REVISION RECORD FOR THE STATE OF CALIFORNIA

SUPPLEMENT

January 30, 2004

2001 Title 24, Part 2, California Building Code

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

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suggested that the section number as well as the page number be checked when inserting this material and removing the superseded material.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

It is further suggested that the superseded material be retained with this revision record sheet so that the prior wording of any section can be easily ascertained.

Please keep the removed pages with this revision page for future reference.

NOTE

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specifications, and construction for that permit, it is strongly recommended that the removed pages be retained for historical reference.

VOLUME 2

Remove Existing Pages	Insert Blue Pages
2-38.13 through 2-38.28	2-38.13 through 2-38.28
2-38.37 and 2-38.38	2-38.37 and 2-38.38
2-272.5 and 2-272.6	2-272.5 and 2-272.6
2-272.9 and 2-272.10	2-272.9 and 2-272.10
2-497 and 2-498	2-497 and 2-498

Item No. 5522SP01

ECCENTRICALLY BRACED FRAME (EBF) is a steel-braced frame designed in conformance with Section 2213A.10.

ELASTIC RESPONSE PARAMETERS are forces and deformations determined from an elastic dynamic analysis using an unreduced ground motion representation, in accordance with Section 1630A.

ESSENTIAL FACILITIES are those structures that are necessary for emergency operations subsequent to a natural disaster.

FLEXIBLE ELEMENT or system is one whose deformation under lateral load is significantly larger than adjoining parts of the system. Limiting ratios for defining specific flexible elements are set forth in Section 1630A.6.

HORIZONTAL BRACING SYSTEM is a horizontal truss system that serves the same function as a diaphragm.

HOSPITAL BUILDINGS *are hospital buildings and all other medical facilities as defined in Section 1250, Health and Safety Code.*

INCIDENTAL STRUCTURAL ALTERATIONS OR ADDITIONS *are alterations or additions which would not reduce the story lateral shear force-resisting capacity by more than 5 percent or increase the story shear by more than 5 percent in any existing story.*

IRREGULAR STRUCTURE *is a structure designated as having one or more plan or vertical irregularities.*

LATERAL-FORCE-RESISTING SYSTEM is that part of the structural system designed to resist the Design Seismic Forces.

MAJOR STRUCTURAL ALTERATIONS OR ADDITIONS *are those alterations or additions of greater extent than minor structural alterations or additions.*

MINOR STRUCTURAL ALTERATIONS OR ADDITIONS *are alterations or additions of greater extent than incidental structural alterations or additions which would not reduce the story shear lateral-force-resisting capacity by more than 10 percent or increase the base shear by more than 10 percent.*

MOMENT-RESISTING FRAME is a frame in which members and joints are capable of resisting forces primarily by flexure.

MOMENT-RESISTING WALL FRAME (MRWF) is a masonry wall frame especially detailed to provide ductile behavior and designed in conformance with Section 2108A.2.5.

NONREQUIRED STRUCTURAL ALTERATION *is any alteration of existing structural elements or provision of new structural elements which is not necessary for vertical or lateral support of other work and is initiated by the applicant primarily for the purpose of increasing the vertical or lateral load-carrying strength or stiffness of an existing building.*

NONSTRUCTURAL ALTERATION *is any alteration which neither affects existing structural elements nor requires new structural elements for vertical or lateral support and which does not increase the lateral shear force in any story by more than 5 percent.*

ORDINARY MOMENT-RESISTING FRAME (OMRF) ***[Not adopted by OSHPD]*** is a moment-resisting frame not meeting special detailing requirements for ductile behavior.

ORTHOGONAL EFFECTS are the earthquake load effects on structural elements common to the lateral-force-resisting systems along two orthogonal axes.

OVERSTRENGTH is a characteristic of structures where the actual strength is larger than the design strength. The degree of overstrength is material- and system-dependent.

$P\Delta$ EFFECT is the secondary effect on shears, axial forces and moments of frame members induced by the vertical loads acting on the laterally displaced building system.

RECONSTRUCTION *means the rebuilding of any existing building to bring it into full compliance with these regulations.*

SHEAR WALL is a wall designed to resist lateral forces parallel to the plane of the wall (sometimes referred to as vertical diaphragm or structural wall).

SHEAR WALL-FRAME INTERACTIVE SYSTEM uses combinations of shear walls and frames designed to resist lateral forces in proportion to their relative rigidities, considering interaction between shear walls and frames on all levels.

SOFT STORY is one in which the lateral stiffness is less than 70 percent of the stiffness of the story above. See Table 16A-L.

SOIL-STRUCTURE RESONANCE *is the coincidence of the natural period of a structure with a dominant frequency of the ground motion.*

SPACE FRAME is a three-dimensional structural system, without bearing walls, composed of members interconnected so as to function as a complete self-contained unit with or without the aid of horizontal diaphragms or floor-bracing systems.

SPECIAL CONCENTRICALLY BRACED FRAME (SCBF) is a steel-braced frame designed in conformance with the provisions of Section 2213A.9.

SPECIAL MOMENT-RESISTING FRAME (SMRF) is a moment-resisting frame specially detailed to provide ductile behavior and comply with the requirements given in Chapter 19A or 22A.

SPECIAL TRUSS MOMENT FRAME (STMF) is a moment-resisting frame specially detailed to provide ductile behavior and comply with the provisions of Section 2213A.11.

STORY is the space between levels. Story x is the story below Level x.

STORY DRIFT is the lateral displacement of one level relative to the level above or below.

STORY DRIFT RATIO is the story drift divided by the story height.

STORY SHEAR, V_x, is the summation of design lateral forces above the story under consideration.

STRENGTH is the capacity of an element or a member to resist factored load as specified in Chapters 16A, 18A, 19A, 21A and 22A.

STRUCTURAL ELEMENTS *are floor or roof diaphragms, decking, joists, slabs, beams or girders, columns, bearing walls, retaining walls, masonry or concrete nonbearing walls exceeding one story in height, foundations, shear walls or other lateral-force-resisting members, and any other elements necessary to the vertical and lateral strength or stability of either the building as a whole or any of its parts, including connections between such elements.*

STRUCTURAL REPAIRS *are any changes affecting existing or requiring new structural elements primarily intended to correct the effects of deterioration or impending or actual failure, regardless of cause.*

STRUCTURE is an assemblage of framing members designed to support gravity loads and resist lateral forces. Structures may be categorized as building structures or nonbuilding structures.

SUBDIAPHRAGM is a portion of a larger wood diaphragm designed to anchor and transfer local forces to primary diaphragm struts and the main diaphragm.

VERTICAL LOAD-CARRYING FRAME is a space frame designed to carry vertical gravity loads.

WALL ANCHORAGE SYSTEM is the system of elements anchoring the wall to the diaphragm and those elements within the diaphragm required to develop the anchorage forces, including subdiaphragms and continuous ties, as specified in Sections 1633A.2.8 and 1633A.2.9.

WEAK STORY is one in which the story strength is less than 80 percent of the story above. See Table 16A-L.

SECTION 1628A — SYMBOLS AND NOTATIONS

The following symbols and notations apply to the provisions of this division:

A_B = ground floor area of structure in square feet (m^2) to include area covered by all overhangs and projections.

A_c = the combined effective area, in square feet (m^2), of the shear walls in the first story of the structure.

A_e = the minimum cross-sectional area in any horizontal plane in the first story, in square feet (m^2) of a shear wall.

A_x = the torsional amplification factor at Level x.

a_p = numerical coefficient specified in Section 1632A and set forth in Table 16A-O.

C_a = seismic coefficient, as set forth in Table 16A-Q.

C_t = numerical coefficient given in Section 1630A.2.2.

C_v = seismic coefficient, as set forth in Table 16A-R.

D = dead load on a structural element.

D_e = the length, in feet (m), of a shear wall in the first story in the direction parallel to the applied forces.

E, E_h, E_m, E_v = earthquake loads set forth in Section 1630A.1.

F_i, F_n, F_x = Design Seismic Force applied to Level i, n or x, respectively.

F_p = Design Seismic Forces on a part of the structure.

F_{px} = Design Seismic Force on a diaphragm.

F_t = that portion of the base shear, V, considered concentrated at the top of the structure in addition to F_n.

f_i = lateral force at Level i for use in Formula (30A-10).

g = acceleration due to gravity.

h_i, h_n, h_x = height in feet (m) above the base to Level i, n or x, respectively.

I = importance factor given in Table 16A-K.

I_p = importance factor specified in Table 16A-K.

L = live load on a structural element.

Level i = level of the structure referred to by the subscript i. "i = 1" designates the first level above the base.

Level n = that level that is uppermost in the main portion of the structure.

Level x = that level that is under design consideration. "x = 1" designates the first level above the base.

M = maximum moment magnitude.

N_a = near-source factor used in the determination of C_a in Seismic Zone 4 related to both the proximity of the building or structure to known faults with magnitudes and slip rates as set forth in Tables 16A-S and 16A-U.

N_v = near-source factor used in the determination of C_v in Seismic Zone 4 related to both the proximity of the building or structure to known faults with magnitudes and slip rates as set forth in Tables 16A-T and 16A-U.

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 16A-N or 16A-P.

r = a ratio used in determining ρ. See Section 1630A.1.

$S_A, S_B, S_C, S_D, S_E, S_F$ = soil profile types as set forth in Table 16A-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 (30A-5), (30A-6), (30A-7) or (30A-11).

V_x = the design story shear in Story x.

W = the total seismic dead load defined in Section 1630A.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 1630A.1.1.

Z = seismic zone factor as given in Table 16A-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 1630A.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 (30A-10).

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

Ω_o = Seismic Force Amplification Factor, which is required to account for structural overstrength and set forth in Table 16A-N.

SECTION 1629A — CRITERIA SELECTION

1629A.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 1630A, except as modified by Section 1631A.5.4. Where strength design is used, the load combinations of Section 1612A.2 shall apply. Where Allowable Stress Design is used, the load combinations of Section 1612A.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 1612A.3 are utilized. One- and two-family dwellings in Seismic Zone 1 need not conform to the provisions of this section.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

2. $\rho = 1.0$.

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

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

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

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

1629A.5 Configuration Requirements.

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

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

1629A.5.3 Irregular structures.

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

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

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

1629A.6 Structural Systems.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

1629A.8 Selection of Lateral-force Procedure.

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

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

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

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

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

1. *Not adopted by OSHPD and DSA.*

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

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

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

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

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

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

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

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

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

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

1629A.9 System Limitations.

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

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

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

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

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

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

1629A.10 Alternative Procedures.

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

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

[For OSHPD 1 & 4] For alternate design procedures on seismic isolation systems, refer to Appendix Chapter 16A, Division IV, Earthquake Regulations for Seismic-Isolated Structures.

[For DSA/SS] For alternative design procedures on seismic isolation systems, refer to Chapter 16B, Division IV, Earthquake Regulations for Seismic-Isolated Structures, 1998, California Building Code, Volume 2B.

SECTION 1630A — MINIMUM DESIGN LATERAL FORCES AND RELATED EFFECTS

1630A.1 Earthquake Loads and Modeling Requirements.

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

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

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

WHERE:

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

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

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

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

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

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

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

For **SI:**

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

WHERE:

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

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

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

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

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

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

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

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

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

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

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

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

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

4. Total weight of permanent equipment shall be included.

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

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

1. Stiffness properties of reinforced concrete and masonry elements shall consider the effects of cracked sections.

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

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

1630A.2 Static Force Procedure.

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

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

The total design base shear need not exceed the following:

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

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

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

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

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

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

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

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

WHERE:

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

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

C_t = 0.020 (0.0488) for all other buildings.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

1630A.3 Determination of Seismic Factors.

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

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

1630A.4 Combinations of Structural Systems.

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

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

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

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

1. The entire structure is designed using the lowest R of the lateral-force-resisting systems used, or
2. Two-stage analysis procedures may be used *providing the structure complies with the following:*
   - 2.1 The flexible upper portion shall be designed as a separate structure, supported laterally by the rigid lower portion, using the appropriate values of R and ρ.
   - 2.2 The rigid lower portion shall be designed as a separate structure using the appropriate values of R and ρ. The reactions from the upper portion shall be those determined from the analysis of the upper portion amplified by the ratio of the (R/ρ) of the upper portion over (R/ρ) of the lower portion. *This ratio shall not be less than one.*
   - *2.3 Where design of elements of the upper portion are governed by special seismic load combinations, the special loads shall be considered in the design of the lower portion.*
   - *2.4 The lower portion shall have a stiffness at least 10 times the upper portion.*
   - *2.5 The period of the entire structure shall not be greater than 1.1 times the period of the upper portion considered as a separate structure fixed at the base.*
   - *2.6 The detailing requirements required by the lateral system of the upper portion shall be used for structural components common to the structural system of lower portion.*
   - *2.7 If the separate models are used to design the upper and lower portions, the model boundary conditions of the upper portion shall be compatible with actual strength and stiffness of the supporting elements of the lower portion.*

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

WHERE:

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

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

The value of A_x need not exceed 3.0.

1630A.8 Overturning.

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

1630A.8.2 Elements supporting discontinuous systems.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

1630A.10 Story Drift Limitation.

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

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

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

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

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

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

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

SECTION 1631A — DYNAMIC ANALYSIS PROCEDURES

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

1631A.2 Ground Motion. *The dynamic analysis shall be based on the maximum probable ground motions prescribed in the Supplemental Ground-response Reports described in Section 1637A.2. The maximum probable* ground motion representation shall, as a minimum, be one having a 10-percent probability of being exceeded in 50 years, shall not be reduced by the quantity R and may be one of the following:

1. An elastic design response spectrum constructed in accordance with Figure 16A-3, using the values of C_a and C_v consistent with the specific site. The design acceleration ordinates shall be multiplied by the acceleration of gravity, 386.4 in./sec.^{2} (9.815 m/sec.^{2}). *This spectrum may be used for regular structures only.*

2. A site-specific elastic design response spectrum based on the geologic, tectonic, seismologic and soil characteristics associated with the specific site. The spectrum shall be developed for a damping ratio of 0.05, unless a different value is shown to be consistent with the anticipated structural behavior at the intensity of shaking established for the site. *The site-specific response spectra shall be used for irregular structures and for all structures located on Soil Profile Type S_F.*

3. Ground motion time histories developed for the specific site shall be representative of actual earthquake motions. Response spectra from time histories, either individually or in combination, shall approximate the site design spectrum conforming to Section 1631A.2, Item 2.

4. For structures on Soil Profile Type S_F, the following requirements shall apply when required by Section 1629A.8.4, Item 4:

   4.1 The ground motion representation shall be developed in accordance with Items 2 and 3.

   4.2 Possible amplification of building response due to the effects of soil-structure interaction and lengthening of building period caused by inelastic behavior shall be considered.

5. The vertical component of ground motion may be defined by scaling corresponding horizontal accelerations by a factor of two-thirds. Alternative factors may be used when substantiated by site-specific data. Where the Near Source Factor, N_a, is greater than 1.0, site-specific vertical response spectra shall be used in lieu of the factor of two-thirds.

6. *The "upper bound earthquake" ground motion is defined as the motion having a 10 percent probability of being exceeded in a 100-year period or maximum level of motion which may ever be expected at the building site within the known geological framework. Structures shall be designed to sustain the upperbound earthquake motion, including the PΔ effects, without forming a story collapse mechanism along any frameline. Every structure shall have sufficient ductility and strength to undergo the displacement caused by the upper bound earthquake motion without collapse. For irregular or unusual structures located in an area having large site-specific ground motion, criteria as determined by the project architect or structural engineer and approved by the enforcement agency will be required to demonstrate safety against collapse from the upper bound earthquake motion.*

1631A.3 Mathematical Model. A mathematical model of the physical structure shall represent the spatial distribution of the mass and stiffness of the structure to an extent that is adequate for the calculation of the significant features of its dynamic response. A three-dimensional model shall be used for the dynamic analysis of structures with highly irregular plan configurations such as those having a plan irregularity defined in Table 16A-M and having a rigid or semirigid diaphragm. The stiffness properties used in the analysis and general mathematical modeling shall be in accordance with Section 1630A.1.2. *The mathematical model of buildings with diaphragm discontinuities, as defined in Table 16A-M, Item 3, shall explicitly include the effect of diaphragm stiffness.*

1631A.4 Description of Analysis Procedures.

1631A.4.1 Response spectrum analysis. An elastic dynamic analysis of a structure utilizing the peak dynamic response of all modes having a significant contribution to total structural response. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve which correspond to the modal periods. Maximum modal contributions are combined in a statistical manner to obtain an approximate total structural response.

1631A.4.2 Time-history analysis. An analysis of the dynamic response of a structure at each increment of time when the base is subjected to a specific ground motion time history.

1631A.5 Response Spectrum Analysis.

1631A.5.1 Response spectrum representation and interpretation of results. The ground motion representation shall be in accordance with Section 1631A.2. The corresponding response parameters, including forces, moments and displacements, shall be denoted as Elastic Response Parameters. Elastic Response Parameters may be reduced in accordance with Section 1631A.5.4.

1631A.5.2 Number of modes. The requirement of Section 1631A.4.1 that all significant modes be included may be satisfied by demonstrating that for the modes considered, at least 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction.

1631A.5.3 Combining modes. The peak member forces, displacements, story forces, story shears and base reactions for each mode shall be combined by recognized methods. When three-dimensional models are used for analysis, modal interaction effects shall be considered when combining modal maxima.

1631A.5.4 Reduction of Elastic Response Parameters for design. Elastic Response Parameters may be reduced for purposes of design in accordance with the following items, with the limitation that in no case shall the Elastic Response Parameters be reduced such that the corresponding design base shear is less than the Elastic Response Base Shear divided by the value of R.

1. For all regular structures, ∗ ∗ ∗ Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than *100* percent of the base shear determined in accordance with Section 1630A.2.

2. *Not adopted by DSA/SS and OSHPD.*

3. *[For OSHPD 1 & 4 and DSA/SS] For irregular structures with vertical irregularity Types 1a, 2 or 5, as defined in Table 16A-L, or irregular structures with plan irregularity Type 1B, as defined in Table 16A-M, Elastic Response Parameters, may be re-*

duced such that the corresponding design base shear is not less than 125 percent of the base shear determined in accordance with Section 1630A.2.

EXCEPTION: The Elastic Response Parameters for structures with Vertical Irregularity Types 1a or 2, as defined in Table 16A-L, or plan irregularity Type 1b; as defined in Table 16A-M, may be reduced such that the corresponding design base shear is not less than 100 percent of the base shear determined in accordance with Section 1630A.2, if no interstory drift ratio under design lateral load is greater than 130 percent of the interstory drift ratio of the story immediately above. Torsional effects need not be considered in the calculation of story drifts for the purposes of this determination. The story drift ratio relationships for the top two stories of the structures are not required to be evaluated.

4. For all *other* structures, * * * Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than *100* percent of the base shear determined in accordance with Section 1630A.2.

The reduced design seismic forces shall be used for design in accordance with Section 1612A.

1631A.5.5 Directional effects. Directional effects for horizontal ground motion shall conform to the requirements of Section 1630A.1. The effects of vertical ground motions on horizontal cantilevers and prestressed elements shall be considered in accordance with Section 1630A.11. Alternately, vertical seismic response may be determined by dynamic response methods; in no case shall the response used for design be less than that obtained by the static method.

1631A.5.6 Torsion. The analysis shall account for torsional effects, including accidental torsional effects as prescribed in Section 1630A.7. Where three-dimensional models are used for analysis, effects of accidental torsion shall be accounted for by appropriate adjustments in the model such as adjustment of mass locations, or by equivalent static procedures such as provided in Section 1630A.6.

1631A.5.7 Dual systems. Where the lateral forces are resisted by a dual system as defined in Section 1629A.6.5, the combined system shall be capable of resisting the base shear determined in accordance with this section. The moment-resisting frame shall conform to Section 1629A.6.5, Item 2, and may be analyzed using either the procedures of Section 1630A.5 or those of Section 1631A.5.

1631A.6 Time-history Analysis.

1631A.6.1 Time history. Time-history analysis shall be performed with pairs of appropriate horizontal ground-motion time-history components that shall be selected and scaled from not less than three recorded events. Appropriate time histories shall have magnitudes, fault distances and source mechanisms that are consistent with those that control the design-basis earthquake (or maximum capable earthquake). Where three appropriate recorded ground-motion time-history pairs are not available, appropriate simulated ground-motion time-history pairs may be used to make up the total number required. For each pair of horizontal ground-motion components, the square root of the sum of the squares (SRSS) of the 5 percent-damped site-specific spectrum of the scaled horizontal components shall be constructed. The motions shall be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5 percent-damped spectrum of the design-basis earthquake for periods from $0.2T$ second to $1.5T$ seconds. Each pair of time histories shall be applied simultaneously to the model considering torsional effects.

The parameter of interest shall be calculated for each time-history analysis. If three time-history analyses are performed, then the maximum response of the parameter of interest shall be used for design. If seven or more time-history analyses are performed, then the average value of the response parameter of interest may be used for design.

1631A.6.2 Elastic time-history analysis. Elastic time history shall conform to Sections 1631A.1, 1631A.2, 1631A.3, 1631A.5.2, 1631A.5.4, 1631A.5.5, 1631A.5.6, 1631A.5.7 and 1631A.6.1. Response parameters from elastic time-history analysis shall be denoted as Elastic Response Parameters. All elements shall be designed using Strength Design. Elastic Response Parameters may be scaled in accordance with Section 1631A.5.4.

1631A.6.3 Nonlinear time-history analysis.

1631A.6.3.1 Nonlinear time history. Nonlinear time-history analysis shall meet the requirements of Section 1629A.10, and time histories shall be developed and results determined in accordance with the requirements of Section 1631A.6.1. Capacities and characteristics of nonlinear elements shall be modeled consistent with test data or substantiated analysis, considering the Importance Factor. The maximum inelastic response displacement shall not be reduced and shall comply with Section 1630A.10.

1631A.6.3.2 Design review. *[Not adopted by OSHPD]* When nonlinear time-history analysis is used to justify a structural design, a design review of the lateral-force-resisting system shall be performed by an independent engineering team, including persons licensed in the appropriate disciplines and experienced in seismic analysis methods. The lateral-force-resisting system design review shall include, but not be limited to, the following:

1. Reviewing the development of site-specific spectra and ground-motion time histories.

2. Reviewing the preliminary design of the lateral-force-resisting system.

3. Reviewing the final design of the lateral-force-resisting system and all supporting analyses.

The engineer of record shall submit with the plans and calculations a statement by all members of the engineering team doing the review stating that the above review has been performed.

SECTION 1632A — LATERAL FORCE ON ELEMENTS OF STRUCTURES, NONSTRUCTURAL COMPONENTS AND EQUIPMENT SUPPORTED BY STRUCTURES

1632A.1 General. Elements of structures and their attachments, permanent nonstructural components and their attachments, and the attachments for permanent equipment supported by a structure shall be designed to resist the total design seismic forces prescribed in Section 1632A.2.

Attachments shall include anchorages and required bracing. *Welded, bolted or other intermittent connections, such as inserts for anchorage of nonstructural components, shall not be allowed the one-third increase in allowable stresses permitted in Section 1612A.3.2.* Friction resulting from gravity loads shall not be considered to provide resistance to seismic forces.

When the structural failure of the lateral-force-resisting systems of nonrigid equipment would cause a life hazard, such systems shall be designed to resist the seismic forces prescribed in Section 1632A.2.

When permissible design strengths and other acceptance criteria are not contained in or referenced by this code, such criteria shall be obtained from approved national standards subject to the approval of the *enforcement agency.*

1632A.2 Design for Total Lateral Force. The total design lateral seismic force, F_p, shall be determined from the following formula:

$$F_p = 4.0\, C_a\, I_p\, W_p \qquad (32A\text{-}1)$$

Alternatively, F_p may be calculated using the following formula:

$$F_p = \frac{a_p\ C_a\ I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right) W_p \qquad (32A\text{-}2)$$

Except that:

F_p shall not be less than $0.7C_a I_p W_p$ and

need not be more than $4C_a I_p W_p$ (32A-3)

WHERE:

h_x is the element or component attachment elevation with respect to grade. h_x shall not be taken less than 0.0. *The value of h_x/h_r need not exceed 1.0.*

h_r is the structure roof elevation with respect to grade.

a_p is the in-structure Component Amplification Factor that varies from 1.0 to 2.5.

A value for a_p shall be selected from Table 16A-O. Alternatively, this factor may be determined based on the dynamic properties or empirical data of the component and the structure that supports it. The value shall not be taken less than 1.0. *For elevator-related equipment, see Part 7, Title 24, California Code of Regulations in addition to Table 16A-O.*

R_p is the Component Response Modification Factor that shall be taken from Table 16A-O, except that R_p for anchorages shall equal 1.5 for shallow expansion anchor bolts, shallow chemical anchors or shallow cast-in-place anchors. Shallow anchors are those with an embedment length-to-diameter ratio of less than 8. When anchorage is constructed of nonductile materials, or by use of adhesive, R_p shall equal 1.0.

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

Forces determined using Formula (32A-1) or (32A-2) shall be used to design members and connections that transfer these forces to the seismic-resisting systems. Members and connection design shall use the load combinations and factors specified in Section 1612A.2 or 1612A.3. The Reliability/Redundancy Factor, ρ, may be taken equal to 1.0. *A continuous load path of sufficient strength and stiffness between the component and the supporting structure shall be verified. Local elements of the supporting structure shall be verified for the component loads where they control the design of the elements or their connections. The component loads shall be those determined using Formula (32A-1) or (32A-2), except that modifications to R_p due to anchorage conditions need not be considered.*

For applicable forces and Component Response Modification Factors in connectors for exterior panels and diaphragms, refer to Sections 1633A.2.4, 1633A.2.8 and 1633A.2.9.

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

The design and detailing of anchorages or restraints for architectural components and mechanical and electrical systems shall accommodate the structure drifts as computed in accordance with Section 1633A.2.4 or determined by the wind forces given in Division III—Wind Design multiplied by 2.

1632A.3 Specifying Lateral Forces. Design specifications for equipment shall either specify the design lateral forces prescribed herein or reference these provisions.

1632A.4 Relative Motion of Equipment Attachments. For equipment in Categories 1 and 2 buildings as defined in Table 16A-K, the lateral-force design shall consider the effects of relative motion of the points of attachment to the structure, using the drift based upon Δ_M.

1632A.5 Alternative Designs. Where an approved national standard or approved physical test data provide a basis for the earthquake-resistant design of a particular type of equipment or other nonstructural component, such a standard or data may be accepted as a basis for design of the items with the following limitations:

1. These provisions shall provide minimum values for the design of the anchorage and the members and connections that transfer the forces to the seismic-resisting system.

2. The force, F_p, and the overturning moment used in the design of the nonstructural component shall not be less than 80 percent of the values that would be obtained using these provisions.

1632A.6 HVAC Ductwork, Plumbing/Piping and Conduit Systems. *All pipes, duct and conduit shall be braced to resist the forces prescribed in Section 1632A.2. Ductwork shall be constructed in accordance with provisions contained in Part 4, Title 24, California Mechanical Code. Pipes and their connections constructed of ductile materials (copper, ductile iron, steel, or aluminum with brazed or welded connections) shall have brace spacing not exceeding that specified in Section 1632A.5 or other standards approved by the enforcement agency. Pipes and their connections, constructed of nonductile materials (e.g., cast iron, no-hub pipe, and plastic), or with screwed connections shall have the brace spacing reduced to one-half of the spacing allowed for ductile material in accordance with Section 1632A.5 or other standards approved by the enforcement agency.*

Seismic restraints may be omitted for the following conditions, where flexible connections are provided between components and the associated ductwork, piping and conduit:

1. Fuel, medical gas, and vacuum piping less than 1 inch (25 mm) inside diameter.

2. All other piping less than 2.5 inches (64 mm) diameter, or

All piping suspended by individual hangers 12 inches (305 mm) or less in length from the top of the pipe to the bottom of the structural support for the hanger, or

All electrical conduit less than 2.5 inches (64 mm) trade size.

3. All rectangular air-handling ducts less than 6 square feet (0.56 m^2) in cross-sectional area, or

All round air-handling ducts less than 28 inches (711 mm) in diameter, or

All ducts suspended by hangers 12 inches (305 mm) or less in length from the top of the duct to the bottom of the structural support for the hanger, where the hangers are detailed to avoid bending of the hangers and their connections.

Where lateral restraints are omitted, the piping, ducts or conduit shall be installed such that lateral motion of the piping, duct or conduit will not cause damaging impact with other systems or structural members, or loss of vertical support.

1632A.6.1 *All trapeze assemblies supporting pipes, ducts and conduit shall be braced to resist the forces of Section 1632A.2, considering the total weight of the elements on the trapeze.*

Pipes, ducts and conduit supported by a trapeze where none of those elements would individually be braced need not be braced if connections to the pipe/conduit/ductwork or directional changes do not restrict the movement of the trapeze. If this flexibility is not provided, bracing will be required when the aggregate weight of the pipes and conduit exceed 10 pounds/feet (146 N/m). The weight shall be determined assuming all pipes and conduit are filled with water.

SECTION 1633A — DETAILED SYSTEMS DESIGN REQUIREMENTS

1633A.1 General. All structural framing systems shall comply with the requirements of Section 1629A. Only the elements of the designated seismic-force-resisting system shall be used to resist design forces. The individual components shall be designed to resist the prescribed design seismic forces acting on them. The components shall also comply with the specific requirements for the material contained in Chapters 19A through 23A. In addition, such framing systems and components shall comply with the detailed system design requirements contained in Section 1633A.

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

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

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

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

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

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

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

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

1633A.2 Structural Framing Systems.

1633A.2.1 General. Four types of general building framing systems defined in Section 1629A.6 are recognized in these provisions and shown in Table 16A-N. Each type is subdivided by the types of vertical elements used to resist lateral seismic forces. Special framing requirements are given in this section and in Chapters 19A through 23A.

1633A.2.2 Detailing for combinations of systems. For components common to different structural systems, the more restrictive detailing requirements shall be used.

1633A.2.3 Connections. Connections that resist design seismic forces shall be designed and detailed on the drawings.

1633A.2.4 Deformation compatibility. All structural framing elements and their connections, not required by design to be part of the lateral-force-resisting system, shall be designed and/or detailed to be adequate to maintain support of design dead plus live loads when subjected to the expected deformations caused by seismic forces. $P\Delta$ effects on such elements shall be considered. Expected deformations shall be determined as the greater of the Maximum Inelastic Response Displacement, Δ_M, considering $P\Delta$ effects determined in accordance with Section 1630A.9.2 or the deformation induced by a story drift of 0.0025 times the story height. When computing expected deformations, the stiffening effect of those elements not part of the lateral-force-resisting system shall be neglected.

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

For concrete and masonry elements that are part of the lateral-force-resisting system, the assumed flexural and shear stiffness properties shall not exceed one half of the gross section properties unless a rational cracked-section analysis is performed. Additional deformations that may result from foundation flexibility and diaphragm deflections shall be considered. For concrete elements not part of the lateral-force-resisting system, see Section 1921A.7.

1633A.2.4.1 Adjoining rigid elements. Moment-resisting frames and shear walls may be enclosed by or adjoined by more rigid elements, provided it can be shown that the participation or failure of the more rigid elements will not impair the vertical and lateral-load-resisting ability of the gravity load and lateral-force-resisting systems. The effects of adjoining rigid elements shall be considered when assessing whether a structure shall be designated regular or irregular in Section 1629A.5.1.

1633A.2.4.2 Exterior elements. Exterior nonbearing, nonshear wall panels or elements that are attached to or enclose the exterior shall be designed to resist the forces per Formula (32A-1) or (32A-2) and shall accommodate movements of the structure based on Δ_M and temperature changes. Such elements shall be supported by means of cast-in-place concrete or by mechanical connections and fasteners in accordance with the following provisions:

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

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

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

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

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

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

1633A.2.5 Ties and continuity. All parts of a structure shall be interconnected and the connections shall be capable of transmitting the seismic force induced by the parts being connected. As a minimum, any smaller portion of the building shall be tied to the remainder of the building with elements having at least a strength to resist 0.5 C_aI times the weight of the smaller portion.

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

1633A.2.6 Collector elements. Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces.

Collector elements, splices and their connections to resisting elements shall resist the forces determined in accordance with Formula (33A-1). In addition, collector elements, splices, and their connections to resisting elements shall have the design strength to resist the combined loads resulting from the special seismic load of Section 1612A.4.

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

The quantity E_M need not exceed the maximum force that can be transferred to the collector by the diaphragm and other elements of the lateral-force-resisting system. For Allowable Stress Design, the design strength may be determined using an allowable stress increase of 1.7 and a resistance factor, ϕ, of 1.0. This increase shall not be combined with the one-third stress increase permitted by Section 1612A.3, but may be combined with the duration of load increase permitted in Division III of Chapter 23A.

1633A.2.7 Concrete frames. Concrete frames required by design to be part of the lateral-force-resisting system shall conform to the following:

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

CA 2. *Not adopted by the State of California.*

1633A.2.8 Anchorage of concrete or masonry walls. Concrete or masonry walls shall be anchored to all floors and roofs that provide out-of-plane lateral support of the wall. The anchorage shall provide a positive direct connection between the wall and floor or roof construction capable of resisting the larger of the horizontal forces specified in this section and Sections 1611A.4 and 1632A. In addition, in Seismic Zones 3 and 4, diaphragm to wall anchorage using embedded straps shall have the straps attached to or hooked around the reinforcing steel or otherwise terminated to effectively transfer forces to the reinforcing steel. Requirements for developing anchorage forces in diaphragms are given in Section 1633A.2.9. Diaphragm deformation shall be considered in the design of the supported walls.

1633A.2.8.1 Out-of-plane wall anchorage to flexible diaphragms. This section shall apply in Seismic Zones 3 and 4 where flexible diaphragms, as defined in Section 1630A.6, provide lateral support for walls.

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

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

See Section 1611A.4 for minimum design forces in other seismic zones.

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

3. When pilasters are present in the wall, the anchorage force at the pilasters shall be calculated considering the additional load transferred from the wall panels to the pilasters. However, the minimum anchorage force at a floor or roof shall be that specified in Section 1633A.2.8.1, Item 1.

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

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

1633A.2.9 Diaphragms.

1. The deflection in the plane of the diaphragm shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection that will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads.

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

$$F_{px} = \frac{F_t + \sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \tag{33A-1}$$

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

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

3. Design seismic forces for flexible diaphragms providing lateral supports for walls or frames of masonry or concrete shall be determined using Formula (33A-1) based on the load determined in accordance with Section 1630A.2 using a R not exceeding 4.

4. Diaphragms supporting concrete or masonry walls shall have continuous ties or struts between diaphragm chords to distribute the anchorage forces specified in Section 1633A.2.8. Added chords of subdiaphragms may be used to form subdiaphragms to transmit the anchorage forces to the main continuous crossties. The maximum length-to-width ratio of the wood structural subdiaphragm shall be $2^1/_2$:1.

5. Where wood diaphragms are used to laterally support concrete or masonry walls, the anchorage shall conform to Section 1633A.2.8. In Seismic Zones ∗ ∗ ∗ 3 and 4, anchorage shall not be accomplished by use of toenails or nails subject to withdrawal, wood ledgers or framing shall not be used in cross-grain bending

or cross-grain tension, and the continuous ties required by Item 4 shall be in addition to the diaphragm sheathing.

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

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

Motion of the projecting wings in the same direction.

Motion of the projecting wings in opposing directions.

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

1633A.2.10 Framing below the base. The strength and stiffness of the framing between the base and the foundation shall not be less than that of the superstructure. The special detailing requirements of Chapters 19A and 22A, as appropriate, shall apply to columns supporting discontinuous lateral-force-resisting elements and to SMRF, ✳ ✳ ✳ EBF, STMF and MMRWF system elements below the base, which are required to transmit the forces resulting from lateral loads to the foundation.

1633A.2.11 Building separations. All structures shall be separated from adjoining structures. Separations shall allow for the displacement Δ_M. Adjacent buildings on the same property shall be separated by at least Δ_{MT} where

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

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

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

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

1633A.2.12 Foundations and superstructure-to-foundation connections. *The foundation shall be capable of transmitting the design base shear and the overturning forces from the structure into the supporting soil.*

The foundation and the connection of the superstructure elements to the foundation shall have the strength to resist, in addition to gravity loads, the lesser of the following seismic loads:

1. The strength of the superstructure elements.

2. The maximum forces that would occur in the fully yielded structural system.

3. Ω_o times the forces in the superstructure elements due to the seismic forces as prescribed in this chapter.

EXCEPTIONS: *1. Where structures are designed using $R \leq 2.2$ such as for inverted pendulum-type structures.*

2. When it can be demonstrated that inelastic deformation of the foundation and superstructure-to-foundation connection will not result in a weak story or cause collapse of the structure.

Where moment resistance is assumed at the base of the superstructure elements, the rotation and flexural deformation of the foundation as well as deformation of the superstructure-to-foundation connection shall be considered in the drift and deformation compatibility analyses.

1633A.2.13 Requirements for elevators. *In addition to all of the requirements contained in Part 7, Title 24, California Code of Regulations, the design of elevators in schools and hospitals shall meet the following requirements.*

1633A.2.13.1 *The design of guide rail support-bracket fastenings and the supporting structural framing shall be in accordance with Section 3030 (k), Part 7, Title 24, using the weight of the counterweight or maximum weight of the car plus not more than 40 percent of its rated load. The seismic forces shall be assumed to be distributed one third to the top guiding members and two thirds to the bottom guiding members of cars and counterweights, unless other substantiating data are provided. Minimum seismic forces shall be 0.5g acting in any horizontal direction, using allowable stress design.*

Retainer plates are required for both car and counterweight, designed in accordance with Section 3032 (c), Part 7, Title 24, California Code of Regulations. Retainer plates are required at the top and bottom of the car and counterweight, except where safety devices acceptable to the enforcement agency are provided which meet all requirements of the retainer plates, including full engagement of the machined portion of the rail. The design of the car and counterweight guide rails for seismic forces shall be based on the following requirements:

1. The lateral forces using allowable stress design shall be based on horizontal acceleration of 0.5g for all buildings.

2. W_p shall equal the weight of the counterweight or the maximum weight of the car plus not less than 40 percent of its rated load.

3. With the car or counterweight located in the most adverse position, the stress in the rail shall not exceed the limitations specified in these regulations, nor shall the deflection of the rail relative to its supports exceed the deflection listed below:

RAIL SIZE (weight per foot of length, pounds)	WIDTH OF MACHINED SURFACE (inches)	ALLOWABLE RAIL DEFLECTION (inches)
8	$1^1/_4$	0.20
11	$1^1/_2$	0.30
12	$1^3/_4$	0.40
15	$1^{31}/_{32}$	0.50
$18^1/_2$	$1^{31}/_{32}$	0.50
$22^1/_2$	2	0.50
30	$2^1/_4$	0.50

*For **SI:** 1 inch = 25 mm, 1 foot = 305 mm.*

NOTE: *Deflection limitations are given to maintain a consistent factor of safety against disengagement of retainer plates from the guide rails during an earthquake.*

4. Where guide rails are continuous over supports and rail joints are within 2 feet (610 mm) of their supporting brackets, a simple span may be assumed.

5. The use of spreader brackets is allowed.

6. Cab stabilizers and counterweight frames shall be designed to withstand a lateral load equal to 0.5g using allowable stress design.

SECTION 1634A — NONBUILDING STRUCTURES

1634A.1 General.

1634A.1.1 Scope. Nonbuilding structures include all self-supporting structures other than buildings that carry gravity loads and resist the effects of earthquakes. Nonbuilding structures shall be designed to provide the strength required to resist the displacements induced by the minimum lateral forces specified in this section. Design shall conform to the applicable provisions of other sections as modified by the provisions contained in Section 1634A.

1634A.1.2 Criteria. The minimum design seismic forces prescribed in this section are at a level that produce displacements in a fixed base, elastic model of the structure, comparable to those expected of the real structure when responding to the Design Basis Ground Motion. Reductions in these forces using the coefficient R is permitted where the design of nonbuilding structures provides sufficient strength and ductility, consistent with the provisions specified herein for buildings, to resist the effects of seismic ground motions as represented by these design forces.

When applicable, design strengths and other detailed design criteria shall be obtained from other sections or their referenced standards. The design of nonbuilding structures shall use the load combinations or factors specified in Section 1612A.2 or 1612A.3. For nonbuilding structures designed using Section 1634A.3, 1634A.4 or 1634A.5, the Reliability/Redundancy Factor, ρ, may be taken as 1.0.

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

1634A.1.3 Weight *W*. The weight, W, for nonbuilding structures shall include all dead loads as defined for buildings in Section 1630A.1.1. For purposes of calculating design seismic forces in nonbuilding structures, W shall also include all normal operating contents for items such as tanks, vessels, bins and piping.

1634A.1.4 Period. The fundamental period of the structure shall be determined by rational methods such as by using Method B in Section 1630A.2.2.

1634A.1.5 Drift. The drift limitations of Section 1630A.10 need not apply to nonbuilding structures. Drift limitations shall be established for structural or nonstructural elements whose failure would cause life hazards. $P\Delta$ effects shall be considered for structures whose calculated drifts exceed the values in Section 1630A.1.3.

1634A.1.6 Interaction effects. In Seismic Zones 3 and 4, structures that support flexible nonstructural elements whose combined weight exceeds 25 percent of the weight of the structure shall be designed considering interaction effects between the structure and the supported elements.

1634A.2 Lateral Force. Lateral-force procedures for nonbuilding structures with structural systems similar to buildings (those with structural systems which are listed in Table 16A-N) shall be selected in accordance with the provisions of Section 1629A.

1634A.3 Rigid Structures. Rigid structures (those with period T less than 0.06 second) and their anchorages shall be designed for the lateral force obtained from Formula (34A-1).

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

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

1634A.4 Tanks with Supported Bottoms. Flat bottom tanks or other tanks with supported bottoms, founded at or below grade, shall be designed to resist the seismic forces calculated using the procedures in Section 1634A for rigid structures considering the entire weight of the tank and its contents. Alternatively, such tanks may be designed using one of the two procedures described below:

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

2. A design basis prescribed for the particular type of tank by an approved national standard, provided that the seismic zones and occupancy categories shall be in conformance with the provisions of Sections 1629A.4 and 1629A.2, respectively.

1634A.5 Other Nonbuilding Structures. Nonbuilding structures that are not covered by Sections 1634A.3 and 1634A.4 shall be designed to resist design seismic forces not less than those determined in accordance with the provisions in Section 1630A with the following additions and exceptions:

1. The factors R and Ω_o shall be as set forth in Table 16A-P. The total design base shear determined in accordance with Section 1630A.2 shall not be less than the following:

$$V = 0.56C_a IW \qquad (34A\text{-}2)$$

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

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

2. The vertical distribution of the design seismic forces in structures covered by this section may be determined by using the provisions of Section 1630A.5 or by using the procedures of Section 1631A.

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

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

The seismic zones and occupancy categories shall be in conformance with the provisions of Sections 1629A.4 and 1629A.2, respectively.

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

SECTION 1635A — EARTHQUAKE-RECORDING INSTRUMENTATIONS *

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

Division V—SOIL PROFILE TYPES

SECTION 1636A — SITE CATEGORIZATION PROCEDURE

1636A.1 Scope. This division describes the procedure for determining Soil Profile Types S_A through S_F in accordance with Table 16A-J.

1636A.2 Definitions. Soil profile types are defined as follows:

S_A Hard rock with measured shear wave velocity, $\bar{v}_s > 5{,}000$ ft./sec. (1500 m/s).

S_B Rock with 2,500 ft./sec. $< \bar{v}_s \leq$ 5,000 ft./sec. (760 m/s $< \bar{v}_s \leq$ 1500 m/s).

S_C Very dense soil and soft rock with 1,200 ft./sec. $< \bar{v}_s \leq$ 2,500 ft./sec. (360 m/s $\bar{v}_s \leq$ 760 m/s) or with either $\bar{N} > 50$ or $\bar{s}_u \geq$ 2,000 psf (100 kPa).

S_D Stiff soil with 600 ft./sec. $\leq \bar{v}_s \leq$ 1,200 ft./sec. (180 m/s $\leq \bar{v}_s \leq$ 360 m/s) or with $15 \leq \bar{N} \leq 50$ or 1,000 psf $\leq \bar{s}_u \leq$ 2,000 psf (50 kPa $\leq \bar{s}_u \leq$ 100 kPa).

S_E A soil profile with $\bar{v}_s <$ 600 ft./sec. (180 m/s) or any profile with more than 10 ft. (3048 mm) of soft clay defined as soil with $PI > 20$, $w_{mc} \geq 40$ percent and $s_u <$ 500 psf (25 kPa).

S_F Soils requiring site-specific evaluation:

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

2. Peats and/or highly organic clays [$H >$ 10 ft. (3048 mm) of peat and/or highly organic clay where H = thickness of soil].

3. Very high plasticity clays [$H >$ 25 ft. (7620 mm) with $PI >$ 75].

4. Very thick soft/medium stiff clays [$H >$ 120 ft. (36 580 mm)].

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

The criteria set forth in the definition for Soil Profile Type S_F requiring site-specific evaluation shall be considered. If the site corresponds to this criteria, the site shall be classified as Soil Profile Type S_F and a site-specific evaluation shall be conducted.

1636A.2.1 $\bar{v}_s$, Average shear wave velocity. $\bar{v}_s$ shall be determined in accordance with the following formula:

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \qquad \text{(36A-1)}$$

WHERE:

d_i = thickness of Layer i in feet (m).

v_{si} = shear wave velocity in Layer i in ft./sec. (m/sec).

1636A.2.2 $\bar{N}$, average field standard penetration resistance and $\bar{N}_{CH}$, average standard penetration resistance for cohesionless soil layers. $\bar{N}$ and $\bar{N}_{CH}$ shall be determined in accordance with the following formula:

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \qquad \text{(36A-2)}$$

and

$$\bar{N}_{CH} = \frac{d_s}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \qquad \text{(36A-3)}$$

WHERE:

d_i = thickness of Layer i in feet (mm).

d_s = the total thickness of cohesionless soil layers in the top 100 feet (30 480 mm).

N_i = the standard penetration resistance of soil layer in accordance with approved nationally recognized standards.

1636A.2.3 $\bar{s}_u$, Average undrained shear strength. $\bar{s}_u$ shall be determined in accordance with the following formula:

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{n} \frac{d_i}{S_{ui}}} \qquad \text{(36A-4)}$$

WHERE:

d_c = the total thickness ($100 - d_s$) of cohesive soil layers in the top 100 feet (30 480 mm).

S_{ui} = the undrained shear strength in accordance with approved nationally recognized standards, not to exceed 5,000 psf (250 kPa).

1636A.2.4 Soft clay profile, S_E. The existence of a total thickness of soft clay greater than 10 feet (3048 mm) shall be investigated where a soft clay layer is defined by $s_u <$ 500 psf (24 kPa), $w_{mc} \geq 40$ percent and $PI > 20$. If these criteria are met, the site shall be classified as Soil Profile Type S_E.

1636A.2.5 Soil profiles S_C, S_D and S_E. Sites with Soil Profile Types S_C, S_D and S_E shall be classified by using one of the following three methods with $\bar{v}_s$, $\bar{N}$ and $\bar{s}_u$ computed in all cases as specified in Section 1636A.2.

1. $\bar{v}_s$ for the top 100 feet (30 480 mm) ($\bar{v}_s$ method).
2. $\bar{N}$ for the top 100 feet (30 480 mm) ($\bar{N}$ method).
3. $\bar{N}_{CH}$ for cohesionless soil layers ($PI < 20$) in the top 100 feet (30 480 mm) and average $\bar{s}_u$ for cohesive soil layers ($PI > 20$) in the top 100 feet (30 480 mm) ($\bar{s}_u$ method).

1636A.2.6 Rock profiles, S_A and S_B. The shear wave velocity for rock, Soil Profile Type S_B, shall be either measured on site or estimated by a geotechnical engineer, engineering geologist or seismologist for competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Soil Profile Type S_C.

The hard rock, Soil Profile Type S_A, category shall be supported by shear wave velocity measurement either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 100 feet (30 480 mm), surficial shear wave velocity measurements may be extrapolated to assess $\bar{v}_s$. The rock categories, Soil Profile Types S_A and

TABLE 16A-K—OCCUPANCY CATEGORY

OCCUPANCY CATEGORY	OCCUPANCY OR FUNCTIONS OF STRUCTURE	SEISMIC IMPORTANCE FACTOR, I	SEISMIC IMPORTANCE[1] FACTOR, I_p	WIND IMPORTANCE FACTOR, I_w
1. Essential facilities[2]	*Hospitals and other medical facilities as defined in Section 1250, Health and Safety Code* Group I, Division 1 Occupancies having surgery and emergency treatment areas Fire and police stations, *sheriffs offices, California Highway Patrol offices and California State Police Offices* *Municipal, county and state government disaster operation and communication centers deemed vital in emergencies* *For wind only, building areas where the primary occupancy is for assembly use for more than 300 people and portions of connecting or adjacent structures, the collapse of which would endanger the assembly area or restrict egress from it. (For earthquake, see Category 3.)* Garages and shelters for emergency vehicles and emergency aircraft Structures and shelters in emergency-preparedness centers Aviation control towers Structures and equipment in government communication centers and other facilities required for emergency response Standby power-generating equipment for Category 1 facilities Tanks or other structures containing housing or supporting water or other fire-suppression material or equipment required for the protection of Category 1, 2 or 3 structures	*1.50*	1.50	1.15
2. Hazardous facilities	Group H, Divisions 1, 2, 6 and 7 Occupancies and structures therein housing or supporting toxic or explosive chemicals or substances Nonbuilding structures housing, supporting or containing quantities of toxic or explosive substances that, if contained within a building, would cause that building to be classified as a Group H, Division 1, 2 or 7 Occupancy	1.25	1.50	1.15
3. Special occupancy structures[3]	Group A, Divisions 1, 2 and 2.1 Occupancies *For earthquake only, covered structures whose primary occupancy is public assembly-capacity greater than 300 persons. (For wind, see Category 1.)* Buildings housing Group E, Divisions 1 and 3 Occupancies with a capacity greater than 300 students Buildings housing Group B Occupancies used for college or adult education with a capacity greater than 500 students Group I, Divisions 1 and 2 Occupancies with 50 or more resident incapacitated patients, but not included in Category 1 Group I, Division 3 Occupancies All structures with an occupancy greater than 5,000 persons Structures and equipment in power-generating stations, and other public utility facilities not included in Category 1 or Category 2 above, and required for continued operation	*1.15*	*1.15*	1.00
4. Standard occupancy structures[3]	All structures housing occupancies or having functions not listed in Category 1, 2 or 3 and Group U Occupancy towers	1.00	1.00	1.00
5. Miscellaneous structures	Group U Occupancies except for towers	1.00	1.00	1.00

[1]The limitation of I_p for panel connections in Section 1633A.2.4 shall be 1.0 for the entire connector.
[2]Structural observation requirements are given in Section 1702A.
[3]For anchorage of machinery and equipment required for life-safety systems, the value of I_p shall be taken as 1.5.

TABLE 16A-L—VERTICAL STRUCTURAL IRREGULARITIES

IRREGULARITY TYPE AND DEFINITION	REFERENCE SECTION
1*a*. **Stiffness irregularity—soft story** A soft story is one in which the lateral stiffness is less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above.	1629A.8.4, Item 2
1b. ***Severe soft story*** *A severe soft story is one in which the lateral stiffness is less than 60 percent of that in the story above or less than 70 percent of the average stiffness of three stories above.*	*1629A.9.4*
2. **Weight (mass) irregularity** Mass irregularity shall be considered to exist where the effective mass of any story is more than 150 percent of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.	1629A.8.4, Item 2
3. **Vertical geometric irregularity** Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral-force-resisting system in any story is more than 130 percent of that in an adjacent story. One-story penthouses need not be considered.	1629A.8.4, Item 2
4. **In-plane discontinuity in vertical lateral-force-resisting element** An in-plane offset of the lateral-load-resisting elements greater than the length of *the* elements *below.*	1630A.8.2
5. **Discontinuity in capacity—weak story** A weak story is one in which the *ratio of the story strength to the story shear is less than 80 percent of that in the story above. The story strength is the strength of all seismic-resisting elements sharing the story shear for the direction under consideration. The load deformation characteristics of the elements shall be considered so that the strength is determined for compatible deformations.*	1629A.9.1

TABLE 16A-M—PLAN STRUCTURAL IRREGULARITIES

IRREGULARITY TYPE AND DEFINITION	REFERENCE SECTION
1*a*. **Torsional irregularity—to be considered when diaphragms are not flexible** Torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of the structure.	1633A.1, 1633A.2.9, Item 6
1b. ***Severe torsional irregularity—to be considered when diaphragms are not flexible*** *Severe torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts of the two ends of the structure.*	*1629A.9.5* *1631B.2.9, Item 6*
2. **Re-entrant corners** Plan configurations of a structure and its lateral-force-resisting system contain re-entrant corners, where both projections of the structure beyond a re-entrant corner are greater than 15 percent of the plan dimension of the structure in the given direction.	1633A.2.9, Items 6 and 7
3. **Diaphragm discontinuity** Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50 percent of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50 percent from one story to the next.	1633A.2.9, Item 6
4. **Out-of-plane offsets** Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements.	1630A.8.2; 1633A.2.9, Item 6; 2213A.9.1
5. **Nonparallel systems** The vertical lateral-load-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral-force-resisting system.	1633A.1

Division IV—SEISMIC PROVISIONS FOR STRUCTURAL STEEL BUILDINGS

Based on Seismic Provisions for Structural Steel Buildings of the American Institute of Steel Construction. (April 15, 1997)

SECTION 2210A — ADOPTION

Except for the modifications as set forth in Section 2211A 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, including Supplement No. 2 dated November 10, 2000.

SECTION 2211A — AMENDMENTS

The Seismic Provisions for Structural Steel Buildings, hereinafter referred to as AISC Seismic 97, shall include only Part I (LRFD) and Appendix S. Where other codes, standards or specifications are referred to in AISC Seismic 97, they are considered as supplemental standards and only considered guidelines subject to the approval of the enforcement agency.

***1.* Part I, Glossary. Add the following:**

Rapid Strength Deterioration: A mode of behavior characterized by a sudden loss of strength. In a cyclic test with constant or increasing deformation amplitude, a loss of strength of more than 50% of the strength attained in the previous excursion in the same loading direction.

***2. Part I, Glossary. Ordinary, Intermediate and Special Truss Moment Frame (OMF, IMF and STMF).* Not adopted by OSHPD.**

3. Part 1, Section 7.3c amend this section to read as follows:

For members and connections that are part of the Seismic Force Resisting System, discontinuities located within a plastic hinging zone as defined in Section 7.4a, created by errors or by fabrication or erection operations, such as tack welds, erection aids, air-arc gouging, and thermal cutting, shall be reparied as required by the Engineer of Record and approved by DSA.

***4.* Part I, Section 9.2 amend to read as the following:**

9.2. Beam-to-Column Joints and Connections

9.2a. The design of all beam-to-column joints and connections used in the Seismic Force Resisting System shall be based upon qualifying cyclic test results in accordance with Appendix S that demonstrate an *interstory drift angle of at least 0.04 radians and an* inelastic rotation of at least 0.03 radians.

a. Tests reported in research or documented tests performed for other projects that are demonstrated to reasonably match project conditions.

b. Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations and matching connection processes.

5.* Part I, Section 10. Intermediate Moment Frames (IMF) including Commentary Section C10.** ***Not adopted by OSHPD.

6.* Part I, Section 11. Ordinary Moment Frames (OMF) including Commentary Section C11.** ***Not adopted by OSHPD.

7.* Part I, Section 12. Special Truss Moment Frames (STMF) including Commentary Section C12.** ***Not adopted by OSHPD.

***8.* Part I, Section 15.4b. Add the following to the end of the paragraph:**

15.4b. Where reinforcement at the beam-to-column connection at the Link end precludes yielding of the beam over the reinforced length, the Link is permitted to be the beam segment from the end of the reinforcement to the brace connection. Where such Links are used and the Link length does not exceed 1.6 M_p/V_p, cyclic testing of the reinforced connection is not required if the design strength of the reinforced section and the connection equals or exceeds the required strength calculated based upon the strain-hardened Link as described in Section 15.6a. Full depth stiffeners as required in Section 15.3a. shall be placed at the Link-to-reinforcement interface. *Cyclic testing of the Link connection to the weak axis of a wide flange column is required for any length link.*

***9.* Part I, Section S3. Revise to read as follows:**

S3. DEFINITIONS

Inelastic Rotation. The permanent or plastic portion of the rotation angle between a beam and the column or between a Link and the column of the Test Specimen, measured in radians. The Inelastic Rotation shall be computed based upon an analysis of Test Specimen deformations. Sources of Inelastic Rotation include yielding of members and connectors, yielding of connection elements, and slip between members and connection elements. *For beam-to-column moment connections in Moment Frames, the inelastic rotation is represented by the plastic chord rotation angle calculated as the plastic deflection of the beam or girder, at the center of its span divided by the distance between the center of the beam span and the centerline of the panel zone of the beam column connection. For link-to-column connections in Eccentrically Braced Frames, inelastic rotation shall be computed based upon the assumption that inelastic action is concentrated at a single point located at the intersection of the centerline of the link with the face of the column.*

***10.* Part I, Section S5.2. Revise to read as follows:**

S5.2. Size of Members

1. The size of the beam or Link used in the Test Specimen shall be within the following limits:

   a. *At least one of the test beams or Links shall be 100% of the depth of the prototype beam or Link. For the remaining specimens,* the depth of the test beam or Link shall be no less than 90 percent of the depth of the Prototype beam or Link.

   b. *At least one of the test beams or Links shall be 100% of the weight per foot of the prototype beam or Link. For the remaining specimens,* the weight per foot of the test beam or Link shall be no less than 75 percent of the weight per foot of the Prototype beam or Link.

***11.* Part I, Section *S6.2*. *Add* the following:**

S6.3 Loading Sequence

Continue loading at increments of θ = 0.01 rad., with two cycles of loading at each step.

Or alternatively, the loading sequence may be the following:

1. 3 cycles of loading at $0.25\delta_y < \delta \leq 0.5\delta_y$
2. 3 cycles of loading at $0.6\delta_y < \delta \leq 0.8\delta_y$
3. 3 cycles of loading at $\delta = \delta_y$
4. 3 cycles of loading at $\delta = 2\delta_y$
5. 3 cycles of loading at $\delta = 3\delta_y$
6. 2 cycles of loading at $\delta = 4\delta_y$
7. After completion of the loading cycles at $4\delta_y$, testing shall be continued by applying cyclic loads to produce δ equal to $5\delta_y$, $6\delta_y$, $7\delta_y$, etc. Two cycles of loading shall be applied at each incremental value of deformation.

Other loading sequences are permitted to be used to qualify the Test Specimen when they are demonstrated to be of equivalent severity.

***12.* Part I - Section S10. Revise as follows:**

S10. ACCEPTANCE CRITERIA

For each connection used in the actual frame, at least three cyclic tests are required for each condition in which the Essential Variables, as listed in Section S5, remain within the required limits. All tests shall satisfy the criteria stipulated in Sections 9.2, or 15.4, as applicable. In order to satisfy *interstory drift angle and* Inelastic Rotation requirements, each Test Specimen shall sustain the required *interstory drift angle and inelastic* rotation for at least *two complete loading cycles without exhibiting rapid strength deterioration.*

ratio of rectangular tubes used for columns shall not exceed $110/\sqrt{F_y}$ (For **SI:** $0.65\sqrt{E/F_y}$), unless otherwise stiffened.

2213A.7.4 Continuity plates. When determining the need for girder tension flange continuity plates, the value of P_{bf} in Division III shall be taken as 1.8 $(bt_f)F_{yb}$.

2213A.7.5 Strength ratio. At any moment frame joint, the following relationships shall be satisfied:

$$\Sigma Z_c\ (F_{yc} - f_a)\ /\ \Sigma M_c > 1.0 \qquad (13A\text{-}3\text{-}1)$$

or

$$\Sigma Z_c\ (F_{yc} - f_a)\ /\ 1.25\Sigma M_{pz} > 1.0 \qquad (13A\text{-}3\text{-}2)$$

WHERE:

$f_a > 0$

M_c = the moment at column center line due to the development of plastic hinging in the beam accounting for overstrength and strain hardening.

M_{pz} = the sum of beam moments when panel zone shear strength reaches the value specified in Formula (13A-1) *determined with F_y increased to include the effects of strain hardening and overstrength.*

EXCEPTION: Columns meeting the compactness limitations for beams given in Section 2213A.7.3 *determined with F_y increased for overstrength and strain hardening* need not comply with this requirement provided they conform to one of the following conditions:

1. Columns with f_a less than 0.4 F_y for all load combinations other than loads specified in Section 2213A.5.1, and

1.1 Which are used in the top story of a multistory building with building period greater than 0.7 second

1.2 Where the sum of their resistance is less than 20 percent of the shear in a story, and is less than 33 percent of the shear on each of the column lines within that story. A column line is defined for the purpose of this exception as a single line of columns, or parallel lines of columns located within 10 percent of the plan dimension perpendicular to the line of columns; or

1.3 When the design for combined axial compression and bending is proportioned to satisfy Division III without the one-third permissible stress increase.

2. Columns in any story which have lateral shear strength 50 percent greater than that of the story above.

3. Columns which lateral shear strengths are not included in the design to resist code-required shears.

2213A.7.6 Trusses in SMRF. *Not adopted by DSA and OSHPD.*

2213A.7.7 Girder-column joint restraint.

2213A.7.7.1 Restrained joint. Where it can be shown that the columns of SMRF remain elastic, the flanges of the columns need be laterally supported only at the level of the girder top flange.

Columns may be assumed to remain elastic if one of the following conditions is satisfied:

1. The ratio in Formula (13A-3-1) or (13A-3-2) is greater than 1.25.

2. The flexural strength of the column is at least 1.25 times the moment that corresponds to the panel zone shear strength.

3. *The lesser of the* girder flexural strength, *including the effects of overstrength and strain hardening,* or *the* panel zone strength, *including the effects of overstrength and strain hardening,* will limit column stress $(f_a + f_{bx} + f_{by})$ to F_y of the column.

4. The column will remain elastic under gravity loads plus Ω_o times the design seismic forces.

Where the column cannot be shown to remain elastic, the column flanges shall be laterally supported at the levels of the girder top and bottom flanges. The column flange lateral support shall be capable of resisting a force equal to one percent of the girder flange capacity at allowable stresses and at a limiting displacement perpendicular to the frame of 0.2 inch (5.1 mm). Required bracing members may brace the column flanges directly or indirectly through the column web or the girder flanges.

2213A.7.7.2 Unrestrained joint. Columns without lateral support transverse to a joint shall conform to the requirements of Division III, with the column considered as pin ended and the length taken as the distance between lateral supports conforming with Section 2213A.7.7.1. The column stress, f_a, shall be determined from gravity loads plus the lesser of the following:

1. Ω_o times the design seismic forces.

2. The forces corresponding to either 125 percent of the girder flexural strength or *twice* the panel zone shear strength.

The stress, f_{by}, shall include the effects of the bracing force specified in Section 2213A.7.7.1 and $P\Delta$ effects.

l/r for such columns shall not exceed 60.

At truss frames the column shall be braced at each truss chord for a lateral force equal to one percent of the compression yield strength of the chord.

2213A.7.8 Beam bracing. Both flanges of beams shall be braced directly or indirectly. The beam bracing between column center lines shall not exceed $96r_y$. In addition, braces shall be placed at concentrated loads where a hinge may form.

2213A.7.9 Changes in beam flange area. Abrupt changes in beam flange area are not permitted within possible plastic hinge regions of special moment-resistant frames.

2213A.7.10 Moment frame drift calculations. Moment frame drift calculations shall include bending and shear contributions from the clear girder and column spans, column axial deformation and the rotation and distortion of the panel zone.

EXCEPTIONS: 1. Drift calculations may be based on column and girder center lines where either of the following conditions is met:

1.1 It can be demonstrated that the drift so computed for frames of similar configuration is typically within 15 percent of that determined above.

1.2 The column panel zone strength can develop $0.8\Sigma M_s$ of girders framing to the column flanges at the joint.

2. Column axial deformations may be neglected if they contribute less than 10 percent to the total drift.

2213A.8 Requirements for Braced Frames.

2213A.8.1 General. The provisions of this section apply to all braced frames except special concentrically braced frames designed in accordance with Section 2213A.9 or eccentrically braced frames (EBF) designed in accordance with Section 2213A.10. Those members which resist seismic forces totally or partially by shear or flexure shall be designed in accordance with Section 2213A.7 except Section 2213A.7.3.

2213A.8.2 Bracing members.

2213A.8.2.1 Slenderness. In Seismic Zones 3 and 4, the l/r ratio for bracing members shall not exceed $720/\sqrt{F_y}$ (For **SI:** $4.23\sqrt{E/F_y}$), except as permitted in Sections 2213A.8.5 and 2213A.8.6.

2213A.8.2.2 Stress reduction. The allowable stress, F_{as}, for bracing members resisting seismic forces in compression shall be determined from the following formula:

$$F_{as} = BF_a \qquad (13A\text{-}4)$$

WHERE:

B = the stress-reduction factor determined from the following formula:

$$B = 1/\{1 + [(Kl/r)/2C_c]\} \quad (13A\text{-}5)$$

F_a = the allowable axial compressive stress allowed in Division III.

EXCEPTION: Bracing members carrying gravity loads may be designed using the column strength requirement and load combinations of Section 2213A.5.1, Item 1.

2213A.8.2.3 Lateral-force distribution. The seismic lateral force along any line of bracing shall be distributed to the various members so that neither the sum of the horizontal components of the forces in members acting in tension nor the sum of the horizontal components of forces in members acting in compression exceed 70 percent of the total force.

EXCEPTION: Where compression bracing acting alone has the strength, neglecting the stress-reduction factor B, to resist Ω_o times the design seismic force such distribution is not required.

A line of bracing is defined, for the purpose of this provision, as a single line or parallel lines within 10 percent of the dimension of the structure perpendicular to the line of bracing.

2213A.8.2.4 Built-up members. The l/r of individual parts of built-up bracing members between stitches, when computed about a line perpendicular to the axis through the parts, shall not be greater than 75 percent of the l/r of the member as a whole.

2213A.8.2.5 Compression elements in braces. The width-thickness ratio of stiffened and unstiffened compression elements used in braces shall be as shown in Division III, Table B5.1, for compact sections.

The width-thickness ratio of angle sections shall be limited to $52/\sqrt{f_y}$ (For **SI:** $0.31\sqrt{E/f_y}$). Circular sections shall have outside diameter-wall thickness ratio not exceeding $1{,}300/F_y$ (For **SI:** $7.63\ E/f_y$). Rectangular tubes shall have outside width-thickness ratio not exceeding $110/\sqrt{F_y}$ (For **SI:** $0.65\sqrt{E/F_y}$).

EXCEPTION: Compression elements stiffened to resist local buckling.

2213A.8.3 Bracing connection.

2213A.8.3.1 Forces. Bracing connections shall have the strength to resist the least of the following:

1. The strength of the bracing in axial tension, P_{st}.

2. Ω_o times the force in the brace due to the design seismic forces, in combination with gravity loads.

3. The maximum force that can be transferred to the brace by the system.

Bracing connections shall, as a minimum, satisfy the load combinations required by Section 1612A.2 at load and resistance factor design limits or Section 1612A.3 at allowable stress design limits with stress increases allowed by Section 1612A.3.2. These combinations shall include the provisions for Sections 2213A.8.2.2 and 2213A.8.4.1.

Beam-to-column connections for beams that are part of the bracing system shall have the capacity to transfer the force determined above. Where eccentricities in the frame geometry or connection load path exist, the affected members and connections shall have the strength to resist all secondary forces resulting from the eccentricities in combination with all primary forces using the lesser of the forces determined above.

2213A.8.3.2 Net area. In bolted brace connections, the ratio of effective net section area to gross section area shall satisfy the formula:

$$\frac{A_e}{A_g} \geq \frac{1.2\,\alpha F^*}{F_u} \quad (13A\text{-}6)$$

WHERE:

A_e = effective net area as defined in Division III.

F_u = minimum tensile strength.

F^* = stress in brace as determined in Section 2213A.8.3.1.

α = fraction of the member force from Section 2213A.8.3.1 that is transferred across a particular net section.

2213A.8.4 Bracing configuration.

2213A.8.4.1 Chevron bracing. Chevron bracing shall conform with the following:

1. Bracing members shall be designed for 1.5 times the otherwise prescribed seismic forces, in addition to the requirements of Section 2213A.8.2.2.

2. The beam intersected by chevron braces shall be continuous between columns.

3. Where chevron braces intersect a beam from below, i.e., inverted V brace, the beam shall be capable of supporting all tributary gravity loads presuming the bracing not to exist.

EXCEPTION: This limitation need not apply to penthouses, one-story buildings or the top story of buildings.

2213A.8.4.2 K bracing. K bracing is prohibited except as permitted in Section 2213A.8.5.

2213A.8.4.3 Nonconcentric bracing. Nonconcentric bracing shall conform with the following:

1. Any member intersected by the brace shall be continuous through the connection.

2. When the eccentricity of the brace is greater than the depth of the intersected member at the eccentric location, the affected member shall have the strength to resist the forces prescribed in Section 2213A.8.3.1, including the effects of all secondary forces resulting from the eccentricities.

2213A.8.5 One- and two-story buildings. *[Not adopted by OSHPD]* Braced frames not meeting the requirements of Sections 2213A.8.2 and 2213A.8.4 may be used in buildings not over two stories in height and in roof structures as defined in Chapter 15 if the braces have the strength to resist Ω_o times the design seismic forces. CAC

2213A.8.6 Nonbuilding structures. Nonbuilding structures with R values defined by Table 16A-P need comply only with the provisions of Section 2213A.8.3.

2213A.9 Requirements for Special Concentrically Braced Frames.

2213A.9.1 General. The provisions of this section apply to special concentrically braced frame structures as defined in Section

(Text continues on page 2-272.11.)

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 "<u>*Section 1916A.4.2.*</u>" with "<u>*Section l916A.7.1.*</u>"

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

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

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

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

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