REVISION RECORD FOR THE STATE OF CALIFORNIA

ERRATA

October 1, 2002

Title 24, Part 2, California Building Code

This revision record contains all the errata affecting the above-entitled portion of the California Code of Regulations.

By starting with a full loose-leaf copy of the 2001 *California Building Code* and substituting the revised pages (buff) listed below, the user will have a complete 2001 *California Building Code* in correct numerical sequence. 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suggested that the section number as well as the page number be checked when inserting this material and removing the superseded material.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 since the section numbers must run consecutively.

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

VOLUME 2

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Volume 2

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

Chapter 16
STRUCTURAL DESIGN REQUIREMENTS
Division I—GENERAL DESIGN REQUIREMENTS

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SECTION 1601 — SCOPE

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

SECTION 1602 — DEFINITIONS

The following terms are defined for use in this code:

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

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

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

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

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

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

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

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

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

SECTION 1603 — NOTATIONS

D = dead load.

E = earthquake load set forth in Section 1630.1.

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

F = load due to fluids.

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

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

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

P = ponding load.

S = snow load.

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

W = load due to wind pressure.

SECTION 1604 — STANDARDS

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

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

SECTION 1605 — DESIGN

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

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

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

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

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

1605.2.2 Stability against overturning. Every structure shall be designed to resist the overturning effects caused by the lateral forces specified in this chapter. See Section 1611.6 for retaining walls, Section 1615 for wind and Section 1626 for seismic.

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

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

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

SECTION 1606 — DEAD LOADS

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

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

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

SECTION 1607 — LIVE LOADS

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

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

1607.3 Floor Live Loads.

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

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

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

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

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

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

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

1607.4 Roof Live Loads.

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

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

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

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

1. Snow load in accordance with Section 1614.

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

3. A concentrated gravity load, L_r, of 2,000 pounds (8.9 kN) placed on any span supporting a tributary area greater than 200 square feet (18.58 m^2) to create maximum stresses in the member,

Structures that are designed in accordance with this section shall comply with all other applicable requirements of these provisions.

1631.2 Ground Motion. The 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 16-3, 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}).

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.

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 1631.2, Item 2.

4. For structures on Soil Profile Type S_F, the following requirements shall apply when required by Section 1629.8.4, 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.

1631.3 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 16-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 1630.1.2.

1631.4 Description of Analysis Procedures.

1631.4.1 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.

1631.4.2 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.

1631.5 Response Spectrum Analysis.

1631.5.1 Response spectrum representation and interpretation of results. The ground motion representation shall be in accordance with Section 1631.2. 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 1631.5.4.

1631.5.2 Number of modes. The requirement of Section 1631.4.1 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.

1631.5.3 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.

1631.5.4 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 where the ground motion representation complies with Section 1631.2, Item 1, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 90 percent of the base shear determined in accordance with Section 1630.2.

2. For all regular structures where the ground motion representation complies with Section 1631.2, Item 2, Elastic Response Parameters may be reduced such that the corresponding design base shear is not less than 80 percent of the base shear determined in accordance with Section 1630.2.

3. For all irregular structures, regardless of the ground motion representation, 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 1630.2.

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

1631.5.5 Directional effects. Directional effects for horizontal ground motion shall conform to the requirements of Section 1630.1. The effects of vertical ground motions on horizontal cantilevers and prestressed elements shall be considered in accordance with Section 1630.11. 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.

1631.5.6 Torsion. The analysis shall account for torsional effects, including accidental torsional effects as prescribed in Section 1630.7. 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 1630.6.

1631.5.7 Dual systems. Where the lateral forces are resisted by a dual system as defined in Section 1629.6.5, 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 1629.6.5, Item 2, and may be analyzed using either the procedures of Section 1630.5 or those of Section 1631.5.

1631.6 Time-history Analysis.

1631.6.1 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.

1631.6.2 Elastic time-history analysis. Elastic time history shall conform to Sections 1631.1, 1631.2, 1631.3, 1631.5.2, 1631.5.4, 1631.5.5, 1631.5.6, 1631.5.7 and 1631.6.1. 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 1631.5.4.

1631.6.3 Nonlinear time-history analysis.

1631.6.3.1 Nonlinear time history. Nonlinear time-history analysis shall meet the requirements of Section 1629.10, and time histories shall be developed and results determined in accordance with the requirements of Section 1631.6.1. 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 1630.10.

1631.6.3.2 Design review. 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 1632 — LATERAL FORCE ON ELEMENTS OF STRUCTURES, NONSTRUCTURAL COMPONENTS AND EQUIPMENT SUPPORTED BY STRUCTURES

1632.1 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 1632.2. Attachments for floor- or roof-mounted equipment weighing less than 400 pounds (181 kg), and furniture need not be designed. *[For OSHPD 2] The attachments of the following items need not be detailed on the plans:*

1. Equipment weighing less than 400 pounds (181 kg) supported directly on the floor or roof.

2. Furniture.

3. Temporary or movable equipment.

4. Equipment weighing less than 20 pounds (9 kg) supported by vibration isolators.

5. Equipment weighting less than 20 pounds (9 kg) suspended from a roof or floor, or hung from a wall.

Attachments shall include anchorages and required bracing. *[For OSHPD 2] 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 1612.3.2.* 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 1632.2.

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 building official. *[For OSHPD 2] When acceptable national standards are not available, the standard shall be as established by the enforcing agency.*

1632.2 Design for Total Lateral Force. The total design lateral seismic force, F_p, shall be determined from the following formula:

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

Alternatively, F_p may be calculated using the following formula:

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

Except that:

$$F_p \text{ shall not be less than } 0.7 C_a I_p W_p \text{ and need not be more than } 4 C_a I_p W_p \qquad (32\text{-}3)$$

WHERE:

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

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 16-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.

R_p is the Component Response Modification Factor that shall be taken from Table 16-O, except that R_p for anchorages shall equal 1.5 for shallow expansion anchor bolts, shallow chemical

Method 1 or Method 2. In addition, design of the overall structure and its primary load-resisting system shall conform to Section 1605A.

The base overturning moment *about the bottom of the foundation* for the entire structure, or for any one of its individual primary lateral-*force* resisting elements, shall not exceed two thirds of the dead-load-resisting moment. *The overturning moment shall be calculated using the combined effects of uplift and lateral loads.* For an entire structure with a height-to-width ratio of 0.5 or less in the wind direction and a maximum height of 60 feet (18 290 mm), the combination of the effects of uplift and overturning may be reduced by one third. The weight of earth superimposed over footings may be used to calculate the dead-load-resisting moment.

The individual primary lateral-force-resisting elements, including the foundation, and the interface to the foundation, shall be designed to include the combined effects of uplift and lateral loads. No reduction for height-to-width ratios of 0.5 or less is allowed.

The dead-load-resisting moment shall be based on an estimate of the actual dead load, not necessarily the design dead load which may include excess dead load, for both the stability and capacity calculations.

1621A.2 Method 1 (Normal Force Method). Method 1 shall be used for the design of gabled rigid frames and may be used for any structure. In the Normal Force Method, the wind pressures shall be assumed to act simultaneously normal to all exterior surfaces. For pressures on roofs and leeward walls, C_e shall be evaluated at the mean roof height.

1621A.3 Method 2 (Projected Area Method). Method 2 may be used for any structure less than 200 feet (60 960 mm) in height except those using gabled rigid frames. This method may be used in stability determinations for any structure less than 200 feet (60 960 mm) high. In the Projected Area Method, horizontal pressures shall be assumed to act upon the full vertical projected area of the structure, and the vertical pressures shall be assumed to act simultaneously upon the full horizontal projected area.

SECTION 1622A — ELEMENTS AND COMPONENTS OF STRUCTURES

Design wind pressures for each element or component of a structure shall be determined from Formula (20A-1) and C_q values from Table 16A-H, and shall be applied perpendicular to the surface. For outward acting forces the value of C_e shall be obtained from Table 16A-G based on the mean roof height and applied for the entire height of the structure. Each element or component shall be designed for the more severe of the following loadings:

1. The pressures determined using C_q values for elements and components acting over the entire tributary area of the element.

2. The pressures determined using C_q values for local areas at discontinuities such as corners, ridges and eaves. These local pressures shall be applied over a distance from a discontinuity of 10 feet (3048 mm) or 0.1 times the least width of the structure, whichever is less, *as follows:*

2.1 At Walls . . . . Corners

2.2 At Roofs . . . . Corners, eaves, rakes, ridges and the width of any overhangs

The wind pressures from Sections 1621A and 1622A need not be combined. *A complete load path for resistance to wind uplift forces shall be provided.*

SECTION 1623A — OPEN-FRAME TOWERS

Radio towers and other towers of trussed construction shall be designed and constructed to withstand wind pressures specified in this section, multiplied by the shape factors set forth in Table 16A-H.

SECTION 1624A — MISCELLANEOUS STRUCTURES

Greenhouses, lath houses, agricultural buildings or fences 12 feet (3658 mm) or less in height shall be designed in accordance with Chapter 16A, Division III. However, three fourths of q_s, but not less than 10 psf (0.48 kN/m^2), may be substituted for q_s in Formula (20A-1). Pressures on local areas at discontinuities need not be considered.

SECTION 1625A — OCCUPANCY CATEGORIES

For the purpose of wind-resistant design, each structure shall be placed in one of the occupancy categories listed in Table 16A-K. Table 16A-K lists importance factors, I_w, for each category.

Division IV—EARTHQUAKE DESIGN

SECTION 1626A — GENERAL

1626A.1 Purpose. *[Not adopted by OSHPD]* The purpose of the earthquake provisions herein is primarily to safeguard against major structural failures and loss of life.

1626A.2 Minimum Seismic Design. *Every building or structure and every portion thereof, including the nonstructural components, shall be designed and constructed to resist stresses and limit deflections calculated on the basis of dynamic analysis or equivalent static lateral force analysis as provided in this section. Stresses shall be calculated as the effect of a force applied horizontally at each floor or roof level above the base.*

Surface friction due to gravity loads may be considered effective in resisting seismic forces as provided in Chapter 18A for foundation and bearing soil contact.

1626A.3 Seismic and Wind Design. When the code-prescribed wind design produces greater effects, the wind design shall govern, but detailing requirements and limitations prescribed in this section and referenced sections shall be followed.

1626A.4 [For OSHPD 1 & 4] Configuration. *When the design of a structure, due to the unusual configuration of the structure or parts of the structure, does not provide at least the same safety against earthquake damage as provided by the applicable portions of this section when applied in the design of a similar structure of customary configuration, framing and assembly of materials, the enforcement agency shall withhold its approval.*

SECTION 1627A — DEFINITIONS

For the purposes of this division, certain terms are defined as follows:

ADDITION *means any work which increases the floor or roof area or the volume of enclosed space of an existing building and is dependent on the structural elements of that facility for vertical or lateral support.*

ALTERATION *means any change in an existing building which does not increase and may decrease the floor or roof area or the volume of enclosed space.*

APPROVED EXISTING BUILDING [for OSHPD 1 and 4]. *Any building originally constructed in compliance with the requirements of the 1973 or subsequent edition of the California Building Code.*

ASSOCIATED STRUCTURAL ALTERATIONS *means any change affecting existing structural elements or requiring new structural elements for vertical or lateral support of an otherwise nonstructural alteration.*

BASE is the level at which the earthquake motions are considered to be imparted to the structure or the level at which the structure as a dynamic vibrator is supported. *This level does not necessarily coincide with the ground level.*

BASE SHEAR, V, is the total design lateral force or shear at the base of a structure.

BEARING WALL SYSTEM is a structural system without a complete vertical load-carrying space frame. See Section 1629A.6.2.

BOUNDARY ELEMENT is an element at edges of openings or at perimeters of shear walls or diaphragms.

BRACED FRAME is an essentially vertical truss system of the concentric or eccentric type that is provided to resist lateral forces.

BUILDING FRAME SYSTEM is an essentially complete space frame that provides support for gravity loads. See Section 1629A.6.3.

CANTILEVERED COLUMN ELEMENT is a column element in a lateral-force-resisting system that cantilevers from a fixed base and has minimal moment capacity at the top, with lateral forces applied essentially at the top.

COLLECTOR is a member or element provided to transfer lateral forces from a portion of a structure to vertical elements of the lateral-force-resisting system.

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

COMPONENT, EQUIPMENT, is a mechanical or electrical component or element that is part of a mechanical and/or electrical system.

COMPONENT, FLEXIBLE, is a component, including its attachments, having a fundamental period greater than 0.06 second.

COMPONENT, RIGID, is a component, including its attachments, having a fundamental period less than or equal to 0.06 second.

CONCENTRICALLY BRACED FRAME is a braced frame in which the members are subjected primarily to axial forces.

DESIGN BASIS GROUND MOTION is that ground motion that has a 10 percent chance of being exceeded in 50 years as determined by a site-specific hazard analysis or may be determined from a hazard map. A suite of ground motion time histories with dynamic properties representative of the site characteristics shall be used to represent this ground motion. The dynamic effects of the Design Basis Ground Motion may be represented by the Design Response Spectrum. See Section 1631A.2.

DESIGN RESPONSE SPECTRUM is an elastic response spectrum for 5 percent equivalent viscous damping used to represent the dynamic effects of the Design Basis Ground Motion for the design of structures in accordance with Sections 1630A and 1631A. This response spectrum may be either a site-specific spectrum based on geologic, tectonic, seismological and soil characteristics associated with a specific site or may be a spectrum constructed in accordance with the spectral shape in Figure 16A-3 using the site-specific values of C_a and C_v and multiplied by the acceleration of gravity, 386.4 in./sec.^{2} (9.815 m/sec.^{2}). See Section 1631A.2.

DESIGN SEISMIC FORCE is the minimum total strength design base shear, factored and distributed in accordance with Section 1630A.

DIAPHRAGM is a horizontal or nearly horizontal system acting to transmit lateral forces to the vertical-resisting elements. The term "diaphragm" includes horizontal bracing systems.

DIAPHRAGM or SHEAR WALL CHORD is the boundary element of a diaphragm or shear wall that is assumed to take axial stresses analogous to the flanges of a beam.

DIAPHRAGM STRUT (drag strut, tie, collector) is the element of a diaphragm parallel to the applied load that collects and transfers diaphragm shear to the vertical-resisting elements or distributes loads within the diaphragm. Such members may take axial tension or compression.

DRIFT. See "story drift."

DUAL SYSTEM is a combination of moment-resisting frames and shear walls or braced frames designed in accordance with the criteria of Section 1629A.6.5.

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

$$F_p = 4.0\, C_a\, I_p\, W_p \tag{32A-1}$$

Alternatively, F_p may be calculated using the following formula:

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

Except that:

$$F_p \text{ shall not be less than } 0.7C_aI_pW_p \text{ and need not be more than } 4C_aI_pW_p \tag{32A-3}$$

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 piping less than 1 inch (25 mm) inside diameter.

2. All other piping less than 2.5 inches (64 mm) diameter, except medical gas including vacuum piping, 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 or duct 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.

S_B, shall not be used if there is more than 10 feet (3048 mm) of soil between the rock surface and the bottom of the spread footing or mat foundation.

The definitions presented herein shall apply to the upper 100 feet (30 480 mm) of the site profile. Profiles containing distinctly different soil layers shall be subdivided into those layers designated by a number from 1 to n at the bottom, where there are a total of n distinct layers in the upper 100 feet (30 480 mm). The symbol i then refers to any one of the layers between 1 and n.

SECTION 1637A — SITE DATA FOR HOSPITALS AND STATE-OWNED OR STATE-LEASED ESSENTIAL SERVICES BUILDINGS

1637A.1 Engineering geologic reports.

1637A.1.1 *Geologic and earthquake engineering reports shall be required for all proposed construction.*

> ***EXCEPTIONS:*** *1. Reports are not required for one-story, wood-frame and light-steel-frame buildings of Type II or Type V construction and 4,000 square feet (371m^2) or less in floor area; nonstructural, associated structural or nonrequired structural alterations and incidental structural additions or alterations, and structural repairs for other than earthquake damage.*
>
> *2. A previous report for a specific site may be resubmitted, provided that a reevaluation is made and the report is found to be currently appropriate.*

1637A.1.2 *The purpose of the engineering geologic report shall be to identify geologic and seismic conditions that may require project mitigations. The reports shall contain data which provide an assessment of the nature of the site and potential for earthquake damage based on appropriate investigations of the regional and site geology, project foundation conditions and the potential seismic shaking at the site. The report shall be prepared by a California-certified engineering geologist in consultation with a California-registered geotechnical engineer. The engineering geologic report shall not contain design criteria, but shall contain basic data to be used for a preliminary earthquake engineering evaluation of the project.*

The preparation of the engineering geologic report shall be done in conformance with the most recent Division of Mines and Geology (DMG) Notes 44 and 42; Guidelines for Preparing Engineering Geologic Reports, and Guidelines to Geologic/Seismic Reports, respectively. Upper-bound earthquakes, proposed in the Engineering Geologic Report, must be fully supported by satisfactory data and analysis. In addition, the most recent version of DMG Special Publication 42, Fault Rupture Hazard Zones in California, shall be considered for project sites proposed within an Alquist-Priolo special studies zone.

The report shall include, but shall not be limited to, the following:

1. Geologic investigation.

2. Evaluation of the known active and potentially active faults, both regional and local, including estimates of their upper-bound earthquakes and estimates of the peak ground accelerations at the site resulting from these earthquakes.

3. Evaluation of slope stability at or near the site, and the liquefaction and settlement potential of the earth materials in the foundation.

1637A.1.3 *The engineering geologic report shall be submitted to the enforcement agency for review and approval. The review shall determine whether potential geologic problems and hazards are adequately identified and described in order to provide a timely completion of the subsequent geotechnical report, described in Section 1637A.2.1. The enforcement agency, with consultation of its advisors, may require additional information, analysis and/or clarification of potential geologic problems affecting the proposed building site before approval is given. The results of the approved engineering geologic report shall be used as a basis for further investigations for the geotechnical report. Approval of the engineering geologic report by the enforcement agency shall be required prior to the submission of the geotechnical report.*

1637A.2 Geotechnical and Supplemental Ground-response Reports.

1637A.2.1 Geotechnical report.

1637A.2.1.1 *The geotechnical report shall provide completed evaluations of the foundation conditions of the site and the potential geologic/seismic hazards affecting the site. The geotechnical report shall include, but shall not be limited to, site-specific evaluations of design criteria related to the nature and extent of foundation materials, groundwater conditions, liquefaction potential, settlement potential and slope stability. The report shall contain the results of the analyses of problem areas identified in the engineering geologic report. The geotechnical report shall incorporate estimates of the characteristics of site ground motion provided in the engineering geologic report. The estimates of ground motion shall not be structural design criteria, but shall be provided to characterize the seismic environment of the site, with consideration of the upper-bound earthquakes reported in the engineering geologic report. The ground-motion estimates shall include, but shall not be limited to, peak ground motions and predominant period. The estimates should be derived by accepted methods of current seismological practice and fully documented in the geologic report.*

The geotechnical report shall be prepared by a geotechnical engineer registered in the state of California with the advice of the certified engineering geologist and other technical experts, as necessary. The approved engineering geologic report shall be submitted with or as part of the geotechnical report.

1637A.2.1.2 *The geotechnical report shall be submitted to the enforcement agency for review and approval. The review shall determine whether potential geologic hazards and foundation problems have been adequately evaluated. The enforcement agency, with the consultation of its advisors, may require additional information, analysis or clarification of potential geotechnical issues affecting the proposed building site before approving the geotechnical report.*

Approval of the geotechnical report by the enforcement agency shall be required prior to the approval of the supplemental ground-response report, if required, as described in Section 1637A.2.2. The results of the geotechnical report shall be used as a guide for further investigations for the supplemental ground-response report.

1637A.2.2 Supplemental ground-response report. *A supplemental ground-response report may be required, containing a ground-motion element and an advanced geotechnical element.*

1637A.2.2.1 *The ground-motion element shall be prepared when required by the approved geotechnical report, or when required for dynamic analysis procedures described under Section 1631A.2. The ground-motion element shall be prepared by a registered civil engineer or geophysicist (depending on the scope of the element), or engineering geologist licensed in the state of California, and having professional specialization in earthquake analyses. The ground-motion element shall present a detailed characterization of earthquake ground motions for the site, which incorporates data given in the geotechnical report. The level of* *ground motion considered by the ground-motion element shall be as described in Section 1631A.2. The characterization of ground motion in the ground-motion element shall be given, according to the requirements of the analysis, in terms of:*

1. Peak acceleration, bracketed duration and predominant period.

2. Elastic structural response spectra.

3. Time-history plot of predicted ground motion at the site.

4. Other analyses in conformance with accepted engineering and seismological practice.

1637A.2.2.2 *The advanced geotechnical element shall contain the results of dynamic geotechnical analyses specified by the approved geotechnical report.*

The supplemental ground-response report shall be submitted to the enforcement agency for review and approval. The review shall determine whether the ground-motion response evaluations of the site are adequately represented. The enforcement agency, under consultation with its advisors, may require additional information, analysis or clarification of potential ground-response issues reported in the supplemental ground-response report for the proposed building site.

SECTION 1638A [FOR OSHPD 1 & 4] — ADDITIONS, ALTERATIONS, REPAIRS AND SEISMIC RETROFIT TO EXISTING BUILDINGS OR STRUCTURES

Existing hospital buildings (as defined in Section 7-111, Part 1, Title 24, Building Standards Administrative Code).

NOTE: *Alterations to lateral shear force-resisting capacity and story lateral shear forces shall be considered to be cumulative for purposes of defining incidental or minor alterations or additions. The percentage of cumulative changes shall be based on as-built conditions existing on March 7, 1973.*

1638A.1 Alterations. *For this section, alterations include any additions, alterations, repairs, and/or seismic retrofits to an existing hospital building or portions thereof. The provisions of Section 3403, "Additions, Alterations or Repairs" of Chapter 34 of the California Building Code shall apply for hospital buildings.*

1638A.2 Seismic Retrofit. *Any seismic retrofits of hospital buildings required by Article 2 and Article 11, Chapter 6, Part 1, Title 24, shall meet the requirements of Sections 1640A through 1649A.*

> ***EXCEPTION:*** *Hospital buildings evaluated to SPC 1 due to deficiencies identified by Article 10, Chapter 6, Part 1, Title 24, may be upgraded to SPC 2 by altering, repairing or seismically retrofitting these conditions in accordance with the requirements of Sections 1640A through 1649A.*

1638A.3 Alterations, additions and repairs to existing buildings or structures not required by Chapter 6, Part 1, Title 24.

1638A.3.1 Approved existing buildings. *Structural alterations or repairs may be made to approved buildings provided the entire building, as modified, including the structural alterations or repairs, conforms to Sections 1640A through 1649A requirements for the seismic structural performance category (SPC) of the building as determined in Chapter 6, Part 1, Title 24. Additions shall conform to the requirements of these regulations for new construction.*

1638A.3.2 Pre-1973 buildings.

1638A.3.2.1 Incidental structural alteration, additions or repairs. *The existing structural elements affected by the alteration, addition or repair shall conform or shall be made to conform to the vertical load requirements of these regulations. Incidental structural additions will be permitted provided the additions meet these regulations for new construction using the importance factor, I, equal to or greater than 1.0. Alterations or repairs to the existing lateral load-resisting system must meet the requirements of Sections 1640A through 1649A.*

1638A.3.2.2 Minor structural alterations, additions or repairs. *Minor structural alterations, additions or repairs will be permitted provided they meet the following: Alterations to existing gravity and/or lateral load-resisting system shall be made to conform to the requirements of Section 1640A through 1649A; or additions shall meet all of the requirements of these regulations for new construction using an importance factor, I, equal to or greater than 1.0.*

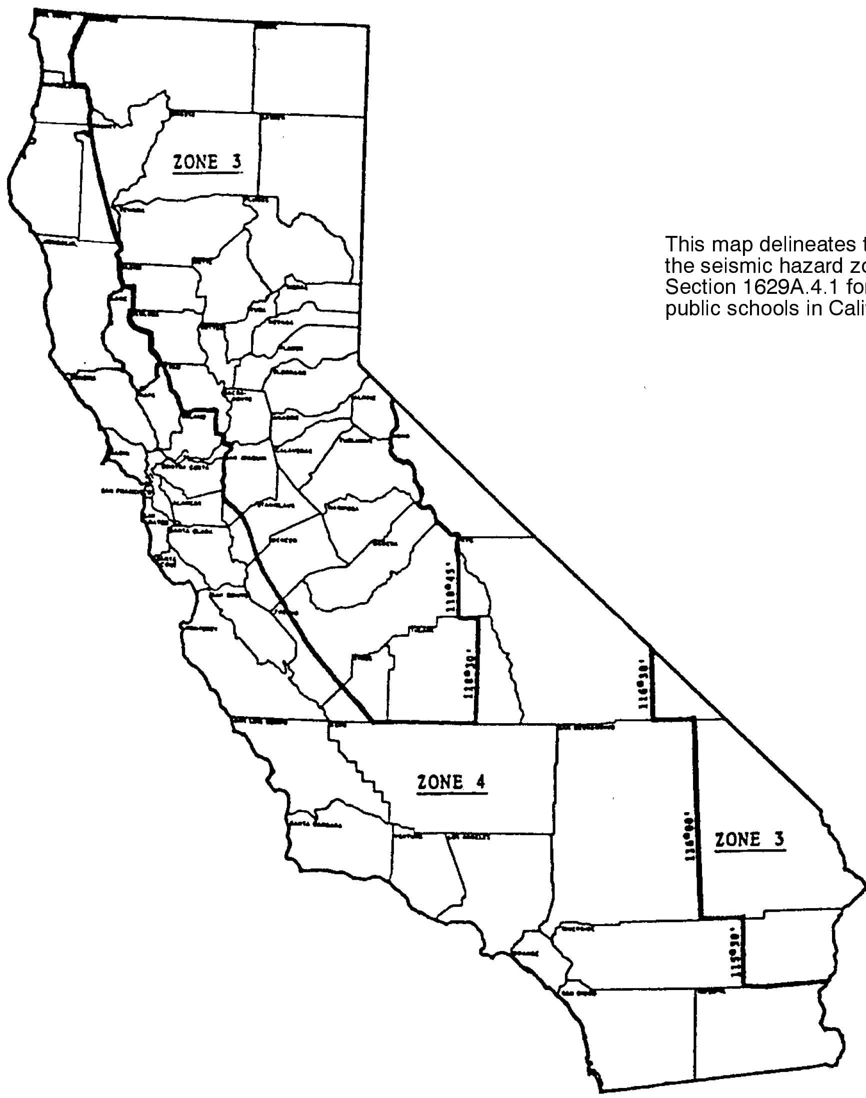
1638A.3.2.3 Major structural alterations, additions or repairs. *Major structural alterations will be permitted provided the entire building, as modified, including the structural alterations or repairs, conforms to the requirements of Sections 1640A through 1649A for no less than SPC 2. Additions shall meet the requirements of these regulations for new construction.*

It shall also be demonstrated by a written report submitted by the structural engineer, acceptable to the enforcement agency, that an investigation of the existing building structure shows it to be constructed in reasonable conformance with the submitted drawings and specifications.

1638A.3.3 *An alteration which involves the removal of one or more entire stories will be permitted if the lateral-load-resisting capacity of the remaining structure is not reduced.*

An alteration which involves the removal of other than one or more entire stories will be permitted in accordance with Sections 1640A through 1649A.

SECTION 1639A — RESERVED



This map delineates the boundaries of the seismic hazard zones as given in Section 1629A.4.1 for hospitals and public schools in California.

FIGURE 16A-2—SEISMIC *HAZARD* ZONE MAP *FOR HOSPITALS AND PUBLIC SCHOOLS IN CALIFORNIA*

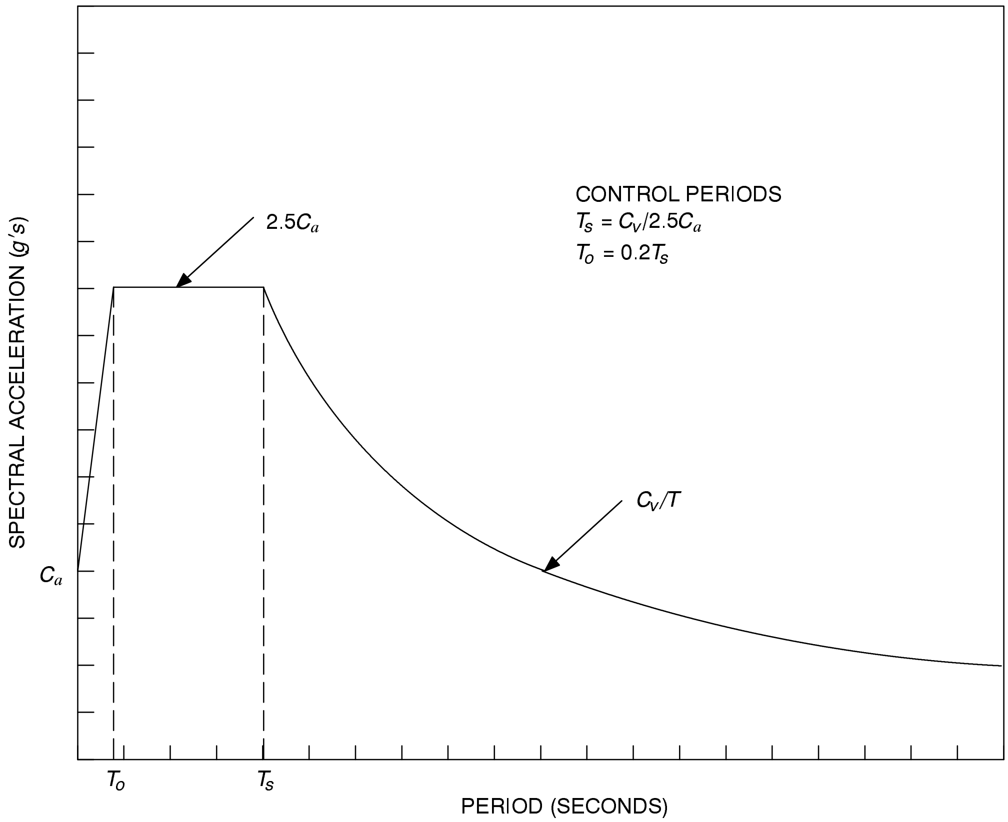


FIGURE 16A-3—DESIGN RESPONSE SPECTRA

servation. The meeting shall include, but is not limited to, the design engineer or architect, structural observer, general constructor, affected subcontractors, the project inspector and a representative of the enforcement agency (designated alternates may attend if approved by the structural observer). The structural observer will schedule and coordinate this meeting.

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

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

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

SECTION *1644A* — METHOD A

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

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

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

***1644A.1.3** Design snow loads of 30 psf (1.44 kN/m^2) or less need not be included. Where design snow loads exceed 30 psf (1.44 kN/m^2), the design snow load shall be included, but may be reduced up to 75 percent where consideration of siting, configuration and load duration warrant when approved by the enforcement agency.*

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

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

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

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

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

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

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

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

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

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

***1644A.3.1** The system R value shall be taken as 4.5 for all existing structural systems except for the following conditions.*

***1644A.3.1.1** R may be taken as 5.5 if the system constructed meets the requirements for a Building Frame System as defined in Section 1629A.6.3.*

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

1644A.4 Static Force Procedures.

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

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

The total design base shear need not exceed the following:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

***1644A.8 Horizontal Torsional Moments.** Provisions shall be made for the increased shears resulting from horizontal torsion where diaphragms are not flexible. The most severe load combination for each element shall be considered for design in conformance with the requirements of Section 1630A.7 for new buildings.*

1644A.9 Overturning.

***1644A.9.1 General.** Every structure shall be designed to resist the overturning effects caused by earthquake forces specified in Section 1630A.5. At any level, the overturning moments to be resisted shall be determined using those seismic forces (F_t and F_x) that act on levels above the level under consideration. At any level, the incremental changes of the design overturning moment shall be distributed to the various resisting elements in the manner prescribed in Section 1630A.6. Overturning effects on every element, wherever possible, shall be carried down directly in a linear path to the foundation. See load combinations in Sections 1644A.4.1.1 and 1644A.4.1.2 for combining gravity and seismic forces.*

***1644A.9.2 Seismic Zones 3 and 4.** In Seismic Zones 3 and 4, where a lateral-load-resisting element is discontinuous, such as for vertical irregularity Type 4 in Table 16A-L or plan irregularity Type 4 in Table 16A-M, columns supporting such elements shall have the strength to resist the axial force resulting from the following load combinations, in addition to all other applicable load combinations:*

$$\phi C_n = D + 0.8L + \Omega_o \beta E \qquad (44A\text{-}9)$$

$$\phi C_n = \Omega_o \beta E - 0.9D \qquad (44A\text{-}10)$$

$\Omega_o \beta E$ in Formulas (44A-9) and (44A-10) need not exceed RE.

***1644A.9.2.1** The axial forces in such columns need not exceed the resultant of the probable strengths of the other elements of the structure that transfer such loads to the column.*

***1644A.9.2.2** Such columns shall be capable of carrying the above-described axial forces without exceeding the usable axial load capacity (ϕC_n) of the column. For designs using working stress methods, this capacity may be determined using an allowable stress increase of 1.7 or acceptable published factors for a given material or element.*

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

1644A.9.2.3 Columns.

***1644A.9.2.3.1 [For DSA/SS]** Such columns shall either resist the above-described axial forces without exceeding the usable axial capacity (ϕC_n), or shall meet the following detailing and member limitations:*

1. Chapter 19, Section 1921.4, for concrete, and Chapter 22, Section 2210, 2211.4 and 2211.5, for steel in structures in Seismic Zones 3 and 4, except for welded steel moment connections where the current SAC Guidelines for columns apply.

2. Chapter 19, Section 1921.8, for concrete, and Chapter 22A, Divisions I and IX, special provisions for developing plastic hinges at ultimate loading, for steel in structures in Seismic Zone 2.

***1644A.9.2.3.2 [For OSHPD 1 & 4]** In order to qualify for a β value equal to 1.0, such columns shall meet the following detailing and member limitations:*

1. Chapter 19A, Section 1921A.4, for concrete, and Chapter 22A, Section 2210A, 2211.4, Items 4 and 5, for steel in structures in Seismic Zones 3 and 4, except for welded steel moment connections where the SAC Interim Guidelines for the Evaluation, Repair, Modification, and Design of Welded Steel Moment Frame Structures, FEMA 267, August, 1995, provisions for columns apply.

***1644A.9.2.4** Transfer girders that support such columns or that provide support for the discontinuous lateral-load-resisting element shall resist the above-described axial forces or support reactions without exceeding the capacity ϕC_n for each mode of failure. For this case, the β factor shall correspond to the properties of the girder.*

***1644A.9.3 At foundation.** See Section 1809A.4 for overturning moments to be resisted at the foundation soil interface. The foundation soil interface shall be capable of resisting the following load combinations on the allowable stress basis of Section 1809A.2 and Table 18A-I-A, and other load combinations need not apply:*

$$D + L + \frac{E}{1.4} \qquad (44A\text{-}11)$$

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

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

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

1645A.5.2 Braced steel frame elements.

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

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

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

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

1645A.6 Wood and Other Sheathing Materials.

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

> ***EXCEPTION:*** *Let-in bracing, plaster (stucco), gypsum wallboard and particle board sheathing shall be classified as limited-ductile or nonductile and assigned a β value given in Table 16A-R-2.*

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

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

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

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

3.1 Indirect shear path.

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

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

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

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

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

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

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

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

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

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

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

1. Communications systems;

2. Emergency power systems;

3. Bulk medical gas systems;

4. Fire alarm systems; and

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

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

1. Those required by Section 1645A.7.1.2;

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

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

> ***EXCEPTIONS:*** *For Section 1645A.7.1.3, seismic restraints need not be provided for cable trays, conduit and HVAC ducting. Seismic restraints may be omitted from piping systems, provided that an approved method of preventing release of the contents of the piping system in the event of a break is provided.*
>
> *2. For Section 1645A.7.1.3, only elevator(s) selected to provide patient, surgical, obstetrical and ground floors during interruption of normal power need meet the structural requirements of Part 2, Title 24.*

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

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

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

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

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

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

1. Those required by Section 1645A.7.1.4;

2. On-site supplies of water and holding tanks for waste water, sufficient for 72 hours of emergency operations, that are inte-

grated into the building plumbing system, including any alternative hook-ups to allow the use of transportable water and sanitary waste water disposal; and

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

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

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

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

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

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

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

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

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

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

SECTION 1646A — DETAILED SYSTEMS DESIGN REQUIREMENTS

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

1646A.1.1 All building components in Seismic Zones 3 and 4 shall be designed to resist the effects of the seismic forces prescribed herein and the effects of gravity loadings from dead, floor live and snow loads.

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

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

1646A.1.3.1 Overturning moment tension resistance for elements and connections: If the tension action due to $\Omega_o E$ - 0.9 D > 0, then the usable tensile strength ϕC_n shall equal or exceed the greater of the tension due to $\Omega_o E - 0.9\,D$ or $E/14$ for semiductile and brittle elements; and $E - 0.9\,D$ or $E/14$ for ductile elements.

1646A.1.3.2 Reversed moment opposite to that caused by gravity loads in beams, slabs and spandrels: If the flexural action due to $\Omega_o E - 0.9\,D > 0$, then the usable flexural strength ϕC_n shall equal or exceed the greater of the moment due to $\Omega_o E - 0.9\,D$ or $E/14$ for semiductile and brittle elements; and $E - 0.9\,D$ or $E/14$ for ductile elements.

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

1646A.1.3.4 For the following conditions:

1. Foundations at the soil-structure interface;

2. Horizontal construction joints in shear walls; or

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

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

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

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

1646A.1.4 In Seismic Zones 3 and 4, provision shall be made for the effects of earthquake forces acting in a direction other than the principal axes in each of the following circumstances:

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

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

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

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

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

1646A.2 Structural Framing Systems.

1646A.2.1 General. Four types of general building framing systems defined in Section 1629A.6 are recognized in these provisions and shown in Table 16A-N. Each type is subdivided by the types of vertical elements used to resist lateral seismic forces. Special framing requirements are given in this section and in Chapters 19A through 23A.

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

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

1646A.2.4 Deformation compatibility. All vertical load-bearing elements not included as a part of the lateral-force-resisting sys-

TABLE 16A-R-1—β VALUES FOR REINFORCED CONCRETE ELEMENTS

These values are given for selected systems; for systems not listed, they are meant to guide the selection of β values by comparison of expected performance at the design level of loading to that for listed systems. Elements that are not included in the lateral-load-resisting system shall be checked for capacity as required in Section 1646A. Refer to Section 1645A.3 for clarification provisions.

ELEMENT/ACTION	β VALUE			NOTES: D = DUCTILE, LD = LIMITED DUCTILE, ND = NONDUCTILE
	Ductile	Limited Ductile	Nonductile	
Beams/Slabs				
Shear	1.0	2.0	4.5	D: Flexure controlled
Flexure	1.0	1.5	2.4	LD: Continuous top and bottom bars, and axial load less than 10% of nominal axial strength capacity.
Columns				
Shear	1.0	2.0	4.5	LD: Ties spaced at not more than the lesser value of one half of column minimum dimension or 12 inches (304.8 mm); or shear resistance governed by flexure. D: Flexure controlled.
Axial-Flexural Yield	1.0	1.5	3.0	Axial-flexural yield is when tensile steel yield occurs before concrete reaches compressive strain limit: below balanced point on interaction curve or where P_u with β 1.0 is less than 0.35 P_o.
Axial-Flexural Crushing	1.0	1.5	4.5	Axial-flexural crushing is when concrete reaches compressive strain limit before tensile steel yield occurs: above balanced point on interaction curve.
Beam-column Joints				
Constrained on four sides	1.0	2.0	N/A	Joints without transverse tie reinforcement may use nominal shear strength equal to two-thirds of the values in Section 1921A.5.3, provided that the β value is applied to the loading. The loading need not exceed the probable yield capacity of the beam longitudinal reinforcement.
Not constrained on four sides	1.0	3.0	N/A	Same as above.
Walls Governed by Flexure				
Shear	1.0	2.0/1.5	3.0	LD: For wall or pier l_w/h_w < $^1/_2$ use 2.00; otherwise, use 1.50. Wall steel to be anchored. Wall or pier shall be considered nonductile if width l_w is less than 4 times wall thickness t.
Flexure	1.0	1.5	2.5	LD: Continuous longitudinal reinforcing steel at top of wall, and axial load less than 10% of nominal axial strength capacity.
Boundary Elements	1.0	1.5	3.0	LD: Continuous edge bars at boundary of walls, and axial load less than 10% of nominal axial strength capacity.
Walls Governed by Shear				
Shear	1.0	2.5/2.0	3.5	LD: For wall and pier l_w/h_w< $^1/_2$ use 2.5; otherwise use 2.0. Wall steel to be anchored. Wall or pier shall be considered nonductile if width l_w is less than 4 times wall thickness t.
Flexure	1.0	2.5	2.5	
Boundary	1.0	3.0	3.0	
Foundations	1.0	1.5	2.5	LD: Longitudinal reinforcing bars are provided, with steel ratio ≥ ρ_{min} but not sufficient to comply with code. ND: Reinforcement is with unknown or not provided.
Shear				
Flexure	1.0	1.5	2.5	LD: Reinforcing bars are provided, but not sufficient to comply with code. ND: Reinforcement is with unknown or not provided.
Shear Friction	1.0	2.0	—	LD: Dowels have embedment length at least 50% of their development length.
Anchorage	1.0	3.0	4.5	
Collectors	1.0	3.0	4.5	

TABLE 16A-R-2—β VALUES FOR WOOD AND OTHER SHEATHING MATERIALS

These values are given for selected systems; for systems not listed, they are meant to guide the selection of β values by comparison of expected performance at the design level of loading to that for listed systems. Elements that are not included in the lateral load-resisting system shall be checked for capacity as required in Section 1646A. Drift should be considered for its effect on nonstructural elements. These values apply to both shear and flexure.

ELEMENT/ACTION	β VALUE			NOTES: D = DUCTILE, LD = LIMITED DUCTILE, ND = NONDUCTILE
	Ductile	Limited Ductile	Nonductile	
Plywood walls	1.0	1.5	3.0	D: If boundary, collector and splice elements present, then use β = 1.00; LD: for $l/h < 1/2$; ND: for $l/h < 1/2$ and lacking hold downs.
Plywood diaphragms	1.0	1.5	2.5	D: If boundary, collector and splice elements present.
Walls with diagonal sheathing	—	2.0	3.0	ND: If in poor condition and limited nailing.
Diaphragms with diagonal sheathing	1.0	2.0	3.0	
Let-in bracing or steel strap bracing	—	—	4.5	
Straight sheathing	—	2.5	4.5	LD: If sheathing is greater than 1 × 6 and well nailed.
Stucco	—	2.0	4.5	LD: Only if verified screen attachment, otherwise, ND.
Lath and plaster	—	2.0	4.0	LD: If nailed lath and plaster in good condition.
Plaster on stiff substrate	—	—	4.5	
Particle board	—	2.0	4.0	LD: If boundary, collectors and splice elements present and nailing is certified to comply.
Gypsum wall board	—	2.5	4.5	LD: With full edge and field nailing without splitting of paper or plaster, and nailing is verified to comply.
Wood bracing (axial)	1.0	1.5	3.0	LD: Nailing relationship to the grain is a particularly important consideration.
Wood in flexure	1.0	2.0	4.0	LD: Nailing relationship to the grain is a particularly important consideration.
Collectors	1.0	1.5	4.0	LD: With added metal continuity elements properly installed.

TABLE 16A-R-3—VALUES OF F_a AS A FUNCTION OF SITE CLASS AND MAPPED SHORT-PERIOD SPECTRAL RESPONSE ACCELERATION S_S

SITE CLASS	MAPPED SPECTRAL ACCELERATION AT SHORT PERIODS S_S				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	*
F	*	*	*	*	*

NOTE: *Use straight-line interpolation for intermediate values of S_S.*
**Site-specific geotechnical investigation and dynamic site response analyses shall be performed.*

TABLE 16A-R-4—VALUES OF F_V AS A FUNCTION OF SITE CLASS AND MAPPED SPECTRAL RESPONSE ACCELERATION AT 1-SECOND PERIOD S_1

SITE CLASS	MAPPED SPECTRAL ACCELERATION AT 1-SECOND PERIOD S_1				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	*
F	*	*	*	*	*

NOTE: *Use straight-line interpolation for intermediate values of S_1.*
**Site-specific geotechnical investigation and dynamic site response analyses shall be performed.*

Chapter 17
STRUCTURAL TESTS AND INSPECTIONS

SECTION 1701 — SPECIAL INSPECTIONS

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

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

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

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

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

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

1701.4 Standards of Quality. The standards listed below labeled a "UBC Standard" are also listed in Chapter 35, Part II, and are part of this code. The other standards listed below are recognized standards. (See Sections 3503 and 3504.)

1. **Concrete.**

ASTM C 94, Ready-mixed Concrete

2. **Connections.**

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

5. **Structural welding.**

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

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

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

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

2.2 Floor and roof deck welding.

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

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

2.5 Welding of stairs and railing systems.

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

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

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

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

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

7. **Structural masonry.**

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

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

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

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

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

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

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

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

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

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

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

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

14. **Smoke-control system.**

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

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

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

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

1701.6 Continuous and Periodic Special Inspection.

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

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

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

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

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

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

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

SECTION 1702 — STRUCTURAL OBSERVATION

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

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

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

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

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

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

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

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

SECTION 1703 — NONDESTRUCTIVE TESTING

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

As a minimum, this program shall include the following:

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

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

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

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

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

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

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

SECTION 1704 — PREFABRICATED CONSTRUCTION

1704.1 General.

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

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

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

1704.1.3 Definition.

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

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

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

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

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

1704.6 Certificate and Inspection.

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

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

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

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

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

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

Chapter 17A *[For DSA/SS, OSHPD 1 & 4]*
STRUCTURAL TESTS AND INSPECTIONS

NOTES: 1. This chapter is applicable to public schools, community colleges and state-owned or state-leased essential services buildings regulated by the Division of the State Architect, Structural Safety Section.

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

[For OSHPD 2] EXCEPTION: Single-story Type V skilled nursing or intermediate care facilities utilizing wood-frame or light-steel-frame construction as defined in Health and Safety Code Section 129725 which shall comply with UBC Chapter 17 and any applicable amendments therein.

SECTION 1701A — SPECIAL INSPECTIONS

1701A.1 General.

1701A.1.1 [For DSA/SS] In addition to the *project inspector required by Title 24, Part 1, Section 4-333, the school district* shall employ one or more special inspectors who shall provide inspections during construction on the types of work listed under Section 1701A.5.

1701A.1.2 [For OSHPD 1 and 4] In addition to the *project inspector required by Title 24, Part 1, Section 7-144, the hospital* shall employ one or more special inspectors who shall provide inspections during construction on the types of work listed under Section 1701A.5, *Chapters 18A, 19A, 20A, 21A, 22A, 23A, and noted in the special test, inspection and observation plan required by Sections 7-141, 7-145 and 7-149 of Title 24, Part 1, of the California Building Standards Code.*

1701A.2 *Project and* Special Inspector.

1701A.2.1 [For DSA/SS] The *project inspectors and all* special inspectors shall be qualified persons who shall demonstrate competence, to the satisfaction of the *enforcement agency,* for inspection of the particular type of construction or operation requiring special inspection *in accordance with Title 24, Part 1, Section 4-333.*

1701A.2.2 [For OSHPD 1 & 4] The *project inspectors and all* special inspectors shall be qualified persons who shall demonstrate competence, to the satisfaction of the *enforcement agency,* for inspection of the particular type of construction or operation requiring special inspection *in accordance with Title 24, Part 1, Section 7-144.*

1701A.3 Duties and Responsibilities of the *Project and* Special Inspectors.

1701A.3.1 [For DSA/SS] The *project and* special inspector*s* shall observe the work assigned for conformance to the approved design drawings and specifications.

The *project inspector and* special inspector*s* shall *submit* inspection report*s* to the *enforcement agency* and other designated persons *as required by Title 24, Part 1, Sections 4-336 and 4-342.* All discrepancies shall be brought to the immediate attention of the contractor for correction *in accordance with Title 24, Part 1, Section 4-342.*

The *project inspector and* special inspector*s* shall submit reports *as required by Title 24, Part 1, Section 4-336 and 4-342 stating that they have personal knowledge that* the work *has been performed and that the materials used and installed are in compliance with* the approved plans and specifications and the applicable workmanship provisions of this code.

1701A.3.2 [For OSHPD 1 and 4] The *project and* special inspectors shall observe the work assigned for conformance to the approved design drawings and specifications.

The *project inspector and* special inspectors shall *submit* inspection reports to the *enforcement agency* and other designated persons *as required by Title 24, Part 1, Sections 7-145 and 7-151.* All discrepancies shall be brought to the immediate attention of the contractor for correction *in accordance with Title 24, Part 1, Section 7-145.*

The *project inspector and* special inspectors shall *submit* reports *as required by Title 24, Part 1, Sections 7-141(g), 7-145 and 7-151, stating that they have personal knowledge that* the work *has been performed and that the materials used and installed are in compliance with* the approved plans and specifications and the applicable workmanship provisions of this code.

1701A.4 Standards of Quality. The standards listed below labeled a "UBC Standard" are also listed in Chapter 35, Part II, and are part of this code. The other standards listed below are recognized standards *and are also part of this code.* (See Sections 3503 and 3504.)

1. **Concrete.**

ASTM C 94, Ready-mixed Concrete

2. **Connections.**

Specification for Structural Joints Using ASTM A 325 or A 490 Bolts-Load and Resistance Factor Design, Research Council of Structural Connections, Section 1701A.5, Item 6.

Specification for Structural Joints Using ASTM A 325 or A 490 Bolts-Allowable Stress Design, Research Council of Structural Connections, Section 1701A.5, Item 6.

3. **Spray-applied Fire-resistive Materials.**

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

1701A.5 Types of Work *Requiring Constant Presence of the Project or Special Inspector.* The types of work listed below shall *require constant presence and inspection (see Title 24, Part 1, Section 4-342) [For OSHPD 1 and 4: see Title 24, Part 1, Section 7-145] by the project or special inspector. A special inspector is needed when stated.*

1. **Concrete.** During the taking of test specimens and placing of reinforced concrete. See Item 12 for shotcrete. *See Chapter 19A, Division IX, Section 1929A for additional requirements.*

2. **Bolts installed in concrete.** Prior to and during the placement of concrete around bolts.

3. **Special moment-resisting concrete frame.** For moment frames resisting design seismic load in structures * * *, the *project or* special inspector * * * shall provide *constant* inspection of the placement of the reinforcement and concrete. *See Chapter 19A, Division IX, Section 1929A for additional requirements.*

4. ***Reinforcing steel and* prestressing steel tendons.** *Special inspector required.*

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

4.2 *[For DSA/SS]* During placing of ✳ ✳ ✳ prestressing tendons ✳ ✳ ✳.

EXCEPTION: The special inspector need not be present *constantly* during placing of ✳ ✳ ✳ prestressing tendons, provided the special inspector has inspected for conformance to the approved plans prior to the closing of forms or the delivery of concrete to the jobsite. *See Chapter 19A, Division IX, Section 1929A for additional requirements.*

4.3 [For OSHPD 1 & 4] During placing of reinforcing steel and prestressing tendons.

***EXCEPTION:** The special inspector need not be present constantly during placing of reinforcing steel and prestressing tendons, provided the special inspector has inspected for conformance to the approved plans prior to the closing of forms or the delivery of concrete to the jobsite. See Chapter 19A, Division IX, Section 1929A, for additional requirements.*

5. **Structural welding.** *Special inspector required.*

5.1 **General.** During the welding of any member or connection that is designed to resist loads and forces required by this code. *See also Chapter 22A, Division XII, Section 2231A for additional requirements.*

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

1. Floor and roof deck welding.

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

3. Welded sheet steel for cold-formed steel framing members such as studs and joists *which are not part of a lateral-force-resisting system.*

4. Welding of stairs and railing systems.

5.2 **Special moment-resisting steel frames.** During the welding of special moment-resisting steel frames. In addition to Item 5.1 requirements, nondestructive testing as required by Section 1703A *and Chapter 22A, Division XII, Section 2231A* of this code.

5.3 **Welding of reinforcing steel.** During the welding of reinforcing steel. *See Chapter 19A, Division IX, Section 1929A for additional requirements.*

6. **High-strength bolting.** *Special inspector required.* The inspection of high-strength A 325 and A 490 bolts shall be in accordance with *the standards adopted as part of this code,* approved nationally recognized standards and the requirements of this section.

While the work is in progress, the special inspector shall determine that the requirements for bolts, nuts, washers and paint; bolted parts; and installation and tightening in such standards are met. ✳ ✳ ✳ The special inspector shall observe the calibration procedures when such procedures are required by the plans or specifications and shall monitor the installation of bolts to determine that all plies of connected materials have been drawn together and that the selected procedure is properly used to tighten all bolts.

7. **Structural masonry.** *Special inspector required.*

7.1 For masonry, ✳ ✳ ✳ during preparation and taking of any required prisms or test specimens, placing of all masonry units, placement of reinforcement, inspection of grout space, immediately prior to closing of cleanouts, and during all grouting operations. *See Section 2105A for additional requirements.*

8. **Reinforced gypsum concrete.** When cast-in-place Class B gypsum concrete is being mixed and placed. *See Chapter 19A, Division IX, Section 1929A for additional requirements.*

9. **Insulating concrete fill.** During the application of insulating concrete fill when used as part of a structural system. *See Chapter 19A, Division IX, Section 1929A for additional requirements.*

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

11. **Piling, drilled piers and caissons.** *Special inspector required.* During driving and testing of piles and construction of cast-in-place drilled piles or caissons. See Items 1 and 4 for concrete and reinforcing steel inspection. *See also Chapter 18A, Division I, Section 1809A, Chapter 19A, Division IX, Section 1929A and Chapter 22A, Division XII, Section 2231A for additional requirements.*

12. **Shotcrete.** *Special inspector required.* During the taking of test specimens and placing of all shotcrete and as required by Sections 1924A.10 and 1924A.11. *See Chapter 19A, Division IX, Section 1929A for additional requirements.*

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

14. **Smoke-control system.**

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

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

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

*16. **Manufactured trusses.** The fabrication of trusses and other assemblages constructed using wood and metal members, or using light metal plate connectors, shall be continuously inspected by a qualified inspector approved by the enforcement agency. See Chapter 23A, Division IX, Section 2337A, for additional requirements.*

*17. **Glued-laminated timber.** The fabrication of all glued-laminated timber shall be continuously inspected by a qualified inspector approved by the enforcement agency. See Chapter 23A, Division IX, Section 2337A, for additional requirements.*

*18. **Post installed anchors.** The installation of drilled-in expansion or chemical-type anchors in concrete or masonry shall be continuously inspected by a qualified inspector approved by the enforcement agency. See Chapter 19A, Division III, Section 1923A.3.5, and the appropriate evaluation report for additional requirements.*

SECTION 1702A — OBSERVATION *OF THE CONSTRUCTION*

1702A.1 [For DSA/SS] Observation *of the construction* shall be provided *by the architect or engineer in general responsible charge as set forth in Title 24, Part 1, Sections 4-333 and 4-341.*

***1702A.2 [For OSHPD 1 and 4]** Observation of the construction shall be provided by the architect or engineer in general responsible charge as set forth in Title 24, Part 1, Sections 7-141 and 7-144.*

SECTION 1703A — NONDESTRUCTIVE TESTING

1703A.1 [For DSA/SS] ✳ ✳ ✳ Welded, fully restrained connections between the primary members of ordinary moment frames ting shall be carried out in such a manner that the carrying capacity of existing piles and structures shall not be impaired. After withdrawal of the jet, piles shall be driven down until the required resistance is obtained.

1807A.9 Protection of Pile Materials. Where the boring records of site conditions indicate possible deleterious action on pile materials because of soil constituents, changing water levels or other factors, such materials shall be adequately protected by methods or processes approved by the *enforcement agency.* The effectiveness of such methods or processes for the particular purpose shall have been thoroughly established by satisfactory service records or other evidence which demonstrates the effectiveness of such protective measures.

1807A.10 Allowable Loads. The allowable loads based on soil conditions shall be established in accordance with Section 1807A.

> **EXCEPTION:** Any uncased cast-in-place pile may be assumed to develop a frictional resistance equal to one sixth of the bearing value of the soil material at minimum depth as set forth in Table 18A-I-A but not to exceed 500 pounds per square foot (24 kPa) unless a greater value is allowed by the *enforcement agency* after a soil investigation as specified in Section 1804A is submitted. Frictional resistance and bearing resistance shall not be assumed to act simultaneously unless recommended after a foundation investigation as specified in Section 1804A.

1807A.11 Use of Higher Allowable Pile Stresses. Allowable compressive stresses greater than those specified in Section 1808A shall be permitted when substantiating data justifying such higher stresses are submitted to and approved by the *enforcement agency.* Such substantiating data shall include a foundation investigation including a report in accordance with Section 1807A.1 by a soils engineer defined as a civil engineer experienced and knowledgeable in the practice of soils engineering.

SECTION 1808A — SPECIFIC PILE REQUIREMENTS

1808A.1 Round Wood Piles.

1808A.1.1 Material. Except where untreated piles are permitted, wood piles shall be pressure treated. Untreated piles may be used only when it has been established that the cutoff will be below lowest groundwater level assumed to exist during the life of the structure.

1808A.1.2 Allowable stresses. The allowable unit stresses for round wood piles shall not exceed those set forth in Chapter 23A, Division III, Part I.

The allowable values listed in Chapter 23A, Division III, Part I, for compression parallel to the grain at extreme fiber in bending are based on load sharing as occurs in a pile cluster. For piles which support their own specific load, a safety factor of 1.25 shall be applied to compression parallel to the grain values and 1.30 to extreme fiber in bending values.

1808A.2 Uncased Cast-in-place Concrete Piles.

1808A.2.1 Material. Concrete piles cast in place against earth in drilled or bored holes shall be made in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. The length of such pile shall be limited to not more than 30 times the average diameter. Concrete shall have a specified compressive strength f'_c of not less than 2,500 psi (17.24 MPa).

> **EXCEPTION:** The length of pile may exceed 30 times the diameter provided the design and installation of the pile foundation is in accordance with an approved investigation report.

1808A.2.2 Allowable stresses. The allowable compressive stress in the concrete shall not exceed $0.33f'_c$. The allowable compressive stress of reinforcement shall not exceed 34 percent of the yield strength of the steel or 25,500 psi (175.7 MPa).

1808A.3 Metal-cased Concrete Piles.

1808A.3.1 Material. Concrete used in metal-cased concrete piles shall have a specified compressive strength f'_c of not less than 2,500 psi (17.24 MPa).

1808A.3.2 Installation. Every metal casing for a concrete pile shall have a sealed tip with a diameter of not less than 8 inches (203 mm).

Concrete piles cast in place in metal shells shall have shells driven for their full length in contact with the surrounding soil and left permanently in place. The shells shall be sufficiently strong to resist collapse and sufficiently watertight to exclude water and foreign material during the placing of concrete.

Piles shall be driven in such order and with such spacing as to ensure against distortion of or injury to piles already in place. No pile shall be driven within four and one-half average pile diameters of a pile filled with concrete less than 24 hours old unless approved by the *enforcement agency.*

1808A.3.3 Allowable stresses. Allowable stresses shall not exceed the values specified in Section 1808A.2.2, except that the allowable concrete stress may be increased to a maximum value of $0.40f'_c$ for that portion of the pile meeting the following conditions:

1. The thickness of the metal casing is not less than 0.068 inch (1.73 mm) (No. 14 carbon sheet steel gage).

2. The casing is seamless or is provided with seams of equal strength and is of a configuration that will provide confinement to the cast-in-place concrete.

3. The specified compressive strength f'_c shall not exceed 5,000 psi (34.47 MPa) and the ratio of steel minimum specified yield strength f_y to concrete specified compressive strength f'_c shall not be less than 6.

4. The pile diameter is not greater than 16 inches (406 mm).

1808A.4 Precast Concrete Piles.

1808A.4.1 Materials. Precast concrete piles shall have a specified compressive strength f'_c of not less than 3,000 psi (20.68 MPa), and shall develop a compressive strength of not less than 3,000 psi (20.68 MPa) before driving.

1808A.4.2 Reinforcement ties. The longitudinal reinforcement in driven precast concrete piles shall be laterally tied with steel ties or wire spirals. Ties and spirals shall not be spaced more than 3 inches (76 mm) apart, center to center, for a distance of 2 feet (610 mm) from the ends and not more than 8 inches (203 mm) elsewhere. The gage of ties and spirals shall be as follows:

For piles having a diameter of 16 inches (406 mm) or less, wire shall not be smaller than 0.22 inch (5.6 mm) (No. 5 B.W. gage).

For piles having a diameter of more than 16 inches (406 mm) and less than 20 inches (508 mm), wire shall not be smaller than 0.238 inch (6.0 mm) (No. 4 B.W. gage).

For piles having a diameter of 20 inches (508 mm) and larger, wire shall not be smaller than $^1/_4$ inch (6.4 mm) round or 0.259 inch (6.6 mm) (No. 3 B.W. gage).

1808A.4.3 Allowable stresses. Precast concrete piling shall be designed to resist stresses induced by handling and driving as well as by loads. The allowable stresses shall not exceed the values specified in Section 1808A.2.2.

1808A.5 Precast Prestressed Concrete Piles (Pretensioned).

1808A.5.1 Materials. Precast prestressed concrete piles shall have a specified compressive strength f'_c of not less than 5,000 psi (34.48 MPa) and shall develop a compressive strength of not less than 4,000 psi (27.58 MPa) before driving.

1808A.5.2 Reinforcement. The longitudinal reinforcement shall be high-tensile seven-wire strand. Longitudinal reinforcement shall be laterally tied with steel ties or wire spirals.

Ties or spiral reinforcement shall not be spaced more than 3 inches (76 mm) apart, center to center, for a distance of 2 feet (610 mm) from the ends and not more than 8 inches (203 mm) elsewhere.

At each end of the pile, the first five ties or spirals shall be spaced 1 inch (25 mm) center to center.

For piles having a diameter of 24 inches (610 mm) or less, wire shall not be smaller than 0.22 inch (5.6 mm) (No. 5 B.W. gage). For piles having a diameter greater than 24 inches (610 mm) but less than 36 inches (914 mm), wire shall not be smaller than 0.238 inch (6.0 mm) (No. 4 B.W. gage). For piles having a diameter greater than 36 inches (914 mm), wire shall not be smaller than $^1/_4$ inch (6.4 mm) round or 0.259 inch (6.6 mm) (No. 3 B.W. gage).

1808A.5.3 Allowable stresses. Precast prestressed piling shall be designed to resist stresses induced by handling and driving as well as by loads. The effective prestress in the pile shall not be less than 400 psi (2.76 MPa) for piles up to 30 feet (9144 mm) in length, 550 psi (3.79 MPa) for piles up to 50 feet (15 240 mm) in length, and 700 psi (4.83 MPa) for piles greater than 50 feet (15 240 mm) in length.

The compressive stress in the concrete due to externally applied load shall not exceed:

$$f_c = 0.33f'_c - 0.27fp_c$$

WHERE:

fp_c = effective prestress stress on the gross section.

Effective prestress shall be based on an assumed loss of 30,000 psi (206.85 MPa) in the prestressing steel. The allowable stress in the prestressing steel shall not exceed the values specified in Section 1918A.

1808A.6 Structural Steel Piles.

1808A.6.1 Material. Structural steel piles, steel pipe piles and fully welded steel piles fabricated from plates shall conform to UBC Standard 22-1 and be identified in accordance with Section 2202A.2.

1808A.6.2 Allowable stresses. The allowable axial stresses shall not exceed 0.35 of the minimum specified yield strength F_y or 12,600 psi (86.88 MPa), whichever is less.

> **EXCEPTION:** When justified in accordance with Section 1807A.11, the allowable axial stress may be increased above 12,600 psi (86.88 MPa) and $0.35F_y$, but shall not exceed $0.5F_y$.

1808A.6.3 Minimum dimensions. Sections of driven H-piles shall comply with the following:

1. The flange projection shall not exceed 14 times the minimum thickness of metal in either the flange or the web, and the flange widths shall not be less than 80 percent of the depth of the section.

2. The nominal depth in the direction of the web shall not be less than 8 inches (203 mm).

3. Flanges and webs shall have a minimum nominal thickness of $^3/_8$ inch (9.5 mm).

Sections of driven pipe piles shall have an outside diameter of not less than 10 inches (254 mm) and a minimum thickness of not less than $^1/_4$ inch (6.4 mm).

1808A.7 Concrete-filled Steel Pipe Piles.

1808A.7.1 Material. The concrete-filled steel pipe piles shall conform to UBC Standard 22-1 and shall be identified in accordance with Section 2202A.2. The concrete-filled steel pipe piles shall have a specified compressive strength f'_c of not less than 2,500 psi (17.24 MPa).

1808A.7.2 Allowable stresses. The allowable axial stresses shall not exceed 0.35 of the minimum specified yield strength F_y of the steel plus 0.33 of the specified compressive strength f'_c of concrete, provided F_y shall not be assumed greater than 36,000 psi (248.22 MPa) for computational purposes.

> **EXCEPTION:** When justified in accordance with Section 2807.11, the allowable stresses may be increased to 0.50 F_y.

1808A.7.3 Minimum dimensions. Driven piles of uniform section shall have a nominal outside diameter of not less than 8 inches (203 mm).

SECTION 1809A — FOUNDATION CONSTRUCTION—SEISMIC ZONES 3 AND 4

1809A.1 General. In Seismic Zones 3 and 4 the further requirements of this section shall apply to the design and construction of foundations, foundation components and the connection of superstructure elements thereto.

1809A.2 Soil Capacity. The foundation shall be capable of transmitting the design base shear and overturning forces prescribed in Section 1630A from the structure into the supporting soil. The short-term dynamic nature of the loads may be taken into account in establishing the soil properties.

1809A.3 Superstructure-to-Foundation Connection. The connection of superstructure elements to the foundation shall be adequate to transmit to the foundation the forces for which the elements were required to be designed.

1809A.4 Foundation-Soil Interface. For regular buildings, the force F_t as provided in Section 1630A.5 may be omitted when determining the overturning moment to be resisted at the foundation-soil interface.

1809A.5 Special Requirements for Piles and Caissons.

1809A.5.1 General. Piles, caissons and caps shall be designed according to the provisions of Section 1603A, including the effects of lateral displacements. *Whenever such members are founded in Type S_D, S_E, or S_F soils,* special detailing requirements as described in Section 1809A.5.2 shall apply for a length of *such members* equal to 120 percent of the flexural length. Flexural length shall be considered as a length of pile from the first point of zero lateral deflection to the underside of the pile cap or grade beam.

1809A.5.2 Steel piles, nonprestressed concrete piles and prestressed concrete piles.

1809A.5.2.1 Steel piles. Piles shall conform to width-thickness ratios of stiffened, unstiffened and tubular compression elements as shown in Chapter 22A, Division *IX.*

1809A.5.2.2 Nonprestressed concrete piles. Piles shall have transverse reinforcement meeting the requirements of Section 1921A.4.

> **EXCEPTION:** Transverse reinforcement need not exceed the amount determined by Formula (21-2) in Section 1921A.4.4.1 for spi-

1921A.3.4. Stirrups shall be placed at not more than $d/2$ throughout the length of the member.

1921A.7.3.3 Members with factored gravity axial forces exceeding ($Agf'_c/10$) shall satisfy Sections 1921A.4.4, 1921A.4.5 and 1921A.5.2.1.

1921A.7.4 Ties at anchor bolts. *Anchor bolts set in the top of a column shall be enclosed with ties as specified in Section 1921A.4.4.8.*

1921A.8 *Not adopted by the State of California.*

SECTION 1922A — ✳ ✳ ✳ PLAIN CONCRETE

1922A.1 *Plain concrete shall not be used other than as fill. The minimum specified compression strength of concrete used as fill shall be 1,500 psi (10.3 MPa) at 28 days.* CA CAC

Division III—DESIGN STANDARD FOR ANCHORAGE TO CONCRETE

SECTION 1923A — ANCHORAGE TO CONCRETE

1923A.1 ***Service Load Design.*** *Bolts and headed stud anchors shall be solidly cast in concrete and the service load shear and tension shall not exceed the values set forth in Table 19A-D. All bolts shall be accurately and securely set prior to placement of concrete, except as indicated in Section 1916A.7.1.*

For combined tension and shear:

$$(P_s/P_t)^{5/3} + (V_s/V_t)^{5/3} \leq 1$$

WHERE:

P_s = *applied service tension load.*

P_t = *Table 19A-D service tension load.*

V_s = *applied service shear load.*

V_t = *Table 19A-D service shear load.*

1923A.2 ***Strength Design.*** *The factored loads on embedded anchor bolts and headed studs shall not exceed the design strengths determined by Section 1923A.3.*

All bolts shall be accurately and securely set prior to placement of concrete except as indicated in Section 1916A.7.1.

*In addition to the load factors in Section 1909A.2, a multiplier of 1.3 shall be used. * * * When anchors are embedded in the tension zone of a member, the load factors in Section 1909A.2 shall have a multiplier of 2.*

1923A.3 Strength of Anchors.

1923A.3.1 General.

1923A.3.1.1 [For DSA/SS] *The strength of headed bolts and headed studs solidly cast in concrete shall be taken as the average of 10 tests approved by the enforcement agency for each concrete strength and anchor size. Alternatively, the strength of the anchor shall be calculated in accordance with Sections 1923A.3.2 through 1923A.3.4. The bearing area of headed anchors shall be at least one and one-half times the shank area.*

1923A.3.1.2 [For OSHPD 1 & 4] *All bolts shall be accurately and securely set prior to placement of concrete, except as indicated in Section 1916A.7.1. The strength of headed bolts and headed studs solidly cast in concrete shall be taken as the average of 10 tests approved by the enforcement agency for each concrete strength and anchor size. Alternatively, the strength of the anchor shall be calculated in accordance with Sections 1923A.3.2 through 1923A.3.4. The bearing area of headed anchors shall be at least one and one-half times the shank area.*

1923A.3.2 Design strength in tension. *The design strength of anchors in tension shall be the minimum of P_{ss} or $\phi\, P_c$ where:*

$$P_{ss} = 0.9\, A_b\, f_{ut}$$

and for an anchor group where the distance between anchors is less than twice their embedment length or for a single anchor or anchor group where the distance between anchors is equal to or greater than twice their embedment length

$$\phi\, P_c = \phi\lambda\, 4\, A_p\, \sqrt{f'_c}$$

For **SI**: $$\phi\, P_c = 0.32\, \phi\lambda\, A_p\, \sqrt{f'_c}$$

WHERE:

A_b = *nominal area [in square inches (mm^2)] of anchor. Must be used with the corresponding steel properties to determine the weakest part of the assembly in tension.*

A_p = *the effective area [in square inches (mm^2)] of the projection of an assumed concrete failure surface upon the surface from which the anchor protrudes. For a single anchor or for an anchor group where the distance between anchors is equal to or greater than twice their embedment length, the failure surface is assumed to be that of a truncated cone radiating at a 45-degree slope from the bearing edge of the anchor toward the surface from which the anchor protrudes. The effective area is the projection of the cone on this surface. For an anchor which is perpendicular to the surface from which it protrudes, the effective area is a circle.*

For an anchor group where the distance between anchors is less than twice their embedment length, the failure surface is assumed to be that of a truncated pyramid radiating at a 45-degree slope from the bearing edge of the anchor group toward the surface from which the anchors protrudes. The effective area is the projection of this truncated pyramid on this surface. In addition, for thin sections with anchor groups, the failure surface shall be assumed to follow the extension of this slope through to the far side rather than be truncated, and the failure mode resulting in the lower value of ϕP_c shall control.

d_b = *anchor shank diameter.*

f'_c = *specified compression strength of concrete, which shall not be taken as greater than 6,000 psi (41.37 MPa) for design.*

f_{ut} = *minimum specified tensile strength [in psi (MPa)] of the anchor. May be assumed to be 60,000 psi (413.7 MPa) for A 307 bolts or A 108 studs.*

P_c = *design tensile strength [in pounds (MPa)].*

P_u = *required tensile strength from factored loads, pounds (N).*

V_c = *design shear strength [in pounds (MPa)].*

V_u = *required shear strength from factored loads, pounds (N).*

λ = *1 for normal-weight concrete, 0.75 for "all lightweight" concrete, and 0.85 for "sand-lightweight" concrete.*

ϕ = *strength reduction factor*

= *0.65.*

EXCEPTION: *When the anchor is attached to or hooked around reinforcing steel or otherwise terminated to effectively transfer forces to the reinforcing steel that is designed to distribute forces and avert sudden local failure, ϕ may be taken as 0.85.*

Where edge distance is less than embedment length, reduce ϕP_c proportionately. For multiple edge distances less than embedment length, use multiple reductions.

1923A.3.3 Design strength in shear. *The design strength of anchors in shear shall be the minimum of V_{ss} or $\phi\, V_c$ where:*

$$V_{ss} = 0.75\, A_b\, f_{ut}$$

and where loaded toward an edge greater than 10 diameters away,

$$\phi\, V_c = \phi\, 800\, A_b\, \lambda\sqrt{f'_c}$$

For **SI**: $$\phi V_c = 66.4\, \phi A_b\, \lambda\sqrt{f'_c}$$

or where loaded toward an edge equal to or less than 10 diameters away,

$$\phi V_c = \phi\, 2\pi d_e^{\,2}\, \lambda\sqrt{f'_c}$$

For **SI**: $$\phi\, V_c = 0.166\, \phi\pi d_e^{\,2}\, \lambda\sqrt{f'_c}$$

Section 2107A.2.2.6 shall be considered as continuous reinforcement.

Horizontal reinforcement shall be provided in the top of footings, at the top of wall openings, at roof and floor levels, and at the top of parapet walls. For walls 12 inches (nominal) (305 mm) or more in thickness, reinforcing shall be equally divided into two layers, except where designed as retaining walls. Where reinforcement is added above the minimum requirements, such additional reinforcement need not be so divided.

In bearing walls of every type of reinforced masonry, there shall not be less than one No. 5 bar or two No. 4 bars on all sides of, and adjacent to, every opening which exceeds 24 inches (610 mm) in either direction, and such bars shall extend not less than 48 diameters, but in no case less than 24 inches (610 mm) beyond the corners of the opening. The bars required by this paragraph shall be in addition to the minimum reinforcement elsewhere required.

When the reinforcement in bearing walls is designed, placed and anchored in position as for columns, the allowable stresses shall be as for columns. The length of the wall to be considered effective shall not exceed the center-to-center distance between loads nor shall it exceed the width of the bearing plus four times the wall thickness.

1. **Column reinforcement.** *The spacing of column ties shall be as follows: not greater than 8 bar diameters, 24 tie diameters, or one half the least dimension of the column for* the full column height. *Top tie shall be within 2 inches (51 mm) of the top of the column or of the bottom of the horizontal bar in the supported beam.*

Column ties shall terminate with a minimum 135-degree hook with extensions not less than six bar diameters or 4 inches (102 mm). Such extensions shall engage the longitudinal column reinforcement and project into the interior of the column. Hooks shall comply with Section 2107A.2.2.5, Item 3.

EXCEPTION: Where ties are placed in horizontal bed joints, hooks shall consist of a 90-degree bend having an inside radius of not less than four tie diameters plus an extension of 32 tie diameters.

2. **Shear walls.**

2.1 **Reinforcement.** The portion of the reinforcement required to resist shear shall be uniformly distributed and shall be joint reinforcement, deformed bars or a combination thereof. The spacing of reinforcement in each direction shall not exceed *24 inches (610 mm) each way.*

Joint reinforcement used in exterior walls and considered in the determination of the shear strength of the member shall be hot-dipped galvanized in accordance with UBC Standard 21-10.

Reinforcement required to resist in-plane shear shall be terminated with a standard hook as defined in Section 2107A.2.2.5 *which encircles the vertical reinforcing* or with an extension of proper embedment length beyond the reinforcement at the end of the wall section. The hook or extension may be turned up, down or horizontally. Provisions shall be made not to obstruct grout placement. Wall reinforcement terminating in columns or beams shall be fully anchored into these elements.

2.2 **Bond.** Multiwythe grouted masonry shear walls shall be designed with consideration of the adhesion bond strength between the grout and masonry units. When bond strengths are not known from previous tests, the bond strength shall be determined by tests.

2.3 **Wall reinforcement.** ***Relocated above.***

2.4 **Stack bond.** ∗ ∗ ∗ Reinforced hollow-unit stacked bond construction which is part of the seismic-resisting system shall use open-end units so that all head joints are made solid, shall use bond beam units to facilitate the flow of grout and shall be grouted solid.

3. **Type N mortar.** Type N mortar shall not be used as part of the vertical- or lateral-load-resisting system.

4. **Concrete abutting structural masonry.** Concrete abutting structural masonry, such as at starter courses or at wall intersections not designed as true separation joints, shall be roughened to a full amplitude of $^1/_{16}$ inch (1.6 mm) and shall be bonded to the masonry in accordance with the requirements of this chapter as if it were masonry. Unless keys or proper reinforcement is provided, vertical joints as specified in Section 2106A.1.4 shall be considered to be stack bond and the reinforcement as required for stack bond shall extend through the joint and be anchored into the concrete.

2106A.2 Working Stress Design and Strength Design Requirements for Unreinforced and Reinforced Masonry.

2106A.2.1 General. In addition to the requirements of Section 2106A.1, the design of masonry structures by the working stress design method and strength design method shall comply with the requirements of this section. Additionally, the design of reinforced masonry structures by these design methods shall comply with the requirements of Section 2106A.3.

2106A.2.2 Specified compressive strength of masonry. The allowable stresses for the design of masonry shall be based on a value of f'_m selected for the construction.

Verification of the value of f'_m shall be based on compliance with Section 2105A.3. Unless otherwise specified, f'_m shall be based on 28-day tests. If other than a 28-day test age is used, the value of f'_m shall be as indicated in design drawings or specifications. Design drawings shall show the value of f'_m for which each part of the structure is designed.

2106A.2.3 Effective thickness.

2106A.2.3.1 Single-wythe walls. The effective thickness of single-wythe walls of either solid or hollow units is the specified thickness of the wall.

2106A.2.3.2 Multiwythe walls. The effective thickness of multiwythe walls is the specified thickness of the wall if the space between wythes is filled with mortar or grout.

2106A.2.3.3 ***Walls and piers.***

Thickness of walls. *For thickness limitations of walls as specified in this chapter, nominal thickness shall be used. Stresses shall be determined on the basis of the net thickness of the masonry, with consideration for reduction, such as raked joints.*

The thickness of masonry walls shall be designed so that allowable maximum stresses specified in this chapter are not exceeded. Also, no masonry wall shall exceed the height or length-to-thickness ratio or the minimum thickness as specified in this chapter and as set forth in Table 21A-R, unless designed in accordance with Section 2108A.2.4.

Piers. *Every pier or wall section which width is less than three times its thickness shall be designed and constructed as required for columns if such pier is a structural member. Every pier or wall section which width is between three and five times its thickness or less than one half the height of adjacent openings shall have all horizontal steel in the form of ties except that in walls 12 inches (305 mm) or less in thickness such steel may be in the form of hairpins.*

2106A.2.3.4 Columns. The effective thickness for rectangular columns in the direction considered is the specified thickness. The effective thickness for nonrectangular columns is the thickness of the square column with the same moment of inertia about its axis as that about the axis considered in the actual column.

2106A.2.4 Effective height. The effective height of columns and walls shall be taken as the clear height of members laterally supported at the top and bottom in a direction normal to the member axis considered. For members not supported at the top normal to the axis considered, the effective height is twice the height of the member above the support. Effective height less than clear height may be used if justified.

2106A.2.5 Effective area. The effective cross-sectional area shall be based on the minimum bedded area of hollow units, or the gross area of solid units plus any grouted area. * * * Where bed joints are raked, the effective area shall be correspondingly reduced. Effective areas for cavity walls shall be that of the loaded wythes.

2106A.2.6 Effective width of intersecting walls. Where a shear wall is anchored to an intersecting wall or walls, the width of the overhanging flange formed by the intersected wall on either side of the shear wall, which may be assumed working with the shear wall for purposes of flexural stiffness calculations, shall not exceed six times the thickness of the intersected wall. Limits of the effective flange may be waived if justified. Only the effective area of the wall parallel to the shear forces may be assumed to carry horizontal shear.

2106A.2.7 Distribution of concentrated vertical loads in walls. The length of wall laid up in running bond which may be considered capable of working at the maximum allowable compressive stress to resist vertical concentrated loads shall not exceed the center-to-center distance between such loads, nor the width of bearing area plus four times the wall thickness. Concentrated vertical loads shall not be assumed to be distributed across continuous vertical mortar or control joints unless elements designed to distribute the concentrated vertical loads are employed. *Structural members framing into or supported by walls or columns shall be securely anchored. The end support of girders, beams or other concentrated loads on masonry shall have at least 3 inches (76 mm) in length upon solid bearing not less than 4 inches (102 mm) thick or upon metal bearing plate of adequate design and dimensions to distribute the loads safely on the wall or pier, or upon a continuous reinforced masonry member projecting not less than 3 inches (76 mm) from the face of the wall or other approved methods.*

Joists shall have bearing at least 3 inches (76 mm) in length upon solid masonry at least $2^1/_2$ inches (64 mm) thick, or other provisions shall be made to distribute safely the loads on the wall or pier.

2106A.2.8 Loads on nonbearing walls. Masonry walls used as interior partitions or as exterior surfaces of a building which do not carry vertical loads imposed by other elements of the building shall be designed to carry their own weight plus any superimposed finish and lateral forces. Bonding or anchorage of nonbearing walls shall be adequate to support the walls and to transfer lateral forces to the supporting elements.

2106A.2.9 Vertical deflection. Elements supporting masonry shall be designed so that their vertical deflection will not exceed $^1/_{600}$ of the clear span under total loads. Lintels shall bear on supporting masonry on each end such that allowable stresses in the supporting masonry are not exceeded. A minimum bearing length of 4 inches (102 mm) shall be provided for lintels bearing on masonry.

2106A.2.10 Structural continuity. Intersecting structural elements intended to act as a unit shall be anchored together to resist the design forces.

2106A.2.11 Walls intersecting with floors and roofs. Walls shall be anchored to all floors, roofs or other elements which provide lateral support for the wall. Where floors or roofs are designed to transmit horizontal forces to walls, the anchorage to such walls shall be designed to resist the horizontal force.

2106A.2.12 Modulus of elasticity of materials.

2106A.2.12.1 Modulus of elasticity of masonry. The moduli for masonry may be estimated as provided below. Actual values, where required, shall be established by test. The modulus of elasticity of masonry.

Modulus of elasticity of clay or shale unit masonry.

$$E_m = 750\,f'_m,\ 3{,}000{,}000 \text{ psi (20.5 GPa) maximum} \quad \text{(6A-3)}$$

Modulus of elasticity of concrete unit masonry.

$$E_m = 750\,f'_m,\ 3{,}000{,}000 \text{ psi (20.5 GPa) maximum} \quad \text{(6A-4)}$$

2106A.2.12.2 Modulus of elasticity of steel.

$$E_s = 29{,}000{,}000 \text{ psi (200 GPa)} \quad \text{(6A-5)}$$

2106A.2.13 Shear modulus of masonry.

$$G = 0.4\,E_m \quad \text{(6A-6)}$$

2106A.2.14 Placement of embedded anchor bolts.

2106A.2.14.1 General. Placement requirements for anchor bolts shall be determined in accordance with this subsection. *Anchor bolts shall be hex headed bolts conforming to ASTM A 307 with the dimensions of the hex head conforming to ANSI/ASME B18.2.1 or plain rod conforming to ASTM A 36 with threaded ends and double hex nuts at the anchored end. Bent bar anchor bolts shall not be used.*

The maximum size anchor shall be $^1/_2$-inch (13 mm) diameter for 6-inch (152 mm) nominal masonry, $^3/_4$-inch (19 mm) diameter for 8-inch (203 mm) nominal masonry, $^7/_8$-inch (22 mm) diameter for 10-inch (254 mm) nominal masonry, and 1-inch (25 mm) diameter for 12-inch (304.8 mm) nominal masonry.

The effective embedment depth l_b for * * * anchor bolts shall be the length of embedment measured perpendicular from the surface of the masonry to the bearing surface of the * * * head of the anchorage. * * * All bolts shall be grouted in place with at least 1 inch (25 mm) of grout between the bolt and the masonry, *and shall be accurately set with templates.*

2106A.2.14.2 Minimum edge distance. The minimum anchor bolt edge distance l_{be} measured from the edge of the masonry parallel with the anchor bolt to the surface of the anchor bolt shall be $1^1/_2$ inches (38 mm).

2106A.2.14.3 Minimum embedment depth. The minimum embedment depth of anchor bolts l_b shall be *eight* bolt diameters but not less than *4 inches (102 mm).*

2106A.2.14.4 Minimum spacing between bolts. The minimum center-to-center distance between anchor bolts shall be *eight* bolt diameters *but not less than 4 inches (102 mm).*

2106A.2.15 ***Not adopted by the State of California.***

2106A.3 Working Stress Design and Strength Design Requirements for Reinforced Masonry.

2106A.3.1 General. In addition to the requirements of Sections 2106A.1 and 2106A.2, the design of reinforced masonry structures by the working stress design method or the strength design method shall comply with the requirements of this section.

HISTORY NOTE APPENDIX

CALIFORNIA BUILDING CODE
(Title 24, Part 2, California Code of Regulations)

For prior history, see the History Note Appendix to the *California Building Code,* 1998 Triennial Edition published in December 1998 and effective July 1, 1999.

1. (DSA/SS 2/01) Adoption of necessary structural safety amendments to the 1998 California Building Code (CCR Title 24, Part 2) for public schools, community colleges and state-owned or state-leased essential service buildings. Approved by the Building Standards Commission on September 25, 2001 and effective on November 1, 2002.

2. (OSHPD 2/01) Adoption of the material and structural standards of the 1997 Uniform Building Code with necessary amendments (CCR, Title 24, Part 2) for hospital buildings and correctional treatment centers. Approved by the Building Standards Commission on September 25, 2001 and effective on November 1, 2002.

3. (HCD 1/01) Adoption of amendments to the California Building Code (CCR, Title 24, Part 2) for hotels, motels, lodging houses, apartment houses, dwellings, employee housing, factory-built housing, and permanent building and accessory buildings in mobile home parks and special occupancy parks. Approved by the Building Standards Commission on November 28, 2001 and effective on November 1, 2002.

4. (SFM 1/01) Adoption of various amendments to the fire and panic safety standards in the California Building Code (CCR, Title 24, Part 2) for State Fire Marshal regulated occupancies. Approved by the Building Standards Commission on November 28, 2001 and effective on November 1, 2002.

5. **Errata October 1, 2002:**

Page 2–1: Delete the words “**Note: This chapter has been revised in its entirety**” from the heading.

Page 2–18: In the last paragraph of **Section 1632.1** revise “[For OSHPD 1]” to “[For OSHPD 2]”.

Page 2–38.12: In **Section 1627A**, under ***APPROVED EXISTING BUILDING***, revise “***[For OSHPD 1, 2 and 4]***” to “***[For OSHPD 1 and 4***]”.

Page 2–38.23: Revise language in **Section 1632A.6**.

Page 2–38.29: Revise **Section 1637A** title to “***SITE DATA FOR HOSPITALS AND STATE OWNED OR STATE-LEASED ESSENTIAL SERVICES BUILDINGS***”.

Page 2–38.45: Revise description to read “This map delineates the boundaries of the seismic hazard zones as given in Section 1629A.4.1 for hospitals and public schools in California”.

Page 2–38.52: Revise item 1. in **Section 1644A.9.2.3.2** to read as follows: “*Chapter 19A, Section 1921A.4, for concrete, and Chapter 22A, Section 2210A, 2211A, items 4 and 5, for steel in structures in .....*”.

Page 2–38.57: Revise **Section 1645A.7.1.3** Item 2. to read “*Nonstructural components, as listed in the 1995 California Building Code, Part 2, Title 24,...*” Revise Item 3. to read “*Equipment listed in the 1995 California Building Code, Part 2, Title 24,...*”

Page 2–38.66: In Table 16A–R–3, for *Site Class E*, in the right column replace the “0” with an “*”. For *Site Class F*, in the left column replace the “0” with an “*”. In Table 16AR–4, for *Site Class E*, in the right column replace the “0” with an “*”.

Page 2–39: Revise title of **Section 1701.4** Item 3. to “**Spray–applied fire–resistive materials.**” Revise title of **Section 1701.5** Item 1.1 to “***[For OSHPD 2] Placing record.***”

Page 2–41: Revise title of **Section 1704.1.2.1** to “***[For HCD 1] Factory-built housing.***”

Page 2–42.2: Revise title of **Section 1704.6.4** Item 17. to ”***Glued-laminated timber.***” Revise Title of Item 18 to “***Post installed anchors.***”

Page 2–96.6: In Section 1809A.5.1, replace “... *Type S3 or S4 soils, ...* ” with “... Type S_D, S_E or S_F soils, ...”

Page 2–184.74: In the last line of Section 1923A, replace “*Section 1916A.4.2.*“ with “*Section l916A.7.1.*”

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

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