REVISION RECORD FOR THE STATE OF CALIFORNIA

EMERGENCY SUPPLEMENT

September 20, 2004

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.

This Emergency Supplement indicates the permanent adoption of emergency standards printed in the November 7, 2003 Emergency Supplement.

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suggested that the section number as well as the page number be checked when inserting this material and removing the superseded material.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It is further suggested that the superseded material be retained with this revision record sheet so that the prior wording of any section can be easily ascertained.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 that the removed pages be retained for historical reference.

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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.

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

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

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

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

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

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

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

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

1612.3.2.1 [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.*

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

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

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

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

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

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

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

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

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

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

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

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

WHERE:

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

= 0.5 for other live loads.

SECTION 1613 — DEFLECTION

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

Division II—SNOW LOADS

SECTION 1614 — SNOW LOADS

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

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

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

$$R_s = \frac{S}{40} - \frac{1}{2} \tag{14-1}$$

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. $\rho = 1.0$.

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

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

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

1629.4.2.1 [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.

CA CA CA CA CA CA CA CA

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

1630.2 Static Force Procedure.

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

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

The total design base shear need not exceed the following:

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

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

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

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

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

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

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.

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.

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-*

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-L—VERTICAL STRUCTURAL IRREGULARITIES

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

TABLE 16-M—PLAN STRUCTURAL IRREGULARITIES

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

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

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

N.L.—no limit

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

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

[3]Prohibited in Seismic Zones 3 and 4.

[4]Includes precast concrete conforming to Section 1921.2.7.

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

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

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

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

TABLE 16.1-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	—	—
	c. Heavy timber	5.6	2.2	65
	5. Special concentrically braced frames			
	a. Steel	6.4	2.2	240
3. Moment-resisting frame system	1. Special moment-resisting frame (SMRF)			
	a. Steel	8.5	2.8	N.L.
	b. Concrete[4]	8.5	2.8	N.L.
	2. Masonry moment-resisting wall frame (MMRWF)	6.5	2.8	160
	3. Intermediate moment-resisting frame (IMRF)			
	a. Steel[6]	4.5	2.8	35[6]
	b. Concrete[5]	5.5	2.8	—
	4. Ordinary moment-resisting frame (OMRF)			
	a. Steel [6]	3.5	2.8	—[6.2]
	b. Concrete[8]	3.5	2.8	—
	5. Special truss moment frames of steel (STMF)	6.5	2.8	240
4. Dual systems	1. Shear walls			
	a. Concrete with SMRF	8.5	2.8	N.L.
	b. Concrete with steel OMRF (Not Permitted)	4.2	2.8	160
	c. Concrete with concrete IMRF [5]	6.5	2.8	160
	d. Masonry with SMRF	5.5	2.8	160
	e. Masonry with steel OMRF (Not Permitted)	4.2	2.8	160
	f. Masonry with concrete IMRF [3]	4.2	2.8	—
	g. Masonry with masonry MMRWF	6.0	2.8	160
	2. Steel EBF			
	a. With steel SMRF	8.5	2.8	N.L.
	b. With steel OMRF (Not Permitted)	4.2	2.8	160
	3. Ordinary braced frames (Not Permitted)			
	a. Steel with steel SMRF	6.5	2.8	N.L.
	b. Steel with steel OMRF	4.2	2.8	160
	c. Concrete with concrete SMRF[3]	6.5	2.8	—
	d. Concrete with concrete IMRF[3]	4.2	2.8	—
	4. Special concentrically braced frames			
	a. Steel with steel SMRF	7.5	2.8	N.L.
	b. Steel with steel OMRF (Not Permitted)	4.2	2.8	160
	5. Steel IMRF (Not permitted)			
5. Cantilevered column building systems	1. Cantilevered column elements	2.2	2.0	35[7]
6. Shear wall-frame interaction systems	1. Concrete[8]	5.5	2.8	160
7. Undefined systems	See Sections 1629.6.7 and 1629.9.2	—	—	—

N.L.– no limit

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

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

[3]Prohibited in Seismic Zones 3 and 4.

[4]Includes precast concrete conforming to Section 1921.2.7.

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

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

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

[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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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.* CA CA CA 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.* CA CA CA 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.*

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 16A-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-E 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 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) 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 combined flexure and axial load shall be designed as compression members in accordance with provisions of Sections 1910.2, 1910.3, 1910.10, 1910.11, 1910.12, 1910.13, 1910.14, 1910.17, 1914.2 and 1914.3.

1914.5 Empirical Design Method.

1914.5.1 Walls of solid rectangular cross section shall be permitted to be designed by the empirical provisions of Section 1914.5 if resultant of all factored loads is located within the middle third of the overall thickness of wall and all limits of Sections 1914.2, 1914.3 and 1914.5 are satisfied.

1914.5.2 Design axial load strength ϕP_{nw} of a wall satisfying limitations of Section 1914.5.1 shall be computed by Formula (14-1) unless designed in accordance with Section 1914.4.

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

where $\phi = 0.70$ and effective length factor k shall be:

For walls braced top and bottom against lateral translation and

1. Restrained against rotation at one or both ends (top and/or bottom) 0.8
2. Unrestrained against rotation at both ends 1.0

For walls not braced against lateral translation 2.0

1914.5.3 Minimum thickness of walls designed by empirical design method.

1914.5.3.1 Thickness of bearing walls shall not be less than $^1/_{25}$ the supported height or length, whichever is shorter, or not less than 4 inches (102 mm).

1914.5.3.2 Thickness of exterior basement walls and foundation walls shall not be less than $7^1/_2$ inches (191 mm).

1914.6 Nonbearing Walls.

1914.6.1 Thickness of nonbearing walls shall not be less than 4 inches (102 mm), or not less than $^1/_{30}$ the least distance between members that provide lateral support.

1914.7 Walls as Grade Beams.

1914.7.1 Walls designed as grade beams shall have top and bottom reinforcement as required for moment in accordance with provisions of Sections 1910.2 through 1910.7. Design for shear shall be in accordance with provisions of Section 1911.

1914.7.2 Portions of grade beam walls exposed above grade shall also meet requirements of Section 1914.3.

1914.8 Alternate Design Slender Walls.

1914.8.1 When flexural tension controls design of walls, the requirements of Section 1910.10 may be satisfied by complying with the limitations and procedures set forth in this section.

1914.8.2 The following limitations apply when this section is employed.

1. Vertical service load stress at the location of maximum moment does not exceed 0.04 f'_c.

2. The reinforcement ratio ρ does not exceed 0.6 ρ_b.

3. Sufficient reinforcement is provided so that the nominal moment capacity times the ϕ factor is greater than M_{cr}.

4. Distribution of concentrated load does not exceed the width of bearing plus a width increasing at a slope of 2 vertical to 1 horizontal down to the design flexural section.

1914.8.3 The required factored moment, M_u at the midheight cross section for combined axial and lateral factored loads, including the $P\,\Delta$ moments, shall be as set forth in Formula (14-2).

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

Unless a more comprehensive analysis is used, the $P\,\Delta$ moment shall be calculated using the maximum potential deflection, Δ_n, as defined in Section 1914.8.4.

1914.8.4 The midheight deflection Δ_s, under service lateral and vertical loads (without load factors), shall be limited by the relation

$$\Delta_s = \frac{l_c}{150} \qquad (14\text{-}3)$$

Unless a more comprehensive analysis is used, the midheight deflection shall be computed with the following formulas:

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

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

WHERE:

$$A_{se} = \frac{P_u + A_s f_y}{f_y}$$

$$I_{cr} = nA_{se}(d - c)^2 + \frac{bc^3}{3}$$

M_s = *the maximum moment in the wall resulting from the application of the unfactored load combinations.*

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

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

SECTION 1915 — FOOTINGS

1915.0 Notations.

A_g = gross area of section, square inches (mm^2).
d_p = diameter of pile at footing base.
β = ratio of long side to short side of footing.

1915.1 Scope.

1915.1.1 Provisions of this section shall apply for design of isolated footings and, where applicable, to combined footings and mats.

1915.1.2 Additional requirements for design of combined footings and mats are given in Section 1915.10.

1915.2 Loads and Reactions.

1915.2.1 Footings shall be proportioned to resist the factored loads and induced reactions, in accordance with the appropriate design requirements of this code and as provided in this section.

1915.2.2 Base area of footing or number and arrangement of piles 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

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. $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
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. $1.2D \pm 1.0E + (0.5L \text{ or } 0.2S)$
6. $0.9D \pm (1.3W \text{ or } 1.0E)$

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.

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.* CA CA CA

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.* CA CA CA

SECTION 2205 — DESIGN AND CONSTRUCTION PROVISIONS

2205.1 General. The following design standards shall apply.

2205.2 Structural Steel Construction. The design, fabrication and erection of structural steel shall be in accordance with the requirements of Division II for Load and Resistance Factor Design or Division III for Allowable Stress Design.

2205.3 Seismic Design Provisions for Structural Steel. Steel structural elements that resist seismic forces shall, in addition to the requirements of Section 2205.2, be designed in accordance with Division IV or V.

2205.4 Cold-formed Steel Construction. The design of cold-formed carbon or low-alloy steel structural members shall be in accordance with the requirements of Division VI for Load and Resistance Factor Design or Division VII for Allowable Stress Design.

2205.5 Cold-formed Stainless Steel Construction. The design of cold-formed stainless steel structural members shall be in accordance with approved national standards (see Section 2202).

2205.6 Design Provisions for Stud Wall Systems. Cold-formed steel stud wall systems that serve as part of the lateral-force-resisting system shall, in addition to the requirements of Section 2205.4 or 2205.5, be designed and constructed in accordance with Division VIII.

2205.7 Open-web Steel Joists and Joist Girders. The design, manufacture and use of steel joist, K, LH, and KLH series and joist girders shall be in accordance with Division IX.

2205.8 Steel Storage Racks. Steel storage racks may be designed in accordance with the provisions of Division X, except that in Seismic Zones 3 and 4 wholesale and retail sales areas, the *W* used in the design of racks over 8 feet (2438 mm) in height shall be equal to the weight of the rack structure and contents with no reductions.

2205.9 Steel Cables. Structural applications of steel cables for buildings shall be in accordance with the provisions of Division XI.

2205.10 Welding. Welding procedures, welder qualification requirements and welding electrodes shall be in accordance with Division II, III, VI or VII and approved national standards.

2205.11 Bolts. The use of high-strength A 325 and A 490 bolts shall be in accordance with the requirements of Divisions II and III.

Anchor bolts shall be set accurately to the pattern and dimensions called for on the plans. The protrusion of the threaded ends through the connected material shall be sufficient to fully engage the threads of the nuts, but shall not be greater than the length of threads on the bolts. Base plate holes for anchor bolts may be oversized as follows:

Bolt Size, inches (mm)	Hole Size, inches (mm)
$^{3}/_{4}$ (19.1)	$^{5}/_{16}$ (7.9) oversized
$^{7}/_{8}$ (22.2)	$^{5}/_{16}$ (7.9) oversized
1 < 2 (25.4 < 50.8)	$^{1}/_{2}$ (12.7) oversized
> 2 (> 50.8)	1 (25.4) > bolt diameter

Chapter 22B *[BSC]*

STEEL

Division I—DESIGN AND CONSTRUCTION PROVISIONS

See Chapter 22, STEEL, Division I for content of this division.

Division II—DESIGN STANDARD FOR LOAD AND RESISTANCE FACTOR DESIGN SPECIFICATION FOR STRUCTURAL STEEL BUILDINGS

See Chapter 22, STEEL, Division II for content of this division.

Division III—DESIGN STANDARD FOR SPECIFICATION FOR STRUCTURAL STEEL BUILDINGS ALLOWABLE STRESS DESIGN AND PLASTIC DESIGN

See Chapter 22, STEEL, Division III for content of this division.

Division IV—SEISMIC PROVISIONS FOR STRUCTURAL STEEL BUILDINGS

Based on Seismic Provisions for Structural Steel Buildings of the American Institute of Steel Construction.

(Part I, dated April 15, 1997 and Supplement No. 2, dated November 10, 2000.)

SECTION 2210B — ADOPTION

Except for the modifications as set forth in Sections 2211B and 2212B of this division and the requirements of the Building Code, the seismic design, fabrication, and erection of structural steel shall be in accordance with the Seismic Provisions for Structural Steel Buildings, April 15, 1997, published by the American Institute of Steel Construction, 1 East Wacker Drive, Suite 3100, Chicago, IL 60601, as if set out at length herein. The adoption of Seismic Provisions for Structural Steel Buildings in this Division, hereinafter referred to as AISC-Seismic, shall include Parts I (LRFD), and Supplement No. 2, dated November 10, 2000.

Where other codes, standards, or specifications are referred to in this specification, they are to be considered as only an indication of an acceptable method or material that can be used with the approval of the Building Official.

SECTION 2211B — DESIGN METHODS

When the load combinations from Section 1612.2 for LRFD are used, structural steel buildings shall be designed in accordance with Chapter 22 Division II (AISC-LRFD) and Part I of AISC-Seismic as modified by this Division.

SECTION 2212B — AMENDMENTS

The AISC-Seismic adopted by this Division applies to the seismic design of structural steel members except as modified by this Section.

The following terms that appear in AISC-Seismic shall be taken as indicated in the 1997 Uniform Building Code.

AISC-Seismic	*1997 Uniform Building Code*
Seismic Force Resisting System	*Lateral Force Resisting System*
Design Earthquake	*Design Basis Ground Motion*
Load Combinations Eqs. (4-1) and (4-2)	*Chapter 16 Eqs. (12-17) and (12-18) respectively*
LRFD Specification Section Eqs. (A4-1) through (A4-6)	Chapter 16 Eqs. (12-1) through (12-6) respectively
$\Omega_o Q_E$	E_m

1. Part I, Sec. 1. of the AISC Seismic Provisions is revised as follows:

1. SCOPE

These provisions are intended for the design and construction of structural steel members and connections in the Seismic Force Resisting Systems in buildings for which the design forces resulting from earthquake motions have been determined on the basis of various levels of energy dissipation in the inelastic range of response. These provisions shall apply to buildings in Seismic Zone 2 with an importance factor I greater than one, in Seismic Zones 3 and 4 or when required by the Engineer of Record.

These provisions shall be applied in conjunction with Chapter 22, Division II, hereinafter referred to as the LRFD Specification. All members and connections in the Lateral Force Resisting System shall have a design strength as provided in the LRFD Specification to resist load combinations 12-1 through 12-6 (in Chapter 16) and shall meet the requirements in these provisions.

Part I includes a Glossary, which is specifically applicable to this Part, and Appendix S.

2. Part I, Sec. 4.1. of the AISC Seismic Provisions is deleted and replaced as follows:

4.1 Loads and Load Combinations

The loads and load combinations shall be those in Section 1612.2 except as modified throughout these provisions.

E_h is the horizontal component of earthquake load E required in Chapter 16. Where required in these provisions, an amplified horizontal earthquake load $\Omega_o E_h$ shall be used in lieu of E_h as given in the load combinations below. The term Ω_o is the system overstrength factor as defined in chapter 16. The additional load combinations using amplified horizontal earthquake load are:

$$1.2\,D + 0.5\,L + 0.2S + \Omega_o E_h \qquad (4\text{-}1)$$

$$0.9\,D + \Omega_o E_h \qquad (4\text{-}2)$$

EXCEPTION: *The load factor on L in load combination 4-1 shall be equal to 1.0 for garages, areas occupied as places of public assembly and all areas where the live load is greater than 100 psf.*

SECTION 2213B — RESERVED

SECTION 2214B — RESERVED

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.

2215B

AISC-Seismic	*1997 Uniform Building Code*
Seismic Force Resisting System	*Lateral Force Resisting System*
Design Earthquake	*Design Basis Ground Motion*
Load Combinations Eqs. (4-1) and (4-2)	*Chapter 16 Eqs. (12-17) and (12-18) respectively*
$\Omega_o Q_E$	E_m

1. Part III, Sec. 1. of the AISC Seismic Provisions is revised as follows:

1. SCOPE

These provisions are intended for the design and construction of structural steel members and connections in the Seismic Force Resisting Systems in buildings for which the design forces resulting from earthquake motions have been determined on the basis of various levels of energy dissipation in the inelastic range of response. These provisions shall apply to buildings in Seismic Zone 2 with an importance factor I greater than one, in Seismic Zones 3 and 4 or when required by the Engineer of Record.

These provisions shall be applied in conjunction with Chapter 22, Division III, hereinafter referred to as the ASD Specification. All members and connections in the Lateral Force Resisting System shall have a design strength as provided in the ASD Specification to resist load combinations 12-1 through 12-6 (in Chapter 16) and shall meet the requirements in these provisions.

Part I includes a Glossary, which is specifically applicable to this Part, and Appendix S.

2. Part III, Sec. 4.1. of the AISC Seismic Provisions is deleted and replaced as follows:

2.1 Loads and Load Combinations

The loads and load combinations shall be those in Section 1612.2 except as modified throughout these provisions.

E_h is the horizontal component of earthquake load E required in Chapter 16. Where required in these provisions, an amplified horizontal earthquake load $\Omega_o E_h$ shall be used in lieu of E_h as given in the load combinations below. The term Ω_o is the system overstrength factor as defined in chapter 16. The additional load combinations using amplified horizontal earthquake load are:

$$1.2\,D + 0.5\,L + 0.2S + \Omega_o E_h \qquad (4\text{-}1)$$

$$0.9\,D + \Omega_o E_h \qquad (4\text{-}2)$$

EXCEPTION: *the load factor on L in load combination 4-1 shall be equal to 1.0 for garages, areas occupied as places of public assembly and all areas where the live load is greater than 100 psf.*

Division V—RESERVED

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.

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	16	$^5/_8$	$^3/_8$	$^3/_8$
M-2 "Exterior Glue"	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 19/32"-3/4" (15 mm-19 mm) 7/8"-1" (22 mm-25 mm) 1 1/8"-1 1/4" (29 mm-32 mm) Combination subfloor-underlayment (to framing): 3/4" (19 mm) and less 7/8"-1" (22 mm-25 mm) 1 1/8"-1 1/4" (29 mm-32 mm)	 6d[3] 8d[4] or 6d[5] 8d[3] 10d[4] or 8d[5] 6d[5] 8d[5] 10d[4] or 8d[5]
27. Panel siding (to framing)[2]: 1/2" (12.7 mm) or less 5/8" (16 mm)	 6d[6] 8d[6]
28. Fiberboard sheathing:[7] 1/2" (12.7 mm) 25/32" (20 mm)	 No. 11 ga.[8] 6d[4] No. 16 ga.[9] No. 11 ga.[8] 8d[4] No. 16 ga.[9]
29. Interior paneling 1/4" (6.4 mm) 3/8" (9.5 mm)	 4d[10] 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	*$^{3}/_{16}$" 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 $3^{1}/_{2}$"*

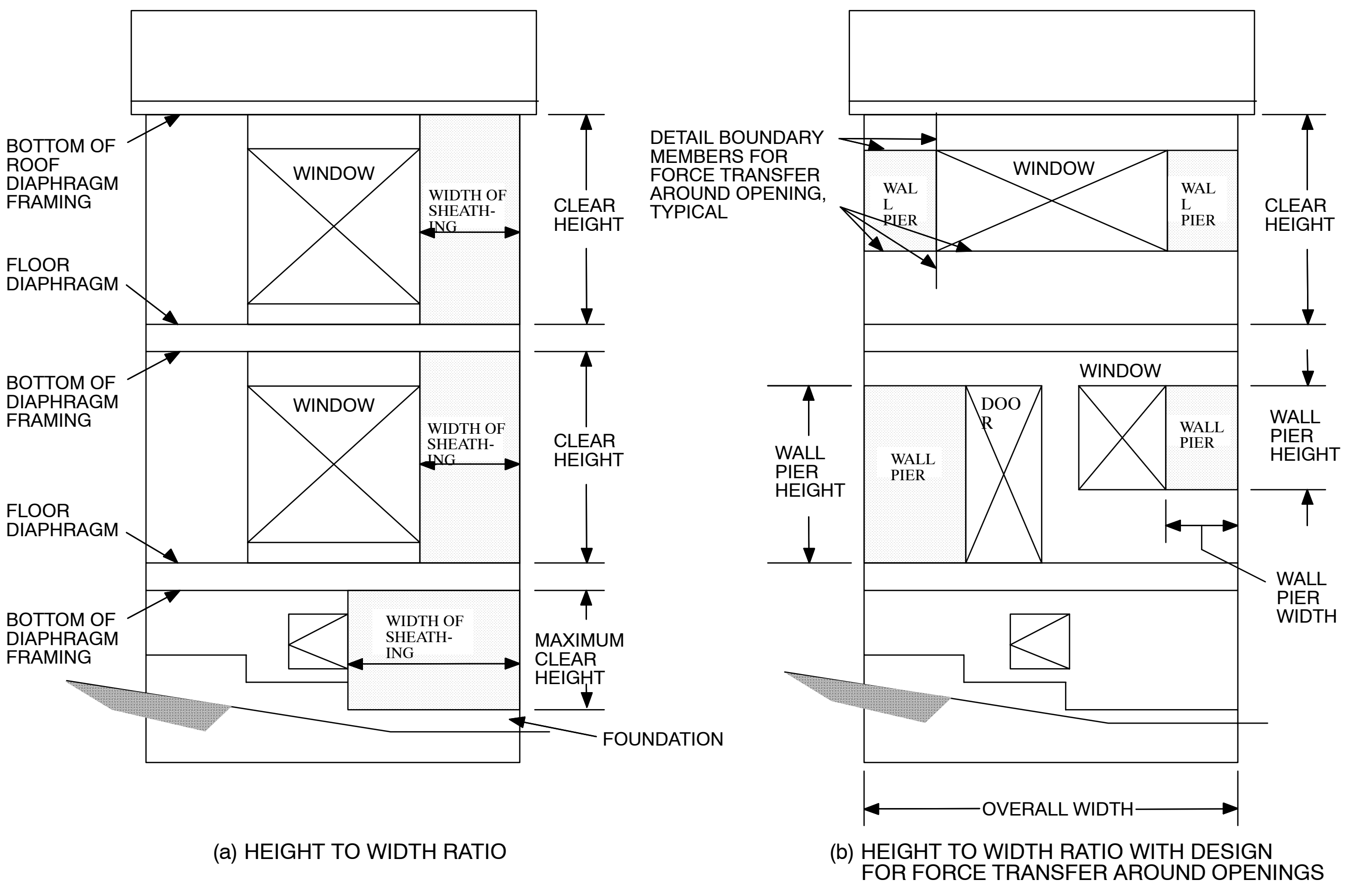


FIGURE 23-II-1—GENERAL DEFINITION OF SHEAR WALL HEIGHT TO WIDTH RATIO

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

Part I—ALLOWABLE STRESS DESIGN OF WOOD

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

The National Design Specification for Wood Construction, Edition *[for BSC, NDS-97]*, and supplement are available from the American Forest and Paper Association, 1111 19th Street, NW, Eighth Floor, Washington, DC, 20036.

SECTION 2301 — DESIGN SPECIFICATIONS

2301.1 Adoption and Scope. The National Design Specification for Wood Construction, Edition (NDS), *[for BSC, 1997 Edition (NDS) as amended by Sec. 2316.2]* which is hereby adopted *[for BSC except for Items 14, 26 and 27]* 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, *[for BSC, NDS-97]* 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.

2301.2 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 23A-II-H, 23A-II-I-1, 23A-II-I-2, 23A-II-J or in Section 2315A.3.

[2]1.60 may be used for members and nailed and bolted connections exhibiting Mode III or IV behavior, except that the increases for wind are not combined with the increase allowed in Section 1612.3. The 60-percent increase shall not apply to the allowable shear values in Tables 23A-II-H, 23A-II-I-1, 23A-II-I-2, 23A-II-J or in Section 2315A.3.

7. Sec. 2.3.4. Add a second paragraph following Table 2.3.4:

The allowable unit stresses for fire-retardant-treated solid-sawn lumber and plywood, including fastener values, subject to prolonged elevated temperatures from manufacturing or equipment processes, but not exceeding 150°F (66°C), shall be developed from approved test methods that properly consider potential strength-reduction characteristics, including effects of heat and moisture.

8. Sec. 2.3.6. Add second, third and fourth paragraphs as follows:

The values for lumber and plywood impregnated with approved fire-retardant chemicals, including fastener values, shall be submitted to the building official for approval. Submittal to the building official shall include all substantiating data. Such values shall be developed from approved test methods and procedures that consider potential strength-reduction characteristics, including the effects of elevated temperatures and moisture. Other adjustments are applicable, except that the impact load-duration factor shall not apply.

Values for glued-laminated timber, including fastener design values, shall be recommended by the treater and submitted to the building official for approval. Submittal to the building official shall include all substantiating data.

In addition to the requirements specified in Section 207, fire-retardant lumber having structural applications shall be tested and identified by an approved inspection agency in accordance with UBC Standard 23-5.

9. Sec. 2.3.8. Add new second and third paragraphs following Table 2.3.8:

For lumber I beams and box beams, the form factor, C_f, shall be calculated as:

$$C_f = \left[1 + \left(\frac{d^2 + 143}{d^2 + 88} - 1\right) C_g\right]$$

For **SI:**

$$C_f = \left[1 + \left(\frac{\left(\frac{d}{25.4}\right)^2 + 143}{\left(\frac{d}{25.4}\right)^2 + 88} - 1\right) C_g\right]$$

WHERE:

C_f = form factor.

C_g = support factor = $p^2(6 - 8p + 3p^2)(1 - q) + q$.

d = depth of I or box beam.

p = ratio of depth of compression flange to full depth of beam.

q = ratio of thickness of web or webs to full width of beam.

10. Sec. 2.3.10. Add a paragraph at end of section as follows:

In joists supported on a ribbon or ledger board and spiked to the studding, the allowable stress in compression perpendicular to grain may be increased 50 percent.

11. Sec. 3.2.1. Add a second sentence as follows:

For continuous beams, the span shall be taken as the distance between centers of bearings on supports over which the beam is continuous.

12. Sec. *3.2.3.3*. 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.

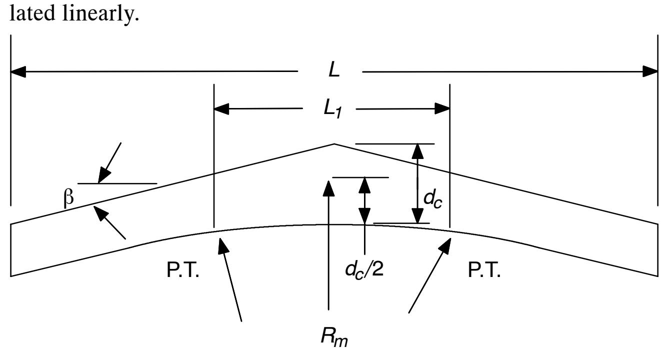
L = span of beam.

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.

Part II—PLYWOOD STRUCTURAL PANELS

SECTION 2302 — PLYWOOD STRUCTURAL PANELS

Values for plywood structural panels shall be in accordance with Table 23-III-A.

Part III—FASTENINGS

SECTION 2303 — TIMBER CONNECTORS AND FASTENERS

2303.1 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.

2303.2 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.

2303.3 Nails and Spikes.

2303.3.1 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 2305.

2303.3.2 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.

2303.3.3 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.

2303.4 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.

2303.5 Miscellaneous Fasteners.

2303.5.1 Drift Bolts and Drift Pins.

2303.5.1.1 Withdrawal design values. Drift bolt and drift pin connections loaded in withdrawal shall be designed in accordance with good engineering practice.

2303.5.1.2 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.

2303.5.2 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 2304 — WOOD SHEAR WALLS AND DIAPHRAGMS

2304.1 Conventional Lumber Diaphragms. Conventional lumber diaphragms of Douglas fir-larch or Southern pine, constructed in accordance with Section 2315A.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.

2304.2 Special Lumber Diaphragms. Special diagonally sheathed diaphragms of Douglas fir-larch or Southern pine, constructed in accordance with Section 2315A.3.2, may be used to resist shears due to wind or seismic loads, provided such shear do not stress the nails beyond their allowable safe lateral strength and do not exceed 600 pounds per lineal foot (8.75 kN/m) of width. Where nails are used with sheathing and framing members with a specific gravity less than 0.49, the allowable unit shear strength of the diaphragm shall be multiplied by the following factors: 0.82 for species with specific gravity greater than or equal to 0.42 but less than 0.49, and 0.65 for species with a specific gravity less than 0.42.

2304.3 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 23A-II-H for horizontal diaphragms and Table 23A-II-I-1 for vertical diaphragms.

Where the wood structural panel is applied to both faces of a shear wall in accordance with Table 23A-II-I-1, allowable shear for the wall may be taken as twice the tabulated shear for one side, except that where the shear capacities are not equal, the allowable shear shall be either the shear for the side with the higher capacity or twice the shear for the side with the lower capacity, whichever is greater.

2304.4 Particleboard Diaphragms. Vertical diaphragms sheathed with particleboard may be used to resist horizontal forces not exceeding those set forth in Table 23A-II-I-2.

2304.5 Fiberboard Sheathing Diaphragms. Wood stud walls sheathed with fiberboard sheathing may be used to resist horizontal forces not exceeding those set forth in Table 23A-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 l916A.7.1.*"

Page 2-236.11: Revise the title of **Section 2106A.1.12.4** Item 2. to "***Shear walls.***" Revise the title of **Section 2106A.2.3.3** to "***Walls and piers.***" and the heading "***Thickness of Walls.***" to "***Thickness of walls.***"

5. (DSA/SS EF 01/03) Emergency adoption/approval of technical design and construction building standards for the adaptive reuse of existing building public school use; CCR, Title 24, Part 2. Approved by the California Building Standards Commission on May 14, 2003 and filed with Secretary of State on May 15, 2003. Effective May 15, 2003.

6. (DSA/SS EF 03/03) Emergency re-adoption/re-approval of technical design and construction building standards for the adaptive reuse of existing building public school use; CCR, Title 24, Part 2. Approved by the California Building Standards Commission on July 16, 2003 and filed with Secretary of State on May 15, 2003. Effective September 10, 2003.

7. (BSC EF 1/03) Amend Title 24, Part 2, Vol. 2, Chapters 2, 16, 17, 19, 22B and 23. Various sections. Filed with the Secretary of State on July 18, 2003. July 18, 2003.

8. (DSA/SS 3/02) Adoption of various amendments to the California Building Code (CCR, Title 24, Part 2) for seismic design of irregular structures. Approved by the Building Standards Commission on May 14, 2003 and effective July 30, 2004.

9. (OSHPD 3/02) Adoption of various amendments to the California Building Code (CCR, Title 24, Part 2) for seismic design of irregular structures. Approved by the Building Standards Commission on May 14, 2003 and effective July 30, 2004.

10. (DSA/SS EF 03/03) Emergency re-adoption/re-approval of technical design and construction building standards for the adaptive reuse of existing building public school use; CCR, Title 24, Part 2, Vol. 2. Approved as permanent by the California Building Standards Commission on January 7, 2004 and filed with the Secretary of State on January 8, 2004. Effective January 8, 2004.

11. (BSC EF 1/03) Amend Title 24, Part 2, Vol. 2, Chapters 2, 16, 17, 19, 22, 22B and 23, various sections; filed as permanent adoption with the Secretary of State on September 20, 2004; effective date September 20, 2004.

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