count $\bar{N}$ > 50 or undrained shear strength $\bar{s}_u$ > 2,000 pounds per square feet (psf) (96 kN/m^2).*

Class D: *Stiff soil with 600 ft/sec (48 kN/m^2) < $\bar{v}_s$ ≤ 1,200 ft/sec (366 m/s) or with 15 < $\bar{N}$ ≤ 50 or 1,000 psf (48 kN/m^2) ≤ $\bar{s}_u$ < 2,000 psf (96 kN/m^2).*

Class E: *Any profile with more than 10 feet (3048 mm) of soft clay defined as soil with plasticity index PI > 20, or water content w > 40 percent, and $\bar{s}_u$ < 500 psf (24 kN/m^2) or a soil profile with $\bar{v}_s$ < 600 ft/sec (183 m/s). If insufficient data are available to classify a soil profile as Type A through D, a Type E profile shall be assumed.*

Class F: *Soils requiring site-specific evaluations:*

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

2. Peats and/or highly organic clays (H > 10 feet (3048 mm) of peat and/or highly organic clay, where H = thickness of soil).

3. Very high plasticity clays (H > 25 feet (7620 mm) with PI > 75 percent).

4. Very thick soft/medium stiff clays (H > 120 feet) [36 576 mm].

The parameters $\bar{v}_s$, $\bar{N}$ and $\bar{s}_u$ are, respectively, the average values of the shear wave velocity, Standard Penetration Test (SPT) blow count, and undrained shear strength of the upper 100 feet (30 480 mm) of soils at the site. These values shall be calculated from Equation (48B-3):

$$\bar{v}_s,\ \bar{N},\ \bar{s}_u = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}},\ \frac{d_i}{N_i},\ \frac{d_i}{s_{ui}}} \qquad (48B\text{-}3)$$

WHERE:

N_i = *SPT blow count in soil layer "i."*

n = *Number of layers of similar soil materials for which data is available.*

d_i = *Depth of layer "i."*

s_{ui} = *Undrained shear strength in layer "i."*

v_{si} = *Shear wave velocity of the soil in layer "i."*

and

$$\sum_{i=1}^{n} d_i = 100\ ft \qquad (48B\text{-}4)$$

Where reliable v_s data are available for the site, such data shall be used to classify the site. If such data are not available, N data shall be used for cohesionless soil sites (sands, gravels), and s_u data for cohesive soil sites (clays). For rock in profile classes B and C, classification may be based either on measured or estimated values of v_s. Classification of a site as Class A rock shall be based on measurements of v_s either for material at the site itself, or for similar rock materials in the vicinity; otherwise, Class B rock shall be assumed. Class A or B profiles shall not be assumed to be present if there is more than 10 feet (3048 mm) of soil between the rock surface and the base of the building.

A general, horizontal response spectrum shall be constructed by plotting the following two functions in the spectral acceleration vs. structural period domain, as shown in Figure 16A-R-1. Where a vertical response spectrum is required, it may be constructed by taking two-thirds of the spectral ordinates, at each period, obtained for the horizontal response spectrum.

$$S_a = (S_{xs} / B_s)\ (0.4 + 3\ T / T_o) \qquad (48A\text{-}5)$$

$$for\ 0 < T \le 0.2\ T_o$$

$$S_a = (S_{xi} / (B_1\ T)),\ for\ T > T_o \qquad (48A\text{-}6)$$

where T_o is given by the equation

$$T_o = (S_{xi}\ B_s) / (S_{xs}\ B_1) \qquad (48A\text{-}7)$$

where B_s and B_1 are taken from Table 16A-R-5.

A 5-percent damped response spectrum shall be used for the design of buildings and structural systems, with the following exceptions:

1. For structures without exterior cladding, an effective viscous damping ratio, β, of 2 percent shall be assumed.

2. For structures with wood diaphragms and a large number of interior partitions and cross walls that interconnect the diaphragm levels, an effective viscous damping ratio, β, of 10 percent may be assumed.

3. For structures rehabilitated using seismic isolation technology or enhanced energy dissipation technology, the equivalent effective viscous damping ratio, β, shall be determined in accordance with Section 1629A.10.2.

1648A.2.4.2.1 [For DSA/SS] 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.4.2.3.1.

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 FEMA 356, "Prestandard and Commentary for the Seismic Rehabilitation of Existing Buildings." 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).*

***Class C:** Very dense soil and soft rock with 1,200 ft/sec (366 m/s) < $\bar{v}_s$ ≤ 2,500 ft/sec (762 m/s) or with either standard blow count $\bar{N}$ > 50 or undrained shear strength $\bar{s}_u$ > 2,000 pounds per square feet (psf) (96 kN/m2).*

***Class D:** Stiff soil with 600 ft/sec (48 kN/m2) < $\bar{v}_s$ ≤ 1,200 ft/sec (366 m/s) or with 15 < $\bar{N}$ ≤ 50 or 1,000 psf (48 kN/m2) ≤ $\bar{s}_u$ < 2,000 psf (96 kN/m2).*

***Class E:** Any profile with more than 10 feet (3048 mm) of soft clay defined as soil with plasticity index PI > 20, or water content w > 40 percent, and $\bar{s}_u$ < 500 psf (24 kN/m2) or a soil profile with $\bar{v}_s$ < 600 ft/sec (183 m/s). If insufficient data are available to classify a soil profile as Type A through C, and there is no evidence of soft clay soils characteristic of Class E in the vicinity of the site, the default site class may be taken as Class D. If there is evidence of Class E soils in the vicinity of the site, and no other data supporting selection of Class A through D, the default site class shall be taken as Class E.*

***Class F:** Soils requiring site-specific evaluations:*

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

2. Peats and/or highly organic clays (H > 10 feet (3048 mm) of peat and/or highly organic clay, where H = thickness of soil).

3. Very high plasticity clays (H > 25 feet (7620 mm) with PI > 75 percent).

4. Very thick soft/medium stiff clays (H > 120 feet) [36 576 mm].

The parameters $\bar{v}_s$, $\bar{N}$ and $\bar{s}_u$ are, respectively, the average values of the shear wave velocity, Standard Penetration Test (SPT) blow count and undrained shear strength of the upper 100 feet (30 480 mm) of soils at the site. These values shall be calculated from Equation (48B-3):

$$\bar{v}_s, \bar{N}, \bar{s}_u = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}, \frac{d_i}{N_i}, \frac{d_i}{s_{ui}}} \qquad (48B\text{-}3)$$

WHERE:

N_i = *SPT blow count in soil layer "i."*

n = *Number of layers of similar soil materials for which data is available.*

d_i = *Depth of layer "i."*

s_{ui} = *Undrained shear strength in layer "i."*

v_{si} = *Shear wave velocity of the soil in layer "i."*

and

$$\sum_{i=1}^{n} d_i = 100 \text{ ft} \qquad (48B\text{-}4)$$

Where reliable v_s data are available for the site, such data shall be used to classify the site. If such data are not available, N data shall be used for cohesionless soil sites (sands, gravels), and s_u data for cohesive soil sites (clays). For rock in profile classes B and C, classification may be based either on measured or estimated values of v_s. Classification of a site as Class A rock shall be based on measurements of v_s either for material at the site itself or for similar rock materials in the vicinity; otherwise, Class B rock shall be assumed. Class A or B profiles shall not be assumed to be present if there is more than 10 feet (3048 mm) of soil between the rock surface and the base of the building.

A general, horizontal response spectrum shall be constructed by plotting the following two functions in the spectral acceleration vs. structural period domain, as shown in Figure 1-1 of FEMA 356, "Prestandard and Commentary for the Seismic Rehabilitation of Existing Buildings." Where a vertical response spectrum is required, it may be constructed by taking two-thirds of the spectral ordinates, at each period, obtained for the horizontal response spectrum.

$$S_a = S_{xs}\left[\left(\frac{5}{B_s} - 2\right)\frac{T}{T_s} + 0.4\right] \qquad (48A\text{-}5)$$

for $0 < T < T_o$, and

$$S_a = \frac{S_{xs}}{B_s} \text{ for } T < T_s, \text{ and} \qquad (48A\text{-}6)$$

$$S_a = \frac{S_{x1}}{B_1 T} \text{ for } T > T_s \qquad (48A\text{-}7)$$

where T_s and T_o are given by Equations 48A-8 and 48A-9 is given by the equation

$$T_s = \frac{S_{x1} B_s}{S_{xs} B_1} \qquad (48A\text{-}8)$$

$$T_o = 0.2\, T_s \qquad (48A\text{-}9)$$

where B_s and B_1 are taken from Table 16A-R-5.

Use of spectral response accelerations calculated using Equation 48A-5 in the extreme short period range ($T < T_o$) shall only be permitted in dynamic analysis procedures and only for modes other than the fundamental mode.

A 5-percent damped response spectrum shall be used for the design of buildings and structural systems, with the following exceptions:

1. For structures without exterior cladding, an effective viscous damping ratio, β, equal to 2 percent of critical damping shall be assumed.

2. For structures with wood diaphragms and a large number of interior partitions and cross walls that interconnect the diaphragm levels at a maximum spacing of 40 feet on center transverse to the direction of motion, an effective viscous damping ratio, β, of 10 percent of critical damping shall be permitted.

3. For structures rehabilitated using seismic isolation technology or enhanced energy dissipation technology, the equivalent effective viscous damping ratio, β, shall be determined in accordance with Section 1629A.10.2.

***1648A.2.4.3 [For OSHPD 1] Site-specific procedure to determine the acceleration response spectra.** Where site-specific ground shaking characterization is used as the basis of the design, the characterization shall be developed in accordance with this section.*

Development of site-specific response spectra shall be based on the geologic, seismologic and soil characteristics associated with the specific site. Response spectra shall be developed for an equivalent viscous damping ratio of 5 percent. Additional spectra may be developed for other damping ratios appropriate to the indicated structural behavior, as discussed in Section 1648A.2.4.2.

When the 5-percent damped site-specific spectrum has spectral amplitudes in the period range of the greatest significance to the structural response that are less than 70 percent of the spectral amplitudes of the General Response Spectrum, an independent peer review of the spectrum shall be made by an individual with expertise in the evaluation of ground motion in accordance with Section 1649A.

The maximum considered earthquake ground motion shall be taken as that motion represented by an acceleration response spectrum having a 2 percent probability of exceedance within a 50-year period. The maximum considered earthquake spectral response acceleration at any period shall be taken from the 2 percent probability of exceedance within a 50 year period spectrum as limited by the following:

Where the spectral response ordinates at 0.2 second or 1 second for a 5 percent damped spectrum having a 2 percent probability of exceedance within a 50-year period exceed the corresponding ordinates of the deterministic limit, the maximum considered earthquake ground motion spectrum shall be taken as the lesser of the probabilistic maximum considered earthquake ground motion or the deterministic maximum considered earthquake ground motion spectrum. The deterministic limit for the maximum considered earthquake ground motion response spectrum shall be calculated as 150 percent of the median spectral response accelerations at all periods resulting from a characteristic earthquake on any known active fault within the region and shall comply with the response spectrum determined in accordance with Figure 16A-R-2, where F_a and F_v are determined in accordance with Section 1648A.2.4.2, with the value of the mapped short period spectral response acceleration, S_s taken as 1.5g, and the value of the mapped spectral response acceleration at 1 second S_1, taken as 0.6g.

When a site-specific response spectrum has been developed and other sections of these regulations require values for the spectral response parameters, S_{xs}, S_{x1}, or T_o, they shall be obtained in accordance with this section. The value of the design spectral response acceleration at short S_{xs}, shall be taken as the response acceleration obtained from the site-specific spectrum at a period of 0.2 second, except that it shall be taken as not less than 90 percent of the peak response acceleration at any period. In order to obtain a value for the design spectral response acceleration parameter S_{x1}, a curve of the form $S_a = S_{x1}/T$ shall be graphically overlaid on the site-specific spectrum such that at any period, the value of S_a obtained from the curve is not less than 90 percent of that which would be obtained directly from the spectrum. The value of T_o shall be determined in accordance with Equation (48A-8). Alternatively, the values obtained in accordance with Section 1648A.2.4.2 may be used for all of these parameters.

$$T_o = S_{x1} / S_{xs} \qquad (48A\text{-}8)$$

Time-history analysis shall be performed with no fewer than three data sets (two horizontal components or, if vertical motion is to be considered, two horizontal components and one vertical component) of appropriate ground motion time histories that shall be selected and scaled from no fewer than three recorded events. Appropriate time histories shall have magnitude, fault distances and source mechanisms that are consistent with those that control the design earthquake ground motion. Where three appropriate recorded ground-motion time-history data sets are not available, appropriate simulated time-history data sets may be used to make up the total number required. For each data set, 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 data sets shall be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5-percent damped spectrum for the design earthquake for periods between 0.2T second and 1.5T seconds (where T is the fundamental period of the building).

Where three time-history data sets are used in the analysis of the structure, the maximum value of each response parameter (e.g., force in a member, displacement at a specific level) shall be used to determine design acceptability. Where seven or more time-history data sets are employed, the average value of each response parameter shall be used to determine design acceptability.

***1648A.2.4.3.1 [For DSA/SS] Site-specific procedure to determine the acceleration response spectra.** Where site-specific ground shaking characterization is selected, or required by DSA, as the basis of the rehabilitation design, the characterization shall be developed in accordance with this section.*

Development of site-specific response spectra shall be based on the geologic, seismologic and soil characteristics associated with the specific site. Response spectra shall be developed for an equivalent viscous damping ratio of 5 percent of critical damping. Additional spectra may be developed for other damping ratios appropriate to the indicated structural behavior, as discussed in Section 1648A.2.2.2.1. The 5-percent damped site-specific spectral amplitudes in the period range of the greatest significance to the structural response shall not be less than 70 percent of the spectral amplitudes of the General Response Spectrum.

The maximum considered earthquake ground motion shall be taken as that motion represented by a 5% damped acceleration response spectrum having a 2 percent probability of exceedance within a 50-year period. The maximum considered earthquake spectral response acceleration, S_{aM}, at any period, T, shall be taken from that spectrum as limited by the following:

Where the spectral response ordinates at 0.2 second or 1 second for a 5% damped spectrum having a 2% probability of exceedance within a 50-year period exceeds the corresponding ordinates of the deterministic limit on the maximum considered earthquake ground motion, the maximum considered earthquake ground motion spectrum shall be taken as the lesser of the probabilistic maximum considered earthquake ground motion or the deterministic maximum considered earthquake ground motion spectrum but not less than the deterministic limit on the maximum considered earthquake ground motion.

The deterministic limit on the maximum considered earthquake ground motion shall be taken as the response spectrum determined in accordance with Figure 16A-R-2, where F_a and F_v are determined in accordance with Section 1648A2.2.2.1 with the value of the mapped short period spectral response acceleration, S_S, taken as 1.5g and the value of the mapped spectral response acceleration at 1 second, S_1, taken as 0.6g.

The deterministic maximum considered earthquake ground motion response spectrum shall be calculated as 150% of the median spectral response accelerations, S_{aM}, at all periods resulting from a characteristic earthquake on any known active fault within the region.

The site-specific response acceleration parameters for ground motion having a 10% probability of exceedance in 50 years shall be taken as the lesser of:

1. The values of the parameters, S_s and S_1, from mean probabilistic site-specific spectra at the 10%/50 year probability of exceedance, and

2. Two-thirds of the values of the parameters, S_s and S_1, determined for the maximum considered earthquake based on site-specific spectra.

When a site-specific response spectrum has been developed and other sections of these regulations require values for the spectral response parameters, S_{xs}, S_{x1}, or T_s, they shall be obtained in ac-

cordance with this section. The value of the design spectral response acceleration at short-periods, S_{xs}, shall be taken as the response acceleration obtained from the site-specific spectrum at a period of 0.2 second, except that it shall be taken as not less than 90 percent of the peak response acceleration at any period. In order to obtain a value for the design spectral response acceleration parameter S_{x1}, a curve of the form $S_a = S_{x1}/T$ shall be graphically overlaid on the site-specific spectra such that at any period, the value of S_a obtained from the curve is not less than 90 percent of that which would be obtained directly from the spectra. The value of T_s shall be determined in accordance with Equation (48A-10). Alternatively, the values obtained in accordance with Section 1648A.4.2.2.1 may be used for all of these parameters.

$$T_s = S_{x1}/S_{xs} \qquad (48A\text{-}10)$$

Time-history analysis shall be performed with no fewer than three data sets (two horizontal components or, if vertical motion is to be considered, two horizontal components and one vertical component) of appropriate ground motion time histories that shall be selected and scaled from no fewer than three recorded events. Appropriate time histories shall have magnitude, fault distances and source mechanisms that are consistent with those that control the design earthquake ground motion. Where three appropriate recorded ground-motion time-history data sets are not available, appropriate simulated time-history data sets may be used to make up the total number required. For each data set, 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 data sets shall be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5-percent damped spectrum for the design earthquake for periods between 0.2T second and 1.5T seconds (where T is the fundamental period of the building).

Where three time-history data sets are used in the analysis of the structure, the maximum value of each response parameter (e.g., force in a member, displacement at a specific level) shall be used to determine design acceptability. Where seven or more time-history data sets are employed, the average value of each response parameter shall be used to determine design acceptability.

1648A.2.5 *Whatever evaluation or analysis method is used in meeting the requirements of Section 1648A, the designer shall, unless the exception of Section 1648A.1 applies, at a minimum.*

[For DSA/SS] *Whatever evaluation or analysis method is used in meeting the requirements of Section 1648A, the designer shall, at a minimum:*

1648A.2.5.1 *Identify all elements and systems (both vertical and horizontal) that are subject to the response loads and deformations due to the specified maximum expected earthquake ground shaking. Elements include beams, columns, joints, connections, walls, diaphragms, construction joints, precast element joints, exterior panel connections, bracing, diaphragms, collectors, diaphragm-to-wall or frame connection and foundations.*

1648A.2.5.2 *Identify the vertical elements (e.g., walls, frames, braced frames, in-filled frames, moment frames, etc.) that will participate in the lateral-load-resisting system.*

1648A.2.5.3 *Identify the horizontal or nearly horizontal elements that form the diaphragm systems that interconnect the vertical elements, along with the chords, drags or collector elements, and connections to the vertical systems, and the internal connections within the diaphragm (precast planks, metal decking, bracing systems, pour-strips for prestressed slabs, etc.).*

1648A.2.5.4 *Identify the foundation system supporting the lateral-load-resisting system, including all connections and the means of resisting the actions of overturning moment and sliding.*

1648A.2.5.5 *Assign the expected strength level to all elements for all of their possible modes of yielding or failure. For reinforced concrete, use nominal capacity. For structural steel, use either 1.7 times allowable stress capacity or the nominal capacity from LRFD. For all other materials, use either 1.7 times allowable stress capacity or estimated strength from tests and/or existing research results.*

1648A.2.5.6 *Assign the effective elastic stiffness for all elements for each type and directional sense of action (flexural, shear, torsion, axial) that the element shall resist. The effective stiffness should be the best estimate of the secant stiffness at the development of the element strength representing the onset of the constant yield threshold.*

1648A.2.5.7 *Assign the element deformation behavior beyond the development of the strength or constant yield threshold for each mode of failure or yielding. Identify elements having a sudden brittle or buckling mode of failure. The effects of reversed cycles of loading should be considered to evaluate the degree of strength degradation and/or the pinching of the shape of the hysterisis loop. The deformation behavior may be in the form of load-deformation curves, allowable inelastic demand ratio (IDR_L) values, or allowable ductility demand (μ_L) values, or maximum allowable strain values ε_L or allowable rotation values θ_L. The classification of the elements as "ductile," "limited-ductile," or "nonductile" may be a part of the element deformation behavior description.*

1648A.2.6 *Prior to implementation, the procedures, methods, material assumptions and acceptance/rejection criteria proposed by the engineer will be peer reviewed as provided in Section 1649A.*

1648A.2.7 *The conclusions and design decisions shall be reviewed and accepted by the peer reviewer(s).*

1648A.3 [Not adopted by DSA/SS] *Any method of analysis meeting the requirements of Sections 1648A.2 and 1648A.3 may be used, subject to acceptance by the peer reviewer(s). Among those that can be used are the following types of analysis and assessment provisions, if the specific characteristics of the structure warrant their use:*

1. *Equivalent stiffness (or substitute structure) methods.*
2. *Inelastic demand ratio methods.*
3. *Pushover or capacity spectrum methods.*
4. *Inelastic time-history methods.*

SECTION 1649A — PEER REVIEW REQUIREMENTS

1649A.1 General. *Independent peer review is an objective technical review by knowledgeable reviewer(s) experienced in the structural design, analysis and performance issues involved. The reviewer(s) shall examine the available information on the condition of the building, the basic engineering concepts employed and the recommendations for action. [For DSA/SS: DSA may require more than one peer reviewer, creating a peer review team, to provide expertise relating to specific aspects of construction and/or the evaluation and retrofit design. When a peer review team is required, the owner shall designate a chairperson to manage the peer review process and prepare any reports required in this section.]*

1649A.2 [Not adopted by DSA/SS] Timing of Independent Review. *The independent reviewer(s) shall be selected prior to initi-*

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 responsible 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.*

Chapter 17
STRUCTURAL TESTS AND INSPECTIONS

SECTION 1701 — SPECIAL INSPECTIONS

1701.1 General. In addition to the inspections required by Section 108, the owner or the engineer or architect of record acting as the owner's agent shall employ one or more special inspectors who shall provide inspections during construction on the types of work listed under Section 1701.5.

EXCEPTION: The building official may waive the requirement for the employment of a special inspector if the construction is of a minor nature.

1701.2 Special Inspector. The special inspector shall be a qualified person who shall demonstrate competence, to the satisfaction of the building official, for inspection of the particular type of construction or operation requiring special inspection.

1701.3 Duties and Responsibilities of the Special Inspector. The special inspector shall observe the work assigned for conformance to the approved design drawings and specifications.

The special inspector shall furnish inspection reports to the building official, the engineer or architect of record, and other designated persons. All discrepancies shall be brought to the immediate attention of the contractor for correction, then, if uncorrected, to the proper design authority and to the building official.

The special inspector shall submit a final signed report stating whether the work requiring special inspection was, to the best of the inspector's knowledge, in conformance to the approved plans and specifications and the applicable workmanship provisions of this code.

1701.4 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.)

1. **Concrete.**

ASTM C 94, Ready-mixed Concrete

2. **Connections.**

Specification for Structural Joints Using ASTM A 325 or A 490 Bolts-Load and Resistance Factor Design, Research Council of Structural Connections, Section 1701.5, Item 6.

Specification for Structural Joints Using ASTM A 325 or A 490 Bolts-Allowable Stress Design, Research Council of Structural Connections, Section 1701.5, Item 6.

3. **Spray-applied fire-resistive materials.**

UBC Standard 7-6, Thickness and Density Determination for Spray-applied Fire-resistive Materials

1701.5 Types of Work. Except as provided in Section 1701.1, the types of work listed below shall be inspected by a special inspector.

1. **Concrete.** During the taking of test specimens and placing of reinforced concrete. See Item 12 for shotcrete.

EXCEPTIONS: 1. Concrete for foundations conforming to minimum requirements of Table 18-I-C or for Group R, Division 3 or Group U, Division 1 Occupancies, provided the building official finds that a special hazard does not exist.

2. For foundation concrete, other than cast-in-place drilled piles or caissons, where the structural design is based on an f'_c no greater than 2,500 pounds per square inch (psi) (17.2 MPa).

3. Nonstructural slabs on grade, including prestressed slabs on grade when effective prestress in concrete is less than 150 psi (1.03 MPa).

4. Site work concrete fully supported on earth and concrete where no special hazard exists.

1.1 [For OSHPD 2] ***Placing record.*** *A record shall be kept on the site of the time and date of placing the concrete in each portion of the structure. Such record shall be kept until the completion of the structure and shall be open to the inspection of the enforcement agency.* C A C A C A C

2. **Bolts installed in concrete.** Prior to and during the placement of concrete around bolts when stress increases permitted by Footnote 5 of Table 19-D or Section 1923 are utilized.

3. **Special moment-resisting concrete frame.** For moment frames resisting design seismic load in structures within Seismic Zones 3 and 4, the special inspector shall provide reports to the person responsible for the structural design and shall provide continuous inspection of the placement of the reinforcement and concrete.

4. **Reinforcing steel and prestressing steel tendons.**

4.1 During all stressing and grouting of tendons in prestressed concrete.

4.2 During placing of reinforcing steel and prestressing tendons for all concrete required to have special inspection by Item 1.

EXCEPTION: The special inspector need not be present continuously during placing of reinforcing steel and prestressing tendons, provided the special inspector has inspected for conformance to the approved plans prior to the closing of forms or the delivery of concrete to the jobsite.

5. **Structural welding.**

5.1 **General.** During the welding of any member or connection that is designed to resist loads and forces required by this code.

EXCEPTIONS: 1. Welding done in an approved fabricator's shop in accordance with Section 1701.7.

2. The special inspector need not be continuously present during welding of the following items, provided the materials, qualifications of welding procedures and welders are verified prior to the start of work; periodic inspections are made of work in progress; and a visual inspection of all welds is made prior to completion or prior to shipment of shop welding:

2.1 Single-pass fillet welds not exceeding $^5/_{16}$ inch (7.9 mm) in size.

2.2 Floor and roof deck welding.

2.3 Welded studs when used for structural diaphragm or composite systems.

2.4 Welded sheet steel for cold-formed steel framing members such as studs and joists.

2.5 Welding of stairs and railing systems.

5.2 **Special moment-resisting steel frames.** During the welding of special moment-resisting steel frames. In addition to Item 5.1 requirements, nondestructive testing as required by Section 1703 of this code.

A. ***[For BSC] Lateral force resisting steel frames.*** *During the welding of lateral force resisting steel frames. In addition to Item 5.1 requirements, nondestructive testing as required by Section 1703 of this code.* C A C A C A C

5.3 **Welding of reinforcing steel.** During the welding of reinforcing steel.

EXCEPTION: The special inspector need not be continuously present during the welding of ASTM A 706 reinforcing steel not larger than No. 5 bars used for embedments, provided the materials, qualifications of welding procedures and welders are verified prior to the start of work; periodic inspections are made of work in progress; and a visual inspection of all welds is made prior to completion or prior to shipment of shop welding.

6. **High-strength bolting.** The inspection of high-strength A 325 and A 490 bolts shall be in accordance with approved nationally recognized standards and the requirements of this section.

While the work is in progress, the special inspector shall determine that the requirements for bolts, nuts, washers and paint; bolted parts; and installation and tightening in such standards are met. Such inspections may be performed on a periodic basis in accordance with the requirements of Section 1701.6. The special inspector shall observe the calibration procedures when such procedures are required by the plans or specifications and shall monitor the installation of bolts to determine that all plies of connected materials have been drawn together and that the selected procedure is properly used to tighten all bolts.

7. **Structural masonry.**

7.1 For masonry, other than fully grouted open-end hollow-unit masonry, during preparation and taking of any required prisms or test specimens, placing of all masonry units, placement of reinforcement, inspection of grout space, immediately prior to closing of clean-outs, and during all grouting operations.

EXCEPTION: For hollow-unit masonry where the f'_m is no more than 1,500 psi (10.34 MPa) for concrete units or 2,600 psi (17.93 MPa) for clay units, special inspection may be performed as required for fully grouted open-end hollow-unit masonry specified in Item 7.2.

7.2 For fully grouted open-end hollow-unit masonry during preparation and taking of any required prisms or test specimens, at the start of laying units, after the placement of reinforcing steel, grout space prior to each grouting operation, and during all grouting operations.

EXCEPTION: Special inspection as required in Items 7.1 and 7.2 need not be provided when design stresses have been adjusted as specified in Chapter 21 to permit noncontinuous inspection.

8. **Reinforced gypsum concrete.** When cast-in-place Class B gypsum concrete is being mixed and placed.

9. **Insulating concrete fill.** During the application of insulating concrete fill when used as part of a structural system.

EXCEPTION: The special inspections may be limited to an initial inspection to check the deck surface and placement of reinforcing. The special inspector shall supervise the preparation of compression test specimens during this initial inspection.

10. **Spray-applied fire-resistive materials.** As required by UBC Standard 7-6.

11. **Piling, drilled piers and caissons.** During driving and testing of piles and construction of cast-in-place drilled piles or caissons. See Items 1 and 4 for concrete and reinforcing steel inspection.

12. **Shotcrete.** During the taking of test specimens and placing of all shotcrete and as required by Sections 1924.10 and 1924.11.

EXCEPTION: Shotcrete work fully supported on earth, minor repairs and when, in the opinion of the building official, no special hazard exists.

13. **Special grading, excavation and filling.** During earth-work excavations, grading and filling operations inspection to satisfy requirements of Chapter 18 and Appendix Chapter 33.

14. **Smoke-control system.**

14.1 During erection of ductwork and prior to concealment for the purposes of leakage testing and recording of device location.

14.2 Prior to occupancy and after sufficient completion for the purposes of pressure difference testing, flow measurements, and detection and control verification.

15. **Special cases.** Work that, in the opinion of the building official, involves unusual hazards or conditions.

*16. **[For OSHPD 2] Manufactured trusses.** The fabrication of trusses and other assemblages constructed using wood and metal members, or using light metal plate connectors, shall be continuously inspected by a qualified inspector approved by the enforcement agency. The inspector shall furnish the architect, structural engineer and the enforcement agency with a report that the lumber species, grades and moisture content; type of glue, temperature and gluing procedure; type of metal members and metal plate connectors; and the workmanship conform in every material respect with the duly approved plans and specifications. Each inspected truss shall be stamped by the inspector with an identifying mark.*

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1701.6 Continuous and Periodic Special Inspection.

1701.6.1 Continuous special inspection. Continuous special inspection means that the special inspector is on the site at all times observing the work requiring special inspection.

1701.6.2 Periodic special inspection. Some inspections may be made on a periodic basis and satisfy the requirements of continuous inspection, provided this periodic scheduled inspection is performed as outlined in the project plans and specifications and approved by the building official.

1701.7 Approved Fabricators. Special inspections required by this section and elsewhere in this code are not required where the work is done on the premises of a fabricator registered and approved by the building official to perform such work without special inspection. The certificate of registration shall be subject to revocation by the building official if it is found that any work done pursuant to the approval is in violation of this code. The approved fabricator shall submit a certificate of compliance that the work was performed in accordance with the approved plans and specifications to the building official and to the engineer or architect of record. The approved fabricator's qualifications shall be contingent on compliance with the following:

1. The fabricator has developed and submitted a detailed fabrication procedural manual reflecting key quality control procedures that will provide a basis for inspection control of workmanship and the fabricator plant.

2. Verification of the fabricator's quality control capabilities, plant and personnel as outlined in the fabrication procedural manual shall be by an approved inspection or quality control agency.

3. Periodic plant inspections shall be conducted by an approved inspection or quality control agency to monitor the effectiveness of the quality control program.

4. It shall be the responsibility of the inspection or quality control agency to notify the approving authority in writing of any change to the procedural manual. Any fabricator approval may be revoked for just cause. Reapproval of the fabricator shall be contingent on compliance with quality control procedures during the past year.

SECTION 1702 — STRUCTURAL OBSERVATION

Structural observation shall be provided in Seismic Zone 3 or 4 when one of the following conditions exists:

1. The structure is defined in Table 16-K as Occupancy Category 1, 2 or 3,

2. The structure is required to comply with Section 403,

3. The structure is in Seismic Zone 4, N_a as set forth in Table 16-S is greater than one, and a lateral design is required for the entire structure,

EXCEPTION: One- and two-story Group R, Division 3 and Group U Occupancies and one- and two-story Groups B, F, M and S Occupancies.

4. When so designated by the architect or engineer of record, or

5. When such observation is specifically required by the building official.

The owner shall employ the engineer or architect responsible for the structural design, or another engineer or architect designated by the engineer or architect responsible for the structural design, to perform structural observation as defined in Section 220. Observed deficiencies shall be reported in writing to the owner's representative, special inspector, contractor and the building official. The structural observer shall submit to the building official a written statement that the site visits have been made and identifying any reported deficiencies that, to the best of the structural observer's knowledge, have not been resolved.

1702.1 [For BSC] Structural Observation. *Structural observation shall be provided in Seismic Zone 3 or 4 when one of the following conditions exists:*

1. The structure is defined in Table 16-K as Occupancy Category I, II or III,

2. The structure is required to comply with Section 403

3. The structure is in Seismic Zone 4 and a lateral design is required for the entire structure.

EXCEPTION: *One- and two-story wood framed Group R, Division 3, B, F, M and S Occupancies, provided the adjacent grade is not steeper than 1 unit vertical in 10 units horizontal (10% sloped).*

4. When so designated by the architect or engineer of record, or

5. When such observation is specifically required by the building official.

The owner shall employ the engineer or architect responsible for the structural design, or another engineer or architect designated by the engineer or architect responsible for the structural design to perform structural observation as defined in Section 220.

The owner or owner's representative shall coordinate and call a preconstruction meeting between the engineer or architect responsible for the structural design, structural observer, contractor, affected subcontractors and deputy inspectors. The structural observer shall preside over the meeting. The purpose of the meeting shall be to identify the major structural elements and connections that affect the vertical and lateral load systems of the structure and to review scheduling of the required observations. A record of the meeting shall be included in the first report submitted to the building official.

Observed deficiencies shall be reported in writing to the owner's representative, special inspector, contractor and the building official. Upon the form prescribed by the building official, the structural observer shall submit to the building official a written statement at each significant construction stage stating that the site visits have been made and identifying any reported deficiencies which, to the best of the structural observer's knowledge, have not been resolved. A final report by the structural observer which states that all observed deficiencies have been resolved is required before acceptance of the work by the building official.

SECTION 1703 — NONDESTRUCTIVE TESTING

In Seismic Zones 3 and 4, welded, fully restrained connections between the primary members of ordinary moment frames and special moment-resisting frames shall be tested by nondestructive methods for compliance with approved standards and job specifications. This testing shall be a part of the special inspection requirements of Section 1701.5. A program for this testing shall be established by the person responsible for structural design and as shown on plans and specifications.

As a minimum, this program shall include the following:

1. All complete penetration groove welds contained in joints and splices shall be tested 100 percent either by ultrasonic testing or by radiography.

EXCEPTIONS: 1. When approved, the nondestructive testing rate for an individual welder or welding operator may be reduced to 25 percent, provided the reject rate is demonstrated to be 5 percent or less of the welds tested for the welder or welding operator. A sampling of at least 40 completed welds for a job shall be made for such reduction evaluation. Reject rate is defined as the number of welds containing rejectable defects divided by the number of welds completed. For evaluating the reject rate of continuous welds over 3 feet (914 mm) in length where the effective throat thickness is 1 inch (25 mm) or less, each 12-inch increment (305 mm) or fraction thereof shall be considered as one weld. For evaluating the reject rate on continuous welds over 3 feet (914 mm) in length where the effective throat thickness is greater than 1 inch (25 mm), each 6 inches (152 mm) of length or fraction thereof shall be considered one weld.

2. For complete penetration groove welds on materials less than $^5/_{16}$ inch (7.9 mm) thick, nondestructive testing is not required; for this welding, continuous inspection is required.

3. When approved by the building official and outlined in the project plans and specifications, this nondestructive ultrasonic testing may be performed in the shop of an approved fabricator utilizing qualified test techniques in the employment of the fabricator.

2. Partial penetration groove welds when used in column splices shall be tested either by ultrasonic testing or radiography when required by the plans and specifications. For partial penetration groove welds when used in column splices, with an effective throat less than $^3/_4$ inch (19.1 mm) thick, nondestructive testing is not required; for this welding, continuous special inspection is required.

3. Base metal thicker than $1^1/_2$ inches (38 mm), when subjected to through-thickness weld shrinkage strains, shall be ultrasonically inspected for discontinuities directly behind such welds after joint completion.

Any material discontinuities shall be accepted or rejected on the basis of the defect rating in accordance with the (larger reflector) criteria of approved national standards.

1703.1 [For BSC] Nondestructive Testing. *In Seismic Zones 3 and 4, welded connections between the primary members of lateral force resisting frames, which are subject to net tensile forces shall be tested by nondestructive methods in accordance with AISC-Seismic Part I Section 16 for compliance with approved standards and job specifications. This testing shall be a part of the special inspection requirements of Section 1701.5. A program for this testing shall be established by the person responsible for structural design and as shown on plans and specifications.*

As a minimum, this program shall include the following:

1. All complete penetration groove welds contained in joints and splices shall be tested 100 percent either by ultrasonic testing or by radiography.

EXCEPTIONS: *1. When approved, nondestructive testing rate for an individual welder or welding operator may be reduced to 25 percent, provided the reject rate is demonstrated to be 5 percent or less of the welds tested for the welder or welding operator. A sampling of at least 40 completed welds for a job shall be made for such reduction evaluation. Reject rate is defined as the number of welds containing rejectable defects divided by the number of welds completed. For evaluating reject rate of continuous welds over 3 feet (914 mm) in length where the effective throat thickness is 1 inch (25 mm) or less, each 12-inch increment (305 mm) or fraction thereof shall be considered one weld. For evaluating the reject rate on continuous welds over 3 feet (914 mm) in length where the effective throat thickness is greater than 1 inch (25 mm), each 6 inches (152 mm) length of fraction thereof shall be considered one weld.*

2. For complete penetration groove welds on materials less than $^5/_{16}$ *inch (7.9 mm) thick, nondestructive testing is not required; for this welding, continuous inspection is required.*

3. When approved by the building official and outlined in the project plans and specification, this nondestructive ultrasonic testing may be performed in the shop of an approved fabricator utilizing qualified test techniques in the employment of the fabricator.

2. Partial penetration groove welds when used in column splices shall be tested either by ultrasonic testing or radiography when required by the plans and specifications. For partial penetration groove welds when used in column splices, with an effective throat less than $^3/_4$ *inch (19.1 mm) thick, nondestructive testing is not required; for this welding, continuous special inspection is required.*

3. Base metal thicker than $1^1/_2$ *inches (38 mm), when subjected to through-thickness weld shrinkage strains, shall be ultrasonically inspected for discontinuities directly behind such welds after joint completion.*

Any material discontinuities shall be accepted or rejected on the basis of the defect rating in accordance with the (larger reflector) criteria of approved national standards.

SECTION 1704 — PREFABRICATED CONSTRUCTION

1704.1 General.

1704.1.1 Purpose. The purpose of this section is to regulate materials and establish methods of safe construction where any structure or portion thereof is wholly or partially prefabricated.

1704.1.2 Scope. Unless otherwise specifically stated in this section, all prefabricated construction and all materials used therein shall conform to all the requirements of this code. (See Section 104.2.8.)

1704.1.2.1 [For HCD 1] Factory-built housing. *The provisions of Health and Safety Code Division 13, Part 6 commencing with Section 19960, and the California Code of Regulations, Title 25, Division 1, Chapter 3 commencing with Section 3000 shall apply to the construction and inspection of Factory-Built Housing as defined in Health and Safety Code Section 19971.*

1704.1.3 Definition.

PREFABRICATED ASSEMBLY is a structural unit, the integral parts of which have been built up or assembled prior to incorporation in the building.

1704.2 Tests of Materials. Every approval of a material not specifically mentioned in this code shall incorporate as a proviso the kind and number of tests to be made during prefabrication.

1704.3 Tests of Assemblies. The building official may require special tests to be made on assemblies to determine their durability and weather resistance.

1704.4 Connections. See Section 1611.11.1 for design requirements of connections for prefabricated assemblies.

1704.5 Pipes and Conduits. See Section 1611.11.2 for design requirements for removal of material for pipes, conduit and other equipment.

1704.6 Certificate and Inspection.

1704.6.1 Materials. Materials and the assembly thereof shall be inspected to determine compliance with this code. Every material shall be graded, marked or labeled where required elsewhere in this code.

1704.6.2 Certificate. A certificate of approval shall be furnished with every prefabricated assembly, except where the assembly is readily accessible to inspection at the site. The certificate of approval shall certify that the assembly in question has been inspected and meets all the requirements of this code. When mechanical equipment is installed so that it cannot be inspected at the site, the certificate of approval shall certify that such equipment complies with the laws applying thereto.

1704.6.3 Certifying agency. To be acceptable under this code, every certificate of approval shall be made by an approved agency.

1704.6.4 Field erection. Placement of prefabricated assemblies at the building site shall be inspected by the building official to determine compliance with this code.

1704.6.5 Continuous inspection. If continuous inspection is required for certain materials where construction takes place on the site, it shall also be required where the same materials are used in prefabricated construction.

EXCEPTION: Continuous inspection will not be required during prefabrication if the approved agency certifies to the construction and furnishes evidence of compliance.

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 com-

bined 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.

$$\phi P_{nw} = 0.55 \phi f'_c A_g \left[1 - \left(\frac{k l_c}{32h} \right)^2 \right] \qquad (14\text{-}1)$$

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 Δ moments, shall be as set forth in Formula (14-2).*

$$M_u \leq \phi M_n \qquad (14\text{-}2)$$

Unless a more comprehensive analysis is used, the P Δ 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:

$$\Delta_s = \Delta_{cr} + \left(\frac{M_s - M_{cr}}{M_n - M_{cr}} \right) (\Delta_n - \Delta_{cr}); \text{ for } M_s > M_{cr} \qquad (14\text{-}4)$$

$$\Delta_s = \frac{5 M_s l_c^{\,2}}{48 E_c I_g}; \text{ for } M_s < M_{cr} \qquad (14\text{-}5)$$

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.*

$\Delta_{cr} = \dfrac{5 M_{cr} l_c^{\,2}}{48 E_c I_g}$

$\Delta_n = \dfrac{5 M_n l_c^{\,2}}{48 E_c I_{cr}}$

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 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.*

1915.2.2.1 [For BSC] *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* CACAC

C A C *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).

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. 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:

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.

1928.1.2.3.1 [For BSC] Basic combinations. 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:

1. 1.4D

2. 1.2D + 1.6L + 0.5(L_r or S or R)

3. 1.2D + 1.6(L_r or S or R) + (0.5L or 0.8W)

4. 1.2D + 1.3W + 0.5L + 0.5(L_r or S or R)

5. 1.2D ± 1.0E + (0.5L or 0.2S)

6. 0.9D ± (1.3W or 1.0E)

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.

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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 $1.2T$.

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. 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).

2204.1.1 [For BSC] Load and Resistance Factor Design. *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.* CACACAC

2204.2 Allowable Stress Design. 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).

2204.2.1 [For BSC] Allowable Stress Design. *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.* CACACAC

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

Chapter 22B *[BSC]*
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 III, 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 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

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 only an indication of an acceptable method or material that can be used with the approval of the Building Official.

SECTION 2214B — DESIGN METHODS

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 ass 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.

ter, nails shall be spaced at 6 inches (152 mm) on center along intermediate framing members. For all other conditions, nails of the same size shall be spaced at 12 inches (305 mm) on center along intermediate framing members.

CACAAC *2315.5.6 [For BSC] Hold-down connectors. Hold-down connector bolts into wood framing require steel plate washers in accordance with Table 23-II-L. Hold-downs shall be re-tightened just prior to covering the wall framing.*

2315.6 Fiberboard Sheathing Diaphragms. Wood stud walls sheathed with fiberboard sheathing may be used to resist horizontal forces not exceeding those set forth in Division III, Part IV. The fiberboard sheathing, 4 feet by 8 feet (1219 mm by 2438 mm), shall be applied vertically to wood studs not less than 2-inch (51 mm) nominal in thickness spaced 16 inches (406 mm) on center. Nailing shown in Table 23-II-J shall be provided at the perimeter of the sheathing board and at intermediate studs. Blocking not less than 2-inch (51 mm) nominal in thickness shall be provided at horizontal joints when wall height exceeds length of sheathing panel, and sheathing shall be fastened to the blocking with nails sized as shown in Table 23-II-J spaced 3 inches (76 mm) on centers each side of joint. Nails shall be spaced not less than $^3/_8$ inch (9.5 mm) from edges and ends of sheathing. Marginal studs of shear walls or shear-resisting elements shall be adequately anchored at top and bottom and designed to resist all forces. The maximum height-width ratio shall be $1^1/_2$:1.

TABLE 23-II-A-1—EXPOSED PLYWOOD PANEL SIDING

MINIMUM THICKNESS[1] (inch)	MINIMUM NUMBER OF PLIES	STUD SPACING (inches) PLYWOOD SIDING APPLIED DIRECTLY TO STUDS OR OVER SHEATHING
× 25.4 for mm		× 25.4 for mm
$^3/_8$	3	16[2]
$^1/_2$	4	24

[1]Thickness of grooved panels is measured at bottom of grooves.
[2]May be 24 inches (610 mm) if plywood siding applied with face grain perpendicular to studs or over one of the following: (1) 1-inch (25 mm) board sheathing, (2) $^7/_{16}$-inch (11 mm) wood structural panel sheathing or (3) $^3/_8$-inch (9.5 mm) wood structural panel sheathing with strength axis (which is the long direction of the panel unless otherwise marked) of sheathing perpendicular to studs.

TABLE 23-II-A-2—ALLOWABLE SPANS FOR EXPOSED PARTICLEBOARD PANEL SIDING

GRADE	STUD SPACING (inches)	MINIMUM THICKNESS (inches) × 25.4 for mm		
		Siding		Exterior Ceilings and Soffits
	× 25.4 for mm	Direct to Studs	Continuous Support	Direct to Supports
M-1 M-S M-2 “Exterior Glue”	16	$^5/_8$	$^3/_8$	$^3/_8$
	24	$^5/_8$	$^3/_8$	$^3/_8$

TABLE 23-II-B-1—NAILING SCHEDULE

CONNECTION	NAILING[1]
1. Joist to sill or girder, toenail	3-8d
2. Bridging to joist, toenail each end	2-8d
3. 1" × 6" (25 mm × 152 mm) subfloor or less to each joist, face nail	2-8d
4. Wider than 1" × 6" (25 mm × 152 mm) subfloor to each joist, face nail	3-8d
5. 2" (51 mm) subfloor to joist or girder, blind and face nail	2-16d
6. Sole plate to joist or blocking, typical face nail Sole plate to joist or blocking, at braced wall panels	16d at 16" (406 mm) o.c. 3-16d per 16" (406 mm)
7. Top plate to stud, end nail	2-16d
8. Stud to sole plate	4-8d, toenail or 2-16d, end nail
9. Double studs, face nail	16d at 24" (610 mm) o.c.
10. Doubled top plates, typical face nail Double top plates, lap splice	16d at 16" (406 mm) o.c. 8-16d
11. Blocking between joists or rafters to top plate, toenail	3-8d
12. Rim joist to top plate, toenail	8d at 6" (152 mm) o.c.
13. Top plates, laps and intersections, face nail	2-16d
14. Continuous header, two pieces	16d at 16" (406 mm) o.c. along each edge
15. Ceiling joists to plate, toenail	3-8d
16. Continuous header to stud, toenail	4-8d
17. Ceiling joists, laps over partitions, face nail	3-16d
18. Ceiling joists to parallel rafters, face nail	3-16d
19. Rafter to plate, toenail	3-8d
20. 1" (25 mm) brace to each stud and plate, face nail	2-8d
21. 1" × 8" (25 mm × 203 mm) sheathing or less to each bearing, face nail	2-8d
22. Wider than 1" × 8" (25 mm × 203 mm) sheathing to each bearing, face nail	3-8d
23. Built-up corner studs	16d at 24" (610 mm) o.c.
24. Built-up girder and beams	20d at 32" (813 mm) o.c. at top and bottom and staggered 2-20d at ends and at each splice
25. 2" (51 mm) planks	2-16d at each bearing
26. Wood structural panels and particleboard:[2]	
Subfloor and wall sheathing (to framing):	
1/2" (12.7 mm) and less	6d[3]
19/32"-3/4" (15 mm-19 mm)	8d[4] or 6d[5]
7/8"-1" (22 mm-25 mm)	8d[3]
1 1/8"-1 1/4" (29 mm-32 mm)	10d[4] or 8d[5]
Combination subfloor-underlayment (to framing):	
3/4" (19 mm) and less	6d[5]
7/8"-1" (22 mm-25 mm)	8d[5]
1 1/8"-1 1/4" (29 mm-32 mm)	10d[4] or 8d[5]
27. Panel siding (to framing)[2]:	
1/2" (12.7 mm) or less	6d[6]
5/8" (16 mm)	8d[6]
28. Fiberboard sheathing:[7]	
1/2" (12.7 mm)	No. 11 ga.[8] 6d[4] No. 16 ga.[9]
25/32" (20 mm)	No. 11 ga.[8] 8d[4] No. 16 ga.[9]
29. Interior paneling	
1/4" (6.4 mm)	4d[10]
3/8" (9.5 mm)	6d[11]

[1]Common or box nails may be used except where otherwise stated.
[2]Nails spaced at 6 inches (152 mm) on center at edges, 12 inches (305 mm) at intermediate supports except 6 inches (152 mm) at all supports where spans are 48 inches (1219 mm) or more. For nailing of wood structural panel and particleboard diaphragms and shear walls, refer to Sections 2315.3.3 and 2315.4. Nails for wall sheathing may be common, box or casing.
[3]Common or deformed shank.
[4]Common.
[5]Deformed shank.
[6]Corrosion-resistant siding or casing nails conforming to the requirements of Section 2304.3.
[7]Fasteners spaced 3 inches (76 mm) on center at exterior edges and 6 inches (152 mm) on center at intermediate supports.
[8]Corrosion-resistant roofing nails with 7/16-inch-diameter (11 mm) head and 1 1/2-inch (38 mm) length for 1/2-inch (12.7 mm) sheathing and 1 3/4-inch (44 mm) length for 25/32-inch (20 mm) sheathing conforming to the requirements of Section 2304.3.
[9]Corrosion-resistant staples with nominal 7/16-inch (11 mm) crown and 1 1/8-inch (29 mm) length for 1/2-inch (12.7 mm) sheathing and 1 1/2-inch (38 mm) length for 25/32-inch (20 mm) sheathing conforming to the requirements of Section 2304.3.
[10]Panel supports at 16 inches (406 mm) [20 inches (508 mm) if strength axis in the long direction of the panel, unless otherwise marked]. Casing or finish nails spaced 6 inches (152 mm) on panel edges, 12 inches (305 mm) at intermediate supports.
[11]Panel supports at 24 inches (610 mm). Casing or finish nails spaced 6 inches (152 mm) on panel edges, 12 inches (305 mm) at intermediate supports.

TABLE 23-II-J—ALLOWABLE SHEARS FOR WIND OR SEISMIC LOADING ON VERTICAL DIAPHRAGMS OF FIBERBOARD SHEATHING BOARD CONSTRUCTION FOR TYPE V CONSTRUCTION ONLY[1]

SIZE AND APPLICATION	NAIL SIZE	SHEAR VALUE IN POUNDS PER FOOT (N/mm) 3-INCH (76 mm) NAIL SPACING AROUND PERIMETER AND 6-INCH (152 mm) AT INTERMEDIATE POINTS × 1.46 for N/mm
1/2" × 4' × 8' (13 × 1219 × 2438 mm)	No. 11 gage galvanized roofing nail 1 1/2" (38 mm) long, 7/16" (11 mm) head	125[2]
25/32" × 4' × 8' (20 × 1219 × 2438 mm)	No. 11 gage galvanized roofing nail 1 3/4" (44 mm) long, 7/16" (11 mm) head	175

[1]Fiberboard sheathing diaphragms shall not be used to brace concrete or masonry walls.
[2]The shear value may be 175 (778 N) for 1/2-inch-by-4-foot-by-8-foot (12.7 by 1219 by 2438 mm) fiberboard nail-base sheathing.

TABLE 23-II-K—WOOD SHINGLE AND SHAKE SIDE WALL EXPOSURES

SHINGLE OR SHAKE	MAXIMUM WEATHER EXPOSURES (inches) × 25.4 for mm			
	Single-Coursing		Double-Coursing	
Length and Type	No. 1	No. 2	No. 1	No. 2
16-inch (405 mm) shingles	7 1/2	7 1/2	12	10
18-inch (455 mm) shingles	8 1/2	8 1/2	14	11
24-inch (610 mm) shingles	11 1/2	11 1/2	16	14
18-inch (455 mm) resawn shakes	8 1/2	—	14	—
18-inch (455 mm) straight-split shakes	8 1/2	—	16	—
24-inch (610 mm) resawn shakes	11 1/2	—	20	—

TABLE 23-II-L— [For BSC] MINIMUM SIZE STEEL PLATE WASHERS USED WITH HOLDOWN CONNECTORS

BOLT SIZE	*PLATE SIZE*
x 25.4 for mm	*x 25.4 for mm*
1/2 in [4]	*3/16" x 2" x 2"*
5/8 in	*1/4" x 2 1/2" x 2 1/2"*
3/4 in	*5/16" x 2 3/4" x 2 3/4"*
7/8 in	*5/16" x 3" x 3"*
1 in	*3/8" x 3 1/2" x 3 1/2"*

CACACACACACACACACACA

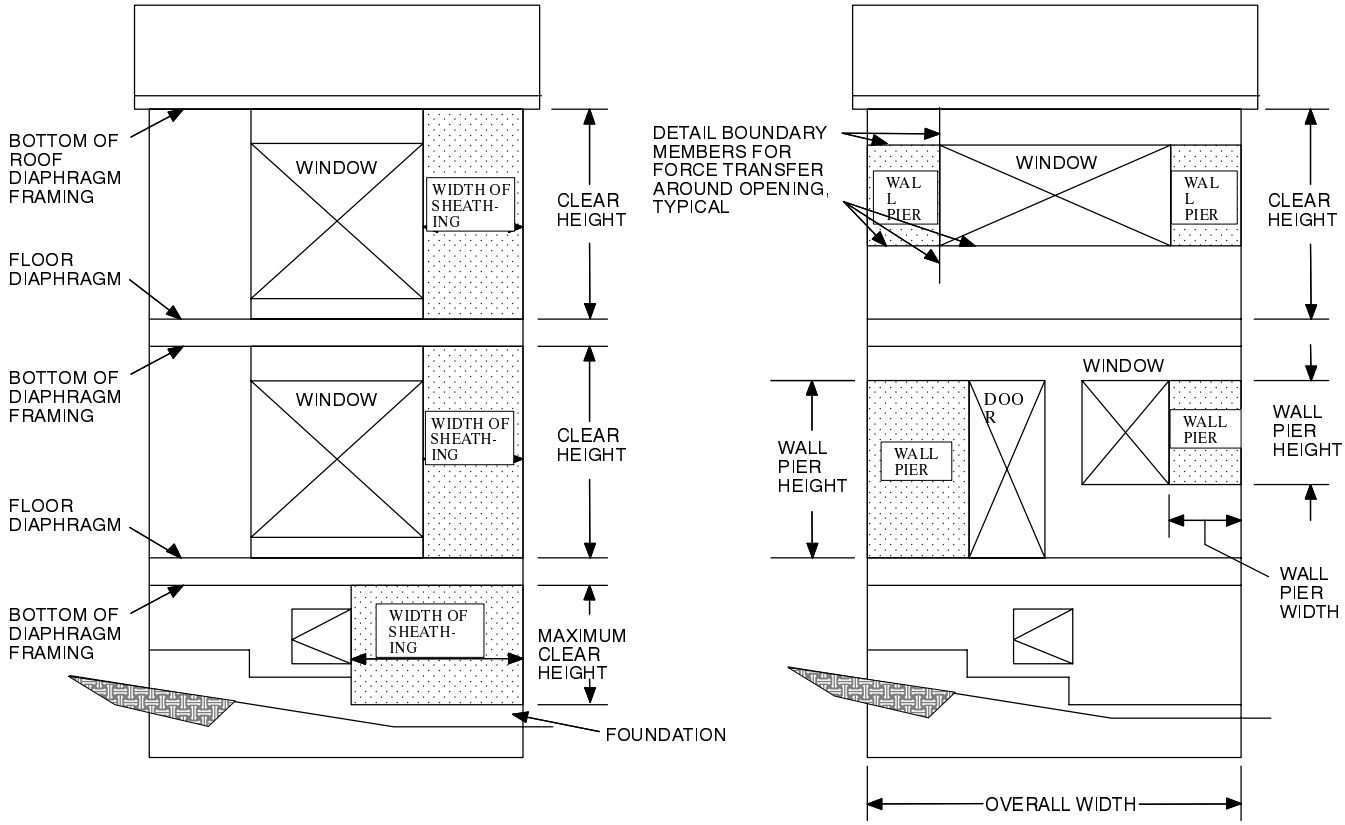


FIGURE 23-II-1—GENERAL DEFINITION OF SHEAR WALL HEIGHT TO WIDTH RATIO

Division III—DESIGN SPECIFICATIONS FOR ALLOWABLE STRESS DESIGN OF WOOD BUILDINGS

Part I—ALLOWABLE STRESS DESIGN OF WOOD

C A C A C **This standard, with certain exceptions, is the ANSI/NFoPA *NDS-97* National Design Specification for Wood Construction of the American Forest and Paper Association, *1997* Edition, and the Supplement to the *1997* Edition, National Design Specification, adopted by reference.**

C A The National Design Specification for Wood Construction, *1997* Edition, and supplement are available from the American Forest and Paper Association, 1111 19th Street, NW, Eighth Floor, Washington, DC, 20036.

SECTION 2316 — DESIGN SPECIFICATIONS

C A **2316.1 Adoption and Scope.** The National Design Specification for Wood Construction, *1997* 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.

2316.2 Amendments.

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.

|| C A **12. Sec. *3.2.3.3*. 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. *Not adopted by the State of California*.**

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.

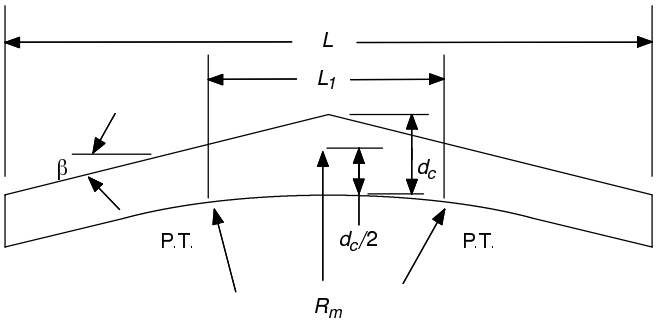
L = span of beam.

L_t = length of beam between tangent points.

A, B . . . 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.

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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, $Z_{||}$, $Z_{m\perp}$ 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.

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 1916A.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.

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