OSHPD Office of Statewide Health Planning and Development

Hospital Building Safety Board
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***** SPECIAL NOTICE *****

**Because of the COVID-19 emergency, this meeting will only be held by teleconference. Committee members and members of the public may fully participate from their own locations.**

NOTICE OF PUBLIC MEETING

# HOSPITAL BUILDING SAFETY BOARD

**Structural and Nonstructural Regulations Committee**

**Date:**
Wednesday, March 24, 2021
9:00 a.m. – 3:00 p.m.

**Teleconference Meeting Access:**
HBSB GoToMeeting SNSR Committee
Access Code: 995-836-517

For more detailed instructions on how to join via GoToMeeting, see page 3.

**Committee Members:**
Rami Elhassan, Chair; Jim Malley, Vice-Chair; Bruce Clark; Mike Hooper; David Khorram; Marshall Lew; Michelle Malone*; Michael O'Connor; Jennifer Thornburg

**OSHPD Staff:**
Joe LaBrie; Roy Lobo; David Neou; Carl Scheuerman; Jamie Schnick; Ali Sumer

**OSHPD Director:**
Elizabeth Landsberg

**FDD Deputy Director:**
Paul Coleman

**Executive Director:**
Ken Yu

*Consulting Member

2. **Review and approve the January 27, 2021 draft meeting report/minutes**

Facilitators: Rami Elhassan, Committee Chair (or designee)

- Discussion and public input

3. **Proposed amendments to the 2022 California Building Code, Title 24, Part 2**

Facilitators: Roy Lobo, Chris Tokas, OSHPD (or designee)

- Possible revisions to Chapters 16A, 17A, 18A, 19A, 21A, 22A and 23A to coordinate with the California Division of the State Architect proposed amendments
- Discussion and public input

# Proposed Revisions to the 2022 CBC Part 2 to align with proposed DSA amendments

Roy Lobo, Ph.D., S.E. – Principal Structural Engineer

# Overview of the 2022 CBC OSHPD Amendments

- Adopt the 2021 IBC with continuation of the existing amendments in the 2019 CBC

- Reference standards updated to correspond with those referenced in the 2021 IBC

- Early adoption of some of changes coming in ASCE 7-22 based on ballot proposals that passed

- Amendments to align with DSA's proposed amendments

# Early adoption items from ASCE 7-22

- Adoption of ASCE 7-16 Supplements 1, 2 and 3
- Code changes to the Two Stage analysis procedures
- Code changes to the response modification factors for mixed systems
- Added the exception to the prohibition of lateral force resisting systems with extreme torsional irregularities for the ballot that passed the ASCE 7-22 main committee

# Alignment with DSA's Proposed Amendments

- Early adoption of the two-stage analysis and response modification factors

- Permit currently prohibited systems with type 4 irregularities when designed with redundancy factor of 1.3 and orthogonal load combinations of 100% and 30%

- Editorial revisions

# DSA Proposed Amendments not Adopted

- Moving of specific testing and inspection requirements from the material chapters to Chapter 17A

## 4. Proposed amendments to the 2022 California Building Code, Title 24, Part 10

Facilitators: Roy Lobo, Ali Sumer, OSHPD (or designees)

- Proposed adoption of ASCE 41 Supplement 1 for non SPC-4D buildings
- Discussion and public input

# Proposed Revisions to the 2022 CBSC Part 10 to include ASCE 41-17 Supplement 1

Roy Lobo, Ph.D., S.E. – Principal Structural Engineer

# Why Adopt ASCE 41-17 Supplement 1?

- Proposed ASCE 41-17 Supplement 1 adopts Supplements 1 and 3 of ASCE 7-16 to address the questions related to the creation of the general response spectrum in ASCE 41 for some conditions

- Clarifies the applicability of the expanded requirements for site specific spectra to only apply to the BSE-1N and BSE-2N seismic hazard level

# General Response Spectrum in ASCE 41

- $S_{xs} = F_a S_s$, Design short period spectral response acceleration parameter
- $S_{x1} = F_v S_1$, Design spectral response acceleration parameter at 1 second

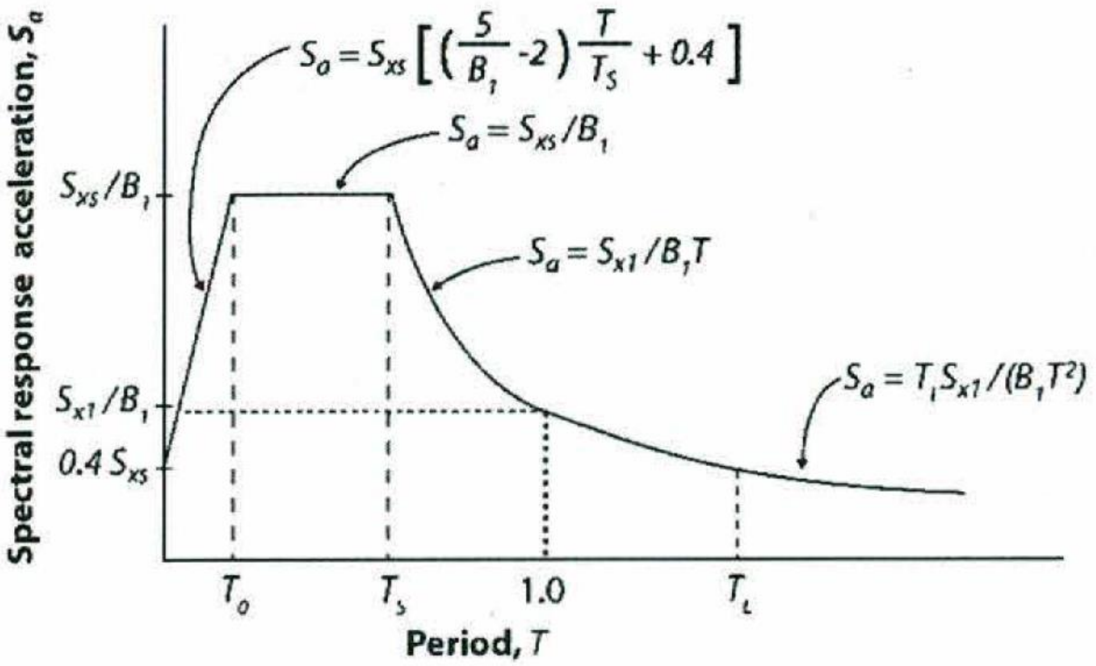


Figure 2-1. General Horizontal Response Spectrum



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# 2.4 Seismic Hazard (ASCE 41-17)

- The site-specific procedure shall be used where any of the following conditions apply:

1. The building is located on Site Class E soils, and the mapped spectral response acceleration at short periods($S_{xs}$) exceeds 2.0;
2. The building is located on Site Class F soils; or
3. The BSE-2N or BSE-1 N hazard parameters are determined and Section 11 .4.8~~7~~ of ASCE 7 requires that site-specific ground motion procedure be used to determine the $MCE_R$ parameters.

# Issues in the Creation of the General Response Spectrum

- General response spectrum cannot be created for the BSE-1E and BSE-2E on Site Class E without the use of site-specific procedures because there is no $F_v$ value for Site Class E. (Resolved in ASCE 7-16 Supplement 1)

- **$F_v$ values are provided in ASCE 7-16 Supplement 1.**

# Clarification of the 1.5 amplification factor applied to $C_s$

- Original exception in ASCE 7-16 required $C_s$ to be multiplied by 1.5. Therefore, the exception in ASCE 7-16 for site class D and E to performing a site-specific ground motion hazard analysis is not applicable without amplifying the BSE-1N and BSE-2N $S_{X1}$ parameter by 1.5. (Resolved in ASCE 7-16 Supplement 3)

- **Supplement 3 revises the exemption to require the $S_{D1}$ parameter be multiplied by 1.5.**

5. **Comments from the Public/Committee Members on issues not on this agenda**

Facilitator: Rami Elhassan, Committee Chair (or designee)

The Committee will receive comments from the Public/Committee Members. Matters raised at this time may be taken under consideration for placement on a subsequent agenda.

**INITIAL EXPRESS TERMS**
**FOR PROPOSED BUILDING STANDARDS**
**OF THE OFFICE OF STATEWIDE HEALTH PLANNING AND DEVELOPMENT**
**REGARDING THE 2022 CALIFORNIA BUILDING CODE**
**CALIFORNIA CODE OF REGULATIONS, TITLE 24, PART 2, VOLUME 2**
**(OSHPD 0x/xx)**

**LEGEND for EXPRESS TERMS (Based on model codes - Parts 2, 2.5, 3, 4, 5, 9, 10)**

1. Model Code language appears upright
2. Existing California amendments appear in *italic*
3. Amended model code or new California amendments appear *underlined & italic*
4. Repealed model code language appears ~~upright and in strikeout~~
5. Repealed California amendments appear in *~~italic and strikeout~~*
6. Ellipsis (...) indicate existing text remains unchanged
7. Amended model code or new California amendment by DSA to DSA/OSHPD common language appear in green font.
8. California amendment by DSA to DSA/OSHPD not common language or in review appear in yellow highlight and green font.

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**INITIAL EXPRESS TERMS**

**Item: #1**

**Chapter: CHAPTER 16A – STRUCTURAL DESIGN**

…

**Section: 1605A – LOAD COMBINATIONS**

…

**1605.*A*.2 (Formerly 1605A.3.2) Alternative allowable stress design load combinations.**

In lieu of the load combinations in ASCE 7, Section 2.4, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following combinations. Where using these alternative allowable stress load combinations that include wind or seismic *loads*, allowable stresses are permitted to be increased or load combinations reduced where permitted by the material chapter of this code or the referenced standards. For load combinations that include the counteracting effects of dead and wind *loads*, only two-thirds of the minimum *dead load* likely to be in place during a design wind event shall be used. Where using these alternative load combinations to evaluate sliding, overturning and soil bearing at the soil-structure interface, the reduction of foundation overturning from Section 12.13.4 in ASCE 7 shall not be used. Where using these alternative basic *load* combinations for proportioning foundations for loadings, which include seismic *loads*, the vertical seismic *load effect*, $E_v$, in Equation 12.4-4 of ASCE 7 is permitted to be taken equal to zero. Where required by ASCE 7, Chapters 12, 13 and 15, the load combinations including overstrength of ASCE 7, Section 2.3.6 shall be used. *Each load combination shall be investigated with one or more of the variable loads set to zero.*

...

***~~1605A.4~~ 1605A.3 Modifications to load combinations in ICC 300.*** *Modify the text of ICC 300~~,~~ as follows:*

> ***1605A.3.1 ICC 300, Section 303.5.2.*** *Modify Section 303.5.2 by adding Equation 3-5a as follows:*
>
> $D + 0.4L + Z$ *(Equation 3-5a)*
>
> ***~~1605A.4.3~~ 1605A.3.2 ICC 300, Section 303.5.3.*** *Modify Section 303.5.3 as follows:*

*The uniform live load L used in Equation 3-2 and 3-4 may be taken as zero when*

…

**SECTION 1606A – DEAD LOADS**

...

***1606A.~~3~~6 Roof dead loads.*** *The design dead load shall provide for the weight of at least one additional roof covering in addition to other applicable loadings if the new roof covering is permitted to be applied over the original roofing without its removal, in accordance with Section ~~1511~~ 1512.*

...

**SECTION 1606A – LIVE LOADS**

...

> **~~1607.13.5.4~~1607.14.4.5 Ballasted photovoltaic panel systems.** Roof structures that provide support for ballasted photovoltaic panel systems shall be designed, or analyzed, in accordance with **{ Section 1604.4 }**; checked in accordance with **{ Section 1604.3.6 }** for deflections; and checked in accordance with **{ Section 1611 }** for ponding.
>
> ***~~1607A.13.6~~ 1607A.14.5 Uncovered open-frame roof structures.*** *Uncovered open-frame roof structures shall be designed for a vertical live load of not less than 10 pounds per square foot (0.48 kN/m²) of the total area encompassed by the framework.*

...

## SECTION: 1613A - EARTHQUAKE LOADS

…

## TABLE 1613A.2.3(1) - VALUES OF SITE COEFFICIENT $F_a$[a]

| SITE CLASS | MAPPED RISK TARGETED MAXIMUM CONSIDERED EARTHQUAKE ($MCE_R$) SPECTRAL RESPONSE ACCELERATION PARAMETER AT SHORT PERIOD | | | | | |
|---|---|---|---|---|---|---|
| | $S_s \leq 0.25$ | $S_s = 0.50$ | $S_s = 0.75$ | $S_s = 1.00$ | $S_s = 1.25$ | $S_s \geq 1.5$ |
| A | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| B | 0.9 | 0.9 | 0.9 | 0.9 | 0.9 | 0.9 |
| C | 1.3 | 1.3 | 1.2 | 1.2 | 1.2 | 1.2 |
| D | 1.6 | 1.4 | 1.2 | 1.1 | 1.0 | 1.0 |
| E | 2.4 | 1.7 | 1.3 | ~~Note b~~ *1.2*[c] | ~~Note b~~ *1.2*[c] | ~~Note b~~ *1.2*[c] |
| F | Note b | Note b | Note b | Note b | Note b | Note b |

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, $S_S$.
b. Values shall be determined in accordance with Section 11.4.8 of ASCE 7.
*c. See requirements for site-specific ground motions in Section 11.4.8 of ASCE 7. These values of $F_a$ shall only be used for calculation of $T_s$, determination of Seismic Design Category, linear interpolation for intermediate values of $S_s$, and when taking the exception under Item 2 within Section 11.4.8 of ASCE 7.*

## TABLE 1613A.2.3(2) - VALUES OF SITE COEFFICIENT $F_v$[a]

...

c. See requirements for site-specific ground motions in Section 11.4.8 of ASCE 7. *These values of $F_v$ shall only be used for calculation of $T_S$, determination of Seismic Design Category, linear interpolation for intermediate values of $S_1$, and when taking the exceptions under Items 1 and 2 of Section 11.4.8 for the calculation of $S_{D1}$.*

## Item: #2

## Chapter: CHAPTER 16A – STRUCTURAL DESIGN

...

## SECTION: 1617A - MODIFICATIONS TO ASCE 7

…

### *1617A.1.3 ~~Reserved~~ ASCE 7, Section 11.4. Modify ASCE 7, Section 11.4 to include the following:*

*Seismic ground motion values shall include updated subsections in Supplement 3.*

***[OSHPD 1 & 4]*** *Use of the 2020 NEHRP Provisions for multi-period spectra shall be permitted, where all of the following are included.:*

1. *A detailed seismic design criterion shall be submitted to and approved by the AHJ.*
2. *Seismic Ground Motion values shall be determined using the 2020 NEHRP Provisions, Section 11.4.*
3. *Geologic Hazard and Geotechnical Investigation shall be performed using the 2020 NEHRP Provisions, Section 11.8.*
4. *Vertical Ground Motions, where required, shall be determined using the 2020 NEHRP Provisions, Section 11.9.*
5. *Site Classification shall be determined using the 2020 NEHRP Provisions, Chapter 20.*
6. *Site Specific Ground Motion Procedures shall be determined using the 2020 NEHRP Provisions, Chapter 21.*
7. *Seismic Ground Motion and Long-period Transition Maps shall be used from Chapter 22 of the 2020 NEHRP Provisions.*
8. *$S_{DS}$ and $S_{D1}$ obtained from the multiperiod spectra determined using the 2020 NEHRP Provisions shall be used, where required in Chapter 12, 13 and 15 of ASCE 7-16.*

…

(Relocated to 1617A.1.5.2)~~***1617A.1.5 ASCE 7, Section 12.2.3.1.*** *Replace ASCE 7, Section 12.2.3.1, Items 1 and 2, by the following:*~~

~~*The value of the response modification coefficient, R, used for design at any story shall not exceed the lowest value of R that is used in the same direction at any story above that story. Likewise, the deflection amplification factor, $C_d$, and the system over strength factor, $\Omega_0$ , used for the design at any story shall not be less than the largest value of these factors that are used in the same direction at any story above that story.*~~

***1617A.1.5 ASCE 7, Section 12.2.3, 12.2.3.1, and 12.2.3.2.*** *Modify ASCE 7, Sections 12.2.3, 12.2.3.1, and 12.2.3.2 as follows:*

***1617A.1.5.1 ASCE 7, Section 12.2.3.*** *Replace ASCE 7, Section 12.2.3 with the following:*

*Where different seismic force-resisting systems are used in combinations to resist seismic forces in the same direction, other than those combinations considered as dual systems the design shall comply with the requirements of this section. The most stringent applicable structural system limitations contained in Table 12.2-1 shall apply, except as otherwise permitted by this section.*

(Relocated from 1617A.1.5) ***1617A.1.5.2 ASCE 7, Section 12.2.3.1.*** *Replace ASCE 7, Section 12.2.3.1, Items 1 and 2, by the following:*

*The value of the response modification coefficient, R, used for design at any story shall not exceed the lowest value of R that is used in the same direction at any story above that story. Likewise, the deflection amplification factor, $C_d$, and the system over strength factor, $\Omega_0$, used for the design at any story shall not be less than the largest value of these factors that are used in the same direction at any story above that story.*

***1617A.1.5.3 ASCE 7, Section 12.2.3.2.*** *Modify ASCE 7, Section 12.2.3.2 by replacing Item a and adding Items f, g, and h, as follows:*

**12.2.3.2 Two-Stage Analysis Procedure**. A two-stage equivalent lateral force procedure is permitted to be used for structures that have a flexible upper portion above a rigid lower portion, provided that the design of the structure complies with all of the following:

a. The stiffness of the lower portion shall be at least 10 times the stiffness of the upper portion. *For purposes of determining this ratio, the base shear shall be computed and distributed vertically according to Section 12.8. Using these forces, the stiffness for each portion shall be computed as the ratio of the base shear for that portion to the elastic displacement, $\delta_{xe}$, computed at the top of that portion, considering the portion fixed at its base. For the lower portion, the applied forces shall include the reactions from the upper portion, modified as required in Item d.*

b. The period of the entire structure shall not be greater than 1.1 times the period of the upper portion considered as a separate structure supported at the transition from the upper to the lower portion.

c. The upper portion shall be designed as a separate structure using the appropriate values of R and ρ.

d. The lower portion shall be designed as a separate structure using the appropriate values of R and ρ. The reactions from the upper portion shall be those determined from the analysis of the upper portion amplified by the ratio of the R/ρ of the upper portion over R/ρ of the lower portion. This ratio shall not be less than 1.0.

e. The upper portion is analyzed with the equivalent lateral force or modal response spectrum procedure, and the lower portion is analyzed with the equivalent lateral force procedure.

***f.*** *The structural height of the upper portion shall not exceed the height limits of Table 12.2-1 for the seismic force-resisting system used, where the height is measured from the base of the upper portion.*

*g. Where Horizontal Irregularity Type 4 or Vertical Irregularity Type 4 exists at the transition from the upper to the lower portion, the reactions from the upper portion shall be amplified in accordance with Sections 12.3.3.3, 12.10.1.1, and 12.10.3.3 as applicable, in addition to amplification required by Item d.*

*h.* (Relocated from 1617A.1.6) *Where design of vertical elements of the upper portion is governed by special seismic load combinations, the special loads shall be considered in the design of the lower portion.*

(Relocated to 1617A.1.5.3, Item h) ***1617A.1.6 ~~ASCE 7, Section 12.2.3.2.~~*** *~~Modify ASCE 7 Section 12.2.3.2 by adding the following additional requirement:~~* ***Reserved.***

*~~f. Where design of vertical elements of the upper portion is governed by special seismic load combinations, the special loads shall be considered in the design of the lower portion.~~*

...

***1617A.1.10 ASCE 7, Section 12.3.3.1.*** *Modify first sentence of ASCE 7 Section 12.3.3.1 as follows:*

> ***12.3.3.1 Prohibited Horizontal and Vertical Irregularities for Seismic Design Categories D through F.*** *Structures assigned to Seismic Design Category D, E, or F having horizontal structural irregularity Type 1b of Table 12.3-1 or vertical structural irregularities Type 1b, 5a or 5b of Table 12.3-2 shall not be permitted.*
>
> ***~~Exception~~ Exceptions:***
>
> *1. Structures with reinforced concrete or reinforced masonry shear wall systems and rigid or semi-rigid diaphragms, consisting of concrete slabs or concrete-filled metal deck having a span-to-depth ratio of 3 or less, having a horizontal structural irregularity Type 1b of Table 12.3-1 are permitted, provided the maximum story drift in the direction of the irregularity, computed including the torsional amplification factor from Section 12.8.4.3, is less than 10% of the allowable story drift in ASCE 7 Table 12.12-1.*
>
> *2. Structures having a horizontal structural irregularity Type 1b of Table 12.3-1 are permitted, provided a redundancy factor, $\rho$, of 1.3 as defined in ASCE 7 12.3.4 is assigned to the seismic force-resisting system in both orthogonal directions and the structure is designed for one of the orthogonal procedures as defined in ASCE 7 12.5.3.1.*

...

***1617A.1.14 ~~[Reserved]~~ ~~1617A.1.15~~ ASCE 7, Section 12.12.3. [OSHPD 1 & 4]***

*Replace ASCE 7 Equation 12.12-1 by the following:*

$\delta_M = C_d \delta_{max}$ *(Equation 12.12-1)*

***~~1617A.1.16~~ 1617A.1.15 ASCE 7, Section 12.13.1.*** *Modify ASCE 7 section 12.13.1 by adding Section 12.13.1.1 as follows:*

***12.13.1.1 Foundations and superstructure-to-foundation connections.*** *The foundation shall be capable of transmitting the design base shear and the overturning forces from the structure into the supporting soil. Stability against overturning and sliding shall be in accordance*

*with Section 1605A.1.1.*

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

*1. The strength of the superstructure elements.*

*2. The maximum forces that can be delivered to the foundation in a fully yielded structural system.*

*3. Forces from the load combinations with overstrength factor in accordance with ASCE 7, Section 12.4.3.1. 11*

***Exceptions:***

*1. Where referenced standards specify the use of higher design loads.*

*2. When it can be demonstrated that inelastic deformation of the foundation and superstructure-*

*to foundation connection will not result in a weak story or cause collapse of the structure.*

*3. Where seismic force-resisting system consists of light framed walls with shear panels, unless*

*the reference standard specifies the use of higher design loads.*

*Where the computation of the seismic overturning moment is by the equivalent lateral-force method or the modal analysis method, reduction in overturning moment permitted by section 12.13.4 of ASCE 7 may be used.*

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

***1617A.1.16 ASCE 7, Section 12.13.9.2.*** *Modify ASCE 7 section 12.13.9.2 by the following sentence added to the end of item b as follows:*

12.13.9.2 Shallow Foundations. Building structures shall be permitted to be supported on shallow foundations provided that the foundations are designed and detailed in accordance with Section 12.13.9.2.1 and the conditions provided in items (a) and (b) of Section 12.13.9.2 are met.

a. The geotechnical investigation report indicates that permanent horizontal ground displacement induced by lateral spreading associated with MCEG earthquake motions does not exceed the value in Table 12.13-2.
b. The foundation and superstructure are designed to accommodate differential settlements caused by liquefaction without loss of the ability to support gravity loads. For structures assigned to Risk Category II or III, residual strength of members and connections shall not be less than 67% of the undamaged nominal strength, considering the nonlinear behavior of the structure or, alternatively, demands on all members and connections shall not exceed the element's nominal strength when subjected to differential settlements. For structures assigned to Risk Category IV, demands on all members and connections shall not exceed the element's nominal strength when subjected to differential settlements. *Seismic load effects determined in accordance with Section 12.4 need not be considered in this check.*

…

***1617A.1.18 ASCE 7, Section 13.1.4.*** *Replace ASCE 7, Section 13.1.4, with the following: [**DSA-SS, for OSHPD see Section 13.1.4.a**]*

*…*

***13.1.4.a [OSHPD 1, ~~1R,~~ 2, 4 & 5].*** *The following nonstructural components and equipment shall be anchored in accordance with this section. Design and detailing shall be in accordance with Chapter 13 except as modified by this section.*

*…*

*10. Wall/Roof or Floor Hung Equipment: Seismic design and seismic details shall be provided for wall, roof or floor hung nonstructural components and equipment when the component weighs more than 20 lb or, in the case of a distributed system, more than 5 lb/ft.*

*…*

...

***1617A.1.26 ASCE 7, Section 13.6.7.3.*** *Replace ASCE 7, Section 13.6.7.3 with the following:*

***13.6.7.3 Additional provisions for piping and tubing systems.***

*A) Design for the seismic forces of Section 13.3 shall not be required for piping systems where flexible connections, expansion loops, or other assemblies are provided to accommodate the relative displacement between component and piping, where the piping system is positively attached to the structure, and where any of the following conditions apply:*

1. *Trapeze assemblies are supported by 3/8-inch (10 mm) or ½-inch (13 mm) diameter rod hangers not exceeding 12 inches (305 mm) in length from the pipe support point to the connection at the supporting structure, ~~do not support piping with Ip greater than 1.0,~~ and no single pipe exceeds the diameter limits set forth in item 2b below or 2 inches (50 mm) ~~for Seismic Design Category D, E, or F~~ where $I_p$ is greater than 1.0 and the total weight supported by any single trapeze is 100 pounds (445 N) or less; or*

…

**Item: #3**

**Chapter: CHAPTER 17A**

**SPECIAL INSPECTIONS AND TESTS**

**…**

## Section: 1705A, REQUIRED SPECIAL INSPECTIONS AND TESTS

…

***1705A.2.5 Inspection and tests of structural welding.*** *Inspection and testing (including non-destructive testing) of all shop and field welding operations shall be in accordance with this section, Section 1705A.2.1, and Table 1705A.2.1. Inspections shall be made by a qualified welding inspector approved by the enforcement agency. The minimum requirements for a qualified welding inspector shall be as those for an AWS Certified Welding Inspector (CWI), as defined in the provisions of the AWS QC1.*

***[DSA-SS, DSA-SS/CC]*** *Welding inspector approval by the enforcement agency shall occur when specified in the California Administrative Code. Nondestructive testing shall be performed by qualified NDT Level II personnel employed by the approved agency.*

*The welding inspector shall make a systematic daily record of all welds. In addition to other records, this record shall include:*

1. *Identification marks of welders.*
2. *List of defective welds.*
3. *Manner of correction of defects.*

*The welding inspector shall check the material, details of construction and procedure, as well as workmanship of the welds. The inspector shall verify that the installation and tests of end-welded stud shear connectors is in accordance with the requirements of AWS D1.1, Sections 7.7 and 7.8 ~~2213A.2 ([DSA-SS/CC] 2212.6.2)~~ and the approved plans and specifications. The* *approved agency shall furnish the architect, structural engineer, and the enforcement agency with a verified report that the welding has been done in conformance with AWS D1.1, D1.3, D1.4, D1.8, and the approved construction documents.*

***1705A.2.6 Special inspection and tests of high-strength fastener assemblies.*** *Special inspections and tests for high-strength fasteners shall be in accordance with this section, Section 1705A.2.1, and Table 1705A.2.1. ~~Tests of h~~High-strength bolts, nuts, and washers shall be sampled and tested by an approved agency for conformance with the requirements of applicable ASTM standards~~in accordance with Section 2213A.1 ([DSA-SS/CC] 2212.6.1)~~.*

…

**TABLE 1705*A*.3**

**REQUIRED SPECIAL INSPECTIONS AND TESTS OF CONCRETE CONSTRUCTION**

| TYPE | CONTINUOUS SPECIAL INSPECTION | PERIODIC SPECIAL INSPECTION | REFERENCED STANDARD[a] | ~~IBC~~*CBC* REFERENCE |
|---|---|---|---|---|
| 1. Inspect *and test* reinforcement, including prestressing tendons, and verify placement.<br>*a. Reinforcement in special moment frames, boundary elements of special structural wall, and coupling beams.*<br>*b. All other reinforcement* | <br>X<br>– | <br>–<br>X | ACI 318: Ch. 20, 25.2, 25.3, *25.5.1,* 26.6.1- 26.6.3, *26.13.1, 26.13.3.2, 26.13.3.3* | *~~1908A.3, 1908A.4, 1910A.2,~~ 1705A.3.2.2 ~~1910A.3;~~ 1705A.3.4.1*<br>*~~[DSA-SS/CC] 1909.2.4, 1909.2.5~~*<br>***OSHPD[ 1 & 4]*** |
| 2. Reinforcing bar welding:<br>a. Verify weldability of reinforcing bars other than ASTM A706. | – | X | (Relocated ACI 318 references 18.2.8, 25.5.7 from Table 1705A.2.1 item b1 and b2.)<br>AWS D1.4 | *1705A.3.1, 1903A.8* |

| | | | | |
|---|---|---|---|---|
| b. Inspect single pass fillet welds, maximum 5/16”, *not defined in 2.d or 2.e.* ~~and~~<br><br>c. Inspect all other welds.<br><br>(Relocated from Table 1705A.2.1 items 1 and 2 to new items d and e.)<br><br>*d. Reinforcing steel resisting flexural and axial forces in intermediate and special moment frames, and boundary elements and coupling beams of special structural walls of concrete and shear reinforcement.*<br><br>*e. Shear reinforcement.* | –<br><br>X<br><br>*X*<br><br>*X* | X<br><br>–<br><br>–<br><br>– | ACI 318: *18.2.8, 25.5.7*, 26.6.4,<br><br>*26.13.1.4, 26.13.3.2, 26.13.3.3* | |
| 3. Inspect anchors cast in concrete. | – | X | ACI 318: 17.8.2, *26.7.2, 26.8.2, 26.13.1, 26.13.3.3* | – |
| 4. Inspect *and test* anchors post-installed in hardened concrete members.[b, c]<br><br>a. Adhesive anchors installed horizontally or upwardly inclined | X | – | | |

| | | | | |
|---|---|---|---|---|
| orientations to resist sustained tension loads.<br><br>b. Mechanical anchors and adhesive anchors not defined in 4.a. | – | X | ACI 318: 17.8.2.4 26.7.2, 26.13.1, 26.13.3.2<br><br>ACI 318: 17.8.2 26.7.2, 26.13.1, 26.13.3.3 | *1705A.3.8~~, 1910A.5,~~*<br><br>*~~[DSA-SS/CC] 1909.2.7~~*<br><br>*~~1705A.3.8,~~ ~~1910A.5,~~*<br><br>*~~[DSA-SS/CC] 1909.2.7~~* |
| 5. Verify use of required design mix. | ~~–~~X | ~~X~~– | ACI 318: Ch.19, *26.4*, ~~26.4.3, 26.4.4~~ *26.13.3.2* | *1903A.5, 1903A.6, 1903A.7*, 1904*A*.1, 1904*A*.2, *~~1908A.2, 1908A.3, 1910A.1,~~ 1705A.3.2.1, [DSA-SS/CC] 1909.2.1, 1909.2.2~~, 1909.2.3~~* |
| 6. Prior to *and during* concrete placement, fabricate specimens for strength tests, perform slump and air content tests, and determine the temperature of the concrete. | X | – | ASTM C31<br><br>ASTM C172<br><br>ACI 318: *26.4,* 26.5, 26.12 | *1705A.3.5, 1705A.3.6, 1705A.3.9, 1905A.1.16, ~~1908A.5, 1908A.10,~~ [DSA-SS/CC] ~~1908.5,~~ 1909.3.~~7~~9,~~1908.10, 1909.4.1~~* |
| 7. Inspect concrete and shotcrete for proper application techniques. | X | – | ACI 318: 26.5, *26.13*<br><br>*ACI 506: 3.4* | *1705A.3.9 ~~1908A.5, 1908A.6, 1908A.7, 1908A.8, 1908A.10, 1908A.12, [DSA-SS/CC] 1909.4.5~~* |
| 8. Verify maintenance of specific curing temperature and techniques. | – | X | ACI 318: 26.5.3–26.5.5, *26.13.3.3* | *~~1908A.9,~~*<br><br>– |
| 9. Inspect prestressed concrete for:<br><br>a. Application of prestressing forces; and | X | – | ACI 318: 26.10.2, *26.13.1, 26.13.3.2* | *1705A.3.4* |

| | | | | |
|---|---|---|---|---|
| b. Grouting of bonded prestressing tendons. | X | — | | |
| 10. Inspect erection of precast concrete members. | — | X | ACI 318: 26.9.2, *26.13.1, 26.13.3.3* | — |
| 11. For precast concrete diaphragm connections or reinforcement at joints classified as moderate or high deformability elements (MDE or HDE) in structures assigned to Seismic Design Category ~~C,~~ D, E or F, inspect such connections and reinforcement in the field for:<br>a. Installation of the embedded parts<br>b. Completion of the continuity of reinforcement across joints.<br>c. Completion of connections in the field. | <br>X<br>X<br>X | <br>—<br>—<br>— | ACI 318: 26.13.1.3<br><br>ACI 550.5 | — |
| 12. Inspect installation tolerances of precast concrete diaphragm connections for compliance with ACI 550.5. | — | X | ACI 318: 26.13.1.3 | — |

| | | | | |
|---|---|---|---|---|
| 13. Verify in-situ concrete strength, prior to stressing of tendons in post-tensioned concrete and prior to removal of shores and forms from beams and structural slabs. | – | X | ACI 318: *26.10.2,* 26.11.2, *26.13.3.3* | ~~*1911A.1, [DSA-SS/CC] 1909.5,*~~ |
| 14. Inspect formwork for shape, location and dimensions of the concrete member being formed | – | X | ACI 318: 26.11.1.2(b), *26.13.3.3* | *1908A.~~11~~3, [DSA-SS/CC] 1909.4.~~4~~3* |

…

…

***1705A.5.~~4~~5 Structural glued laminated and cross-laminated timber.*** *Manufacture of all structural glued laminated and cross-laminated timber shall be continuously inspected by an approved agency.*

*The approved agency shall verify that proper quality control procedures and tests have been employed for all materials and the manufacturing process, and shall perform visual inspection of the finished product. Each inspected member shall be stamped by the approved agency with an identification mark.*

> ***Exception:*** *Special Inspection is not required for non-custom prismatic glued laminated members identified on drawings and sourced from stock or general inventory of 5 1/2-inch maximum width and 18-inch maximum depth, and with a maximum clear span of 32 feet, manufactured and marked in accordance with ANSI~~/APA~~ A190.1 Section 13.1 for non-custom members.*

…

**…**

***1705A.6.~~2~~3 Earth retaining shoring.*** *Special inspections and tests of earth retaining shoring shall be in accordance with applicable portions of Section 1812A~~.~~* (Relocated from Section 1812A.6, Item #2) *Testing, inspection and observation shall be in accordance with testing, inspection and observation requirements approved by the building official. The following activities and materials shall be tested, inspected, or observed by the special inspector and geotechnical engineer:*

1. *~~a.~~ Sampling and testing of concrete in soldier pile and tie-back anchor shafts.*
2. *~~b.~~ Fabrication of tie-back anchor pockets on soldier beams*
3. *~~c.~~ Installation and testing of tie-back anchors.*
4. *~~d.~~ Survey monitoring of soldier pile and tie-back load cells.*

*5. ~~e.~~ Survey monitoring of existing buildings.*

…

**Item: #4**

**Chapter: CHAPTER 18A**

**SOILS AND FOUNDATIONS**

…

**SECTION 1807A – FOUNDATION WALLS, RETAINING WALLS AND EMBEDDED POSTS AND POLES**

…

***1807A.2.45 Freestanding cantilever walls.*** *Freestanding cantilever walls shall comply with Section 15.6.8 of ASCE 7.* ***[OSHPD 1 & 4]*** *A stability check against the possibility of overturning shall be performed for isolated spread footings which support freestanding cantilever walls. The stability check shall be made by dividing R~~p~~ used for the wall by ~~2.0~~1.25. The allowable soil pressure may be doubled for this evaluation.*

> ***Exception:*** *For overturning about the principal axis of rectangular footings with symmetrical vertical loading and the design lateral force applied, a triangular or trapezoidal soil pressure distribution which covers the full width of the footing will meet the stability requirement.*

…

**SECTION 1808A - FOUNDATIONS**

…

**1808A.8.2 Concrete cover.**
The concrete cover provided for prestressed and nonprestressed reinforcement in foundations shall be not less than the largest applicable value specified in { Table 1808A.8.2 }. Longitudinal bars spaced less than 11/2 inches (38 mm) clear distance apart shall be considered to be bundled bars for which the concrete cover provided shall be not less than that required by Section 20.5.1.3.5 ~~20.6.1.3.4~~ of { ACI 318 }. Concrete cover shall be measured from the concrete surface to the outermost surface of the steel to which the cover requirement applies. Where concrete is placed in a temporary or permanent casing or a mandrel, the inside face of the casing or mandrel shall be considered to be the concrete surface.

**TABLE 1808A.8.2**
**MINIMUM CONCRETE COVER**

| FOUNDATION ELEMENT OR CONDITION | MINIMUM COVER |
| --- | --- |

| | |
|---|---|
| 1. Shallow foundations | In accordance with Section 20.~~6~~5 of { ACI 318 } |
| 2. Precast nonprestressd deep foundation elements<br>Exposed to seawater<br>Not manufactured under plant conditions<br>Manufactured under plant control conditions | <br>3 inches<br>2 inches<br>In accordance with Section 20.~~6~~5.1.3.~~3~~4 of { ACI 318 } |
| 3. Precast prestressed deep foundation elements<br>Exposed to seawater<br>Other | <br>2.5 inches<br>In accordance with Section 20.~~6~~5.1.3.~~3~~ 4of { ACI 318 } |
| 4. Cast-in-place deep foundation elements not enclosed by a steel pipe, tube or permanent casing | 2.5 inches |
| 5. Cast-in-place deep foundation elements enclosed by a steel pipe, tube or permanent casing | 1 inch |
| 6. Structural steel core within a steel pipe, tube or permanent casing | 2 inches |
| 7. Cast-in-place drilled shafts enclosed by a stable rock socket | 1.5 inches |

In accordance with Section 20.6.1.3.3 of { ACI 318 }
For SI: 1 inch = 25.4 mm.

…

**Item: #5**

## Chapter: CHAPTER 18A

## SOILS AND FOUNDATIONS

…

### SECTION 1810A – DEEP FOUNDATIONS

…

**1810A.3.3.1.9 Helical piles.**

The allowable axial design load, $P_a$, of helical piles shall be determined as follows:

$$P_a = 0.5\ P_u$$

where $P_u$ is the least value of:

1. Base capacity plus shaft resistance of the helical pile. The base capacity is equal to the sum of the areas of the helical bearing plates times the ultimate bearing capacity of the soil or rock comprising the bearing stratum. The shaft resistance is equal to the area of the shaft above the uppermost helical bearing plate times the ultimate skin resistance.
2. Ultimate capacity determined from well-documented correlations with installation torque.
3. Ultimate capacity determined from load tests ~~where required by { Section 1810.3.3.1.2 }.~~
4. Ultimate axial capacity of pile shaft.
5. Ultimate axial capacity of pile shaft couplings.
6. Sum of the ultimate axial capacity of helical bearing plates affixed to pile.

…

**1810*A*.3.5.3.3 Structural steel sheet piling.** Individual sections of structural steel sheet piling shall conform to the profile indicated by the manufacturer and shall conform to the general requirements specified by ASTM A6.

*Installation of sheet piling shall satisfy inspection, monitoring, and observation requirements in Sections ~~1705A.6.2,~~ 1812A.6 and 1812A.7.*

…

**1810*A*.3.6 Splices.**
Splices shall be constructed so as to provide and maintain true alignment and position of the component parts of the deep foundation element during installation and subsequent thereto and shall be designed to resist the axial and shear forces and moments occurring at the location of the splice during driving and for design load combinations. Where deep foundation elements of the same type are being spliced, splices shall develop not less than 50 percent of the bending strength of the weaker section. Where deep foundation elements of different materials or different types are being spliced, splices shall develop the full compressive strength and not less than 50 percent of the tension and bending strength of the weaker section. Where structural steel cores are to be spliced, the ends shall be milled or ground to provide full contact and shall be full-depth welded.

> ~~Exception: For buildings assigned to Seismic Design Category A or B, splices need not comply with the 50-percent tension and bending strength requirements where justified by supporting data.~~

Splices occurring in the upper 10 feet (3048 mm) of the embedded portion of an element shall be designed to resist at allowable stresses the moment and shear that would result from an assumed eccentricity of the axial load of 3 inches (76 mm), or the element shall be braced in accordance with **{ Section 1810.2.2 }** to other deep foundation elements that do not have splices in the upper 10 feet (3048 mm) of embedment.

**…**

***1810A.3.8 Precast concrete Plies.***

1810.3.8 Precast concrete piles.
Precast concrete piles shall be designed and detailed in accordance with {ACI 318}.

~~Exceptions:~~

1. ~~For precast prestressed piles in Seismic Design Category C, the minimum volumetric ratio of spirals or circular hoops required by Section 18.13.5.10.4 of { ACI 318 } shall not apply in cases where the design includes full consideration of load combinations specified in { ASCE 7 }, Section 2.3.6 or Section 2.4.5 and the applicable overstrength factor, $\Omega_0$. In such cases, minimum transverse reinforcement index shall be as specified in Section 13.4.5.6 of { ACI 318 }.~~
2. ~~For precast prestressed piles in Seismic Design Categories D through F, the minimum volumetric ratio of spirals or circular hoops required by Section 18.13.5.10.5(c) of { ACI 318 } shall not apply in cases where the design includes full consideration of load combinations specified in { ASCE 7 }, Section 2.3.6 or Section 2.4.5 and the applicable overstrength factor, Ω0. In such cases, minimum transverse reinforcement shall be as specified in Section 13.4.5.6 of { ACI 318 }.~~

   ***Exception:*** *Where the axial load from seismic forces is amplified by the applicable overstrength factor, $\Omega_0$ the axial load limits Section 18.13.5.10.6 of ACI 318 may be increased by two times.*

~~1810.3.8.1 Reinforcement.~~
~~Longitudinal steel shall be arranged in a symmetrical pattern and be laterally tied with steel ties or wire spiral spaced center to center as follows:~~

…

**1810*A*.3.9.4.1 Seismic reinforcement in Seismic Design Category C.** *Not permitted by DSA-SS, DSA-SS/CC or OSHPD.* ~~For structures assigned to Seismic Design Category C, cast-in-place deep foundation elements shall be reinforced as specified in this section. Reinforcement shall be provided where required by analysis.~~

~~…~~

…

***1810A.3.10.4 Seismic requirements.*** *For structures assigned to Seismic Design Category D, E or F, a permanent steel casing having a minimum thickness of 3/ 8 inch shall be provided from the top of the micropile down to a minimum of 120 percent of the point of zero curvature. Capacity of micropiles shall be determined in accordance with Section 1810A.3.3 by at least two project-specific preproduction tests for each soil profile, size and depth of micropile. At least two percent of all production piles shall be proof tested to the load determined in accordance with Section 1617A.1.16.*

*Steel casing length in soil shall be considered as unbonded and shall not be considered as contributing to friction. Casing shall provide confinement at least equivalent to hoop reinforcing required by ACI 318 Section ~~18.13.4.~~18.13.5.*

*Reinforcement shall have Class 1 corrosion protection in accordance with PTI Recommendations for Prestressed Rock and Soil Anchors. Steel casing design shall include at least 1/ 16-inch corrosion allowance.*

*Micropiles shall not be considered as carrying any horizontal loads.*

**…**

**1810*A*.3.11.2 Seismic Design Categories D through F.**

For structures assigned to Seismic Design Category D, E or F, deep foundation element resistance to uplift forces or rotational restraint shall be provided by anchorage into the pile cap, designed considering the combined effect of axial forces due to uplift and bending moments due to fixity to the pile cap. Anchorage shall develop not less than 25 percent of the strength of the element in tension. Anchorage into the pile cap shall comply with the following:

1. In the case of uplift, the anchorage shall be capable of developing the least of the following:

1.1. The nominal tensile strength of the longitudinal reinforcement in a concrete element.

1.2. The nominal tensile strength of a steel element.

1.3. The frictional force developed between the element and the soil multiplied by 1.3.

**Exception:** The anchorage is permitted to be designed to resist the axial tension force resulting from the seismic load effects including overstrength factor in accordance with Section 2.3.6 or 2.4.5 of **{ ASCE 7 }**.

2. In the case of rotational restraint, the anchorage shall be designed to resist the axial and shear forces, and moments resulting from the seismic load effects including overstrength factor in accordance with Section 2.3.6 or 2.4.5 of **{ ASCE**

**7 }** or the anchorage shall be capable of developing the full axial, bending and shear nominal strength of the element.

3. The connection between the pile cap and the steel H-piles or unfilled steel pipe piles in structures assigned to Seismic Design Category D, E or F shall be designed for a tensile force of not less than 10 percent of the pile compression capacity.

   Exceptions:
   1. Connection tensile capacity need not exceed the strength required to resist seismic load effects including overstrength of { ASCE 7 } Section 12.4.3 or 12.14.3.2.
   2. ~~Connections need not be provided where the foundation or supported structure does not rely on the tensile capacity of the piles for stability under the design seismic force.~~

Where the vertical lateral-force-resisting elements are columns, the pile cap flexural strengths shall exceed the column flexural strength. The connection between batter piles and pile caps shall be designed to resist the nominal strength of the pile acting as a short column. Batter piles and their connection shall be designed to resist forces and moments that result from the application of seismic load effects including overstrength factor in accordance with Section 2.3.6 or 2.4.5 of **{ ASCE 7 }**.

**…**

**1810*A*.3.12 Grade beams.**

Grade beams shall comply with the provisions of { ACI 318 }.

> Exception: Grade beams designed to resist the seismic load effects including overstrength factor in accordance with Section 2.3.6 or 2.4.5 of { ASCE 7} *need not comply with Section 18.13.3 of ACI 318*.

…

**Item: #6**

**Chapter: CHAPTER 18A**

**SOILS AND FOUNDATIONS**

**…**

**SECTION 1811A – PRESTRESSED ROCK AND SOIL FOUNDATION ANCGORS**

…

***1811A.2 Adoption.*** *Except for the modifications as set forth in Sections ~~1705A.6.2,~~ 1811A.3 and 1811A.4, all prestressed rock and soil foundation anchors shall comply with PTI Recommendations for Prestressed Rock and Soil Anchors.*

***1811A.3 Geotechnical requirements.*** *Geotechnical report for the prestressed rock and soil foundation anchors shall address the following:*

1. *Minimum diameter and minimum spacing for the anchors including consideration of group effects.*
2. *Maximum unbonded length and minimum bonded length of the tendon.*
3. *Maximum recommended anchor tension capacity based upon the soil or rock strength/grout bond and anchor depth/spacing.*
4. *Allowable bond stress at the ground/grout interface and applicable factor of safety for ultimate bond stress.*
5. *Anchor axial tension stiffness recommendations at the anticipated anchor axial tension displacements, when required for structural analysis.*
6. *Minimum grout pressure for installation and post-grout pressure.*
7. *Class I ~~C~~corrosion ~~P~~protection is required for all permanent and extended temporary anchors in service more than 2 years. A minimum of Class II ~~C~~corrosion ~~P~~protection is required for temporary anchors in service less than or equal to 2 years.*
8. *Performance test shall be at a minimum of 1.6 times the design loads, but shall not exceed 80 percent of the specified minimum tensile strength of the tendons. There shall be a minimum of two preproduction test anchors. Preproduction test anchors shall be tested to ultimate load or maximum of 0.80 times the specified minimum tensile strength of the tendon. A creep test is required for all prestressed anchors with greater than 10 kips of lock-off prestressing load.*

…

***1811A.4 Structural Requirements.***

1. *Tendons shall be thread-bar anchors conforming to ASTM A722.*
2. *The anchors shall be placed vertical.*
3. *Design loads shall be based upon the load combinations in Section 2.4 of ASCE 7 ~~1605A.3.1~~ and shall not exceed 60 percent of the specified minimum tensile strength of the tendons.*

…

…

**Item: #7**

**Chapter: CHAPTER 18A**

## SOILS AND FOUNDATIONS

### SECTION 1812A – EARTH RETAINING SHORING

…

***1812A.4.2 Structural requirements**:*

1. *Tendons shall be thread-bar anchors conforming to ASTM A722.*
2. *Anchor design loads shall be based upon the load combinations in Section 2.4 of ASCE 7 ~~1605A.3.1~~ and shall not exceed 60 percent of the specified minimum tensile strength of the tendons.*

…

***1812A.4.3 Testing of tie-back anchors**: ~~Tests shall be in accordance with Section 1705A.6.3.1.~~* (Stricken items #1 and #4 relocated to Section 1705A.6.3.1. Stricken items #2 and #3 relocated to Section 1812A.5)

1. *The geotechnical engineer shall keep a record at job site of all test loads, total anchor movement, and report their accuracy.*
2. *If a tie-back anchor initially fails the testing requirements, the anchor shall be permitted to be regrouted and retested. If anchor continues to fail, the followings steps shall be taken:*
   a. *The contractor shall determine the cause of failure – variations of the soil conditions, installation methods, materials, etc.*
   b. *The contractor shall propose a solution to remedy the problem. The proposed solution will need to be reviewed and approved by the geotechnical engineer, shoring design engineer and building official.*
3. *After a satisfactory test, each anchor shall be locked-off in accordance with Section 8.4 of PTI Recommendations for Prestressed Rock and Soil Anchors.*
4. *The shoring design engineer shall specify design loads for each anchor.*

…

***1812A.5 Construction.** The construction procedure shall address the following:*

1. *Holes drilled for piles/tie-back anchors shall be done without detrimental loss of ground, sloughing or caving of materials and without endangering previously installed shoring members or existing foundations.*
2. *Drilling of earth anchor shafts for tie-backs shall occur when the drill bench reaches two to three feet below the level of the tie-back pockets.*
3. *Casing or other methods shall be used where necessary to prevent loss of ground and collapse of the hole.*
4. *The drill cuttings from earth anchor shaft shall be removed prior to anchor installation.*

5. *Unless tremie methods are used, all water and loose materials shall be removed from the holes prior to installing piles/tie-backs.*

6. *Tie-back anchor rods with attached centralizing devices shall be installed into the shaft or through the drill casing. Centralizing device shall not restrict movement of the grout.*

7. *After lagging installation, voids between lagging and soil shall be backfilled immediately to the full height of lagging.*

8. *The soldier piles shall be placed within specified tolerances in the drilled hole and braced against displacement during grouting. Fill shafts with concrete up to top of footing elevation, rest of the shaft can generally be filled with lean concrete. Excavation for lagging shall not be started until concrete has achieved sufficient strength for all anticipated loads as determined by the shoring design engineer.*

9. *Where boulders and/or cobbles have been identified in the geotechnical reports, contractor shall be prepared to address boulders and/or cobbles that may be encountered during the drilling of soldier piles and tie-back anchors.*

10. *The grouting equipment shall produce grout free of lumps and in dispensed cement. The grouting equipment shall be sized to enable the grout to be pumped in continuous operation. The mixer shall be capable of continuously agitating the grout.*

11. *The quantity of grout and grout pressure shall be recorded. The grout pressure shall be controlled to prevent excessive heave in soils or fracturing rock formations.*

12. *If post-grouting is required, post-grouting operation shall be performed after initial grout has set for 24 hours in the bond length only. Tie-backs shall be grouted over a sufficient length (anchor bond length) to transfer the maximum anchor force to the anchor grout.*

13. *Testing of anchors may be performed after post-grouting operations, provided grout has reached strength of 3,000 psi as required by PTI Recommendations for Prestressed Rock and Soil Anchors Section 6.11.*

14. *Anchors rods shall be tensioned straight and true. Excavation directly below the anchors shall not continue before those anchors are tested.*

~~15.~~ ~~(Relocated from Section 1812A.4.3, Item #2)~~ ~~*If a tie-back anchor initially fails the testing requirements, the anchor shall be permitted to be regrouted and retested. If anchor continues to fail, the followings steps shall be taken:*~~

~~*15.1. The contractor shall determine the cause of failure – variations of the soil conditions, installation methods, materials, etc.*~~

~~*15.2. The contractor shall propose a solution to remedy the problem. The proposed solution will need to be reviewed and approved by the geotechnical engineer, shoring design engineer and building official.*~~

16. ~~(Relocated from Section 1812A.4.3, Item #3)~~ ~~*After a satisfactory test, each anchor shall be locked-off in accordance with Section 8.4 of PTI Recommendations for Prestressed Rock and Soil Anchors.*~~

***1812A.6 Inspection, survey monitoring and observation***~~.~~*: Tests and inspections shall be in accordance with Section 1705A.6.3.* (Stricken text relocated to Section 1705A.6.3)

- ~~1.~~ *The shoring design engineer or his designee shall make periodic ~~inspections of~~ visits to the job site for the purpose of observing the installation of shoring system in accordance with section 1704A.6~~,~~ testing of tie-back anchors and monitoring of survey.*

- ~~2. *Testing, inspection and observation shall be in accordance with testing, inspection and observation requirements approved by the building official. The following activities and materials shall be tested, inspected, or observed by the special inspector and geotechnical engineer:*~~

  - ~~a. *Sampling and testing of concrete in soldier pile and tie-back anchor shafts.*~~
  - ~~b. *Fabrication of tie-back anchor pockets on soldier beams*~~
  - ~~c. *Installation and testing of tie-back anchors.*~~
  - ~~d. *Survey monitoring of soldier pile and tie-back load cells.*~~
  - ~~e. *Survey monitoring of existing buildings.*~~

- ~~3.~~ *A complete and accurate record of all soldier pile locations, depths, concrete strengths, tie-back locations and lengths, tie-back grout strength, quantity of concrete per pile, quantity of grout per tie-back and applied tie-back loads shall be maintained by the special inspector and geotechnical engineer. The shoring design engineer shall be notified of any unusual conditions encountered during installation.*

- ~~4.~~ *Calibration data for each test jack, pressure gauge and master pressure gauge shall be verified by the special inspector and geotechnical engineer. The calibration tests shall be performed by an independent testing laboratory and within 120 calendar days of the data submitted.*

- ~~5.~~ *Monitoring points shall be established at the top and at the anchor heads of selected soldier piles and at intermediate intervals as considered appropriate by the geotechnical engineer.*

- ~~6.~~ *Control points shall be established outside the area of influence of the shoring system to ensure the accuracy of the monitoring readings.*

- ~~7.~~ *The periodic basis of shoring monitoring, as a minimum, shall be as follows:*

  - *a. Initial monitoring shall be performed prior to any excavation.*
  - *b. Once excavation has begun, the periodic readings shall be taken weekly until excavation reaches the estimated subgrade elevation and the permanent foundation is complete.*
  - *c. If performance of the shoring is within established guidelines, shoring design engineer may permit the periodic readings to be bi-weekly. Once initiated, bi-weekly readings shall continue until the building slab at ground floor level is completed and capable of transmitting lateral loads to the permanent structure. Thereafter, readings can be monthly.*

d. *Where the building has been designed to resist lateral earth pressures, the periodic monitoring of the soldier piles and adjacent structure can be discontinued once the ground floor diaphragm and subterranean portion of the structure is capable of resisting lateral soil loads and approved by the shoring design engineer, geotechnical engineer and building official.*

e. *Additional readings shall be taken when requested by the special inspector, shoring design engineer, geotechnical engineer or building official.*

~~8.~~ *Monitoring reading shall be submitted to the shoring design engineer, engineer in responsible charge, and building official within three working days after they are conducted. Monitoring readings shall be accurate to within 0.01 feet. Results are to be submitted in tabular form showing at least the initial date of monitoring and reading, current monitoring date and reading and difference between the two readings.*

~~9.~~ *If the total cumulative horizontal or vertical movement (from start of construction) of the existing buildings reaches ½ inch or soldier piles reaches 1 inch all excavation activities shall be suspended. The geotechnical and shoring design engineer shall determine the cause of movement, if any, and recommend corrective measures, if necessary, before excavation continues.*

~~10.~~ *If the total cumulative horizontal or vertical movement (from start of construction) of the existing buildings reaches ¾ inch or soldier piles reaches $1^1/_2$ inches all excavation activities shall be suspended until the causes, if any, can be determined. Supplemental shoring shall be devised to eliminate further movement and the building official shall review and approve the supplemental shoring before excavation continues.*

~~11.~~ *Monitoring of tie-back anchor loads:*

a. *Load cells shall be installed at the tie-back heads adjacent to buildings at maximum interval of 50 feet, with a minimum of one load cells per wall.*

b. *Load cell readings shall be taken once a day during excavation and once a week during the remainder of construction.*

c. *Load cell readings shall be submitted to the geotechnical engineer, shoring design engineer, engineer in responsible charge and building official.*

d. *Load cell readings can be terminated once the temporary shoring no longer provides support for the buildings.*

…

***1812A.7 Monitoring of existing ~~DSA-SS, DSA-SS/CC, and OSHPD 1 and 4~~ structures.***

1. *The contractor shall complete a written and photographic log of all existing ~~DSA-SS, DSA-SS/CC, and OSHPD 1 & 4~~ structures within 100 ft or three times depth of shoring, prior to construction. A licensed surveyor shall document all existing substantial cracks in adjacent existing structures.*

2. *The contractor shall document existing condition of wall cracks adjacent to shoring walls prior to start of construction.*

…

**Item: #8**

**Chapter: CHAPTER 18A**

**SOILS AND FOUNDATIONS**

…

***SECTION 1813A - VIBRO STONE COLUMNS FOR GROUND IMPROVEMENT***

…

***1813A.3 Shallow foundations.*** *VSCs under the shallow foundation shall be located symmetrically around the centroid of the footing or load.*

1. *There shall be a minimum of four stone columns under each isolated or continuous/combined footing or approved equivalent.*
2. *The VSCs ~~or deep foundation elements~~ shall not be used to resist tension or overturning uplift from the shallow foundations.*

…

**Item: #9**

**Chapter: CHAPTER 19 – CONCRETE**

Note to Publisher: The following sub-sections include text whose origin is a new adopted material standard of this code (ACI 318). Amendments previously located in Section 1908A are relocated here because of their relevance to these ACI 318 sections and the deletion of model code Section 1908 with which they were previously associated. The base language from ACI 318 is shown here in vertical text (not italicized, underlined, nor highlighted) even though it is not continued from the prior version. This text should be printed along with the amendment text, which is formatted in accordance with the legend at the beginning of this document.

***1905.1.9 Modify ACI 318, Section 25.2.10 [OSHPD 1R, 2 & 5].*** *Replace ACI 318 Section 25.2.10 by the following:*

25.2.10 For ties and hoops in columns to be placed with shotcrete, minimum clear spacing shall be 3 in. *Shotcrete shall not be applied to spirally tied columns.*

***1905.1.10 ACI 318, Section 26.5.2. [OSHPD 1R, 2 & 5]*** *Modify ACI 318 Section 26.5.2.1 by replacing items (l), (m), and (n) with the following:*

(l) Shotcrete surfaces intended to receive subsequent shotcrete placement *following an interruption of 30 minutes or more* shall be roughened to a full amplitude of approximately ¼ in. before the shotcrete has reached final set. *The film of laitance which forms on the surface of the shotcrete shall be removed within approximately two hours after application by brushing with a stiff broom. If this film is not removed within two hours, it shall be removed by thorough wire brushing or a mechanical method acceptable to the enforcement agency.*

(m) Before placing additional material onto hardened shotcrete, laitance shall be removed, joints shall be cleaned, and the surface shall be dampened. *Construction joints over eight hours old shall be thoroughly cleaned with air and water prior to receiving shotcrete.*

(n) In-place fresh concrete that exhibits sags, sloughs, segregation, honeycombing, sand pockets, or other obvious defects shall be removed and replaced. *Shotcrete above sags and sloughs shall be removed and replaced while still plastic.*

*(q) Surface preparation: Concrete or masonry to receive shotcrete shall have the entire surface thoroughly cleaned and roughened by a mechanical method acceptable to the enforcement agency, and just prior to receiving shotcrete shall be thoroughly cleaned of all debris, dirt and dust. Concrete and masonry shall be brought to a saturated surface-dry (SSD) condition before shotcrete is deposited.*

**Item: #10**

**Chapter: CHAPTER 19A – CONCRETE**

…

**SECTION 1903A – SPECIFICATION FOR TESTS AND MATERIAL**

…

***1903A.8 Welding of reinforcing bars*** – *Modify ACI 318 Section ~~26.6.4.1(b)~~ 26.6.4.2(b) by adding the following:* …

…

**SECTION 1905A – MODIFICATIONS TO ACI 318**

…

**1905*A*.1 General.** The text of ACI 318 shall be modified as indicated in Sections 1905*A.1.1* through *1905A.1.~~15~~16.*

…

***1905A.1.3 ACI 318, Section 9.6.1.3.*** *Modify ACI 318, Section 9.6.1.3 by adding the following:*

*This section shall not be used for members that resist seismic loads, except for either of the following conditions:~~that reinforcement provided for foundation elements for one-story wood-frame or one-story light steel buildings need not be more than one-third greater than that required by analysis for all loading conditions.~~*

1. *Foundation ~~elements~~ members for one-story wood-frame or one-story light steel buildings.*
2. *Foundation members designed for seismic load combinations including the overstrength factor.*

…

Note to Publisher: The following sub-section includes text and a table whose origin is a new adopted material standard of this code (ACI 318). This new standard has revised the section and table this amendment pertains to such that inclusion of the source language is necessary. The base language from ACI 318 is shown here in vertical text (not italicized, underlined, nor highlighted) even though it is not continued from the prior version. This text should be printed along with the amendment text, which is formatted in accordance with the legend at the beginning of this document.

***1905A.1.~~9~~11 ACI 318, Section 19.2.1.1 and Table 19.2.1.1.*** *Modify ACI 318, Section 19.2.1.1 and Table 19.2.1.1 as follows:*

*~~For concrete designed and constructed in accordance with this chapter, $f'_c$ shall not be less than 3,000 psi (20.7 MPa). Reinforced normal weight concrete with specified compressive strength higher than 8,000 psi (55 MPa) shall require prior approval of structural design method and acceptance criteria by the enforcement agency.~~*

19.2.1.1 The value of $f'_c$ shall be in be in accordance with (a) through *(e)*:

(a) Limits for $f'_c$ in Table 19.2.1.1. Limits apply to both normalweight and lightweight concrete.

(b) Durability requirements in Table 19.3.2.1

(c) Structural strength requirements

(d) $f'_c$ for lightweight concrete in special moment frames and special structural walls, and their foundations, shall not exceed 5000 psi, unless demonstrated by experimental evidence that members made with lightweight concrete provide strength and toughness equal to or exceeding those of comparable members made with normalweight concrete of the same strength.

*(e) Concrete with specified compressive strength higher than 8,000 psi (55 MPa) shall require prior approval of structural design method and acceptance criteria by the enforcement agency.*

| **Application** | **Minimum $f_c'$, psi** |
|---|---|
| General | *3000* |
| ~~Foundations for structures assigned to SDC A, B, or C~~ | ~~2500~~ |
| ~~*Foundations for Residential and Utility use and occupancy classification with stud bearing wall construction two stories or less assigned to SDC D, E, or F*~~ | ~~2500~~ |
| ~~*Foundations for structures assigned to SDC D, E, or F other than Residential and Utility use and occupancy classification with stud bearing wall construction two stories or less*~~ | ~~3000~~ |
| Special moment frames<br>Special structural walls with Grade 60 or 80 reinforcement | 3000 |
| Special structural walls with Grade 100 reinforcement | 5000 |
| Precast-nonprestressed driven piles<br>Drilled shafts | 4000 |
| Precast-prestressed driven piles | 5000 |
| *Shotcrete* | *4000* |

…

Note to Publisher: The following sub-section includes a table whose origin is a new adopted material standard of this code (ACI 318). This new standard has revised a term the term “0.005” to “$\varepsilon_{ty}$+0.003” in each of the last two rows of the first column. This change (highlighted below) should be included in the printing of this section but should not be italicized nor underlined, as it is neither a model code revision nor a California amendment.

***1905A.1.~~13~~12 ACI 318, Table 21.2.2.*** *Replace Table 21.2.2 as follows:*

TABLE 21.2.2

STRENGTH REDUCTION RACTOR $\phi$ FOR MOMENT, AXIAL FORCE, OR COMBINED MOMENT AND AXIAL FORCE

| Net tensile strain ($\varepsilon_t$) | Classification | $\phi$ | |
|---|---|---|---|
| | | Types of transverse reinforcement | |
| | | Spirals conforming to 25.7.3 | Other |

| | | | | | |
|---|---|---|---|---|---|
| $\varepsilon_t \leq \varepsilon_{ty}$ | Compression-controlled | 0.75 | (a) | 0.65 | (b) |
| $\varepsilon_{ty} < \varepsilon_t <$ ~~0.005~~$\varepsilon_{ty}$+0.003 | Transition[1,2] | $0.75 + 0.15\frac{\varepsilon_t - \varepsilon_{ty}}{\varepsilon_t^* - \varepsilon_{ty}}$ | (c) | $0.65 + 0.25\frac{\varepsilon_t - \varepsilon_{ty}}{\varepsilon_t^* - \varepsilon_{ty}}$ | (d) |
| $\varepsilon_t \geq$ ~~0.005~~$\varepsilon_{ty}$+0.003 | Tension-controlled[3] | 0.9 | (e) | 0.9 | (f) |

1. For sections classified as transition, it shall be permitted to use $\phi$ corresponding to compression-controlled sections.
2. *$e_t^*$ is the greater of net tensile strain calculated for $P = 0.1A_g f'_c$ and ~~0.005~~$\varepsilon_{ty}$ + 0.003.*
3. *For sections with factored axial compression force $P_u \geq 0.1A_g f'_c$, $\phi$ shall be calculated using equation (c) or (d) for sections classified as transition, as applicable.*

***1905A.1.~~14~~13 ACI 318, Section 24.2.1.*** …

…

Note to Publisher: The following sub-sections include text whose origin is a new adopted material standard of this code (ACI 318). Amendments previously located in Section 1908A are relocated here because of their relevance to these ACI 318 sections and the deletion of model code Section 1908 with which they were previously associated. The base language from ACI 318 is shown here in vertical text (not italicized, underlined, nor highlighted) even though it is not continued from the prior version. This text should be printed along with the amendment text, which is formatted in accordance with the legend at the beginning of this document.

…

***1905A.1.14 ACI 318, Section 25.2.10.*** *Replace ACI 318 Section 25.2.10 by the following:*

25.2.10 For ties and hoops in columns to be placed with shotcrete, minimum clear spacing shall be 3 in. *Shotcrete shall not be applied to spirally tied columns.*

***1905A.1.15 ACI 318, Section 26.5.2.*** *Modify ACI 318 Section 26.5.2.1 by replacing items (l), (m), and (n) with the following:*

(l) Shotcrete surfaces intended to receive subsequent shotcrete placement *following an interruption of 30 minutes or more* shall be roughened to a full amplitude of approximately ¼ in. before the shotcrete has reached final set. *The film of laitance which forms on the surface of the shotcrete shall be removed within approximately two hours after application by brushing with a stiff broom. If this film is not removed within two hours, it shall be removed by thorough wire brushing or a mechanical method acceptable to the enforcement agency.*

(m) Before placing additional material onto hardened shotcrete, laitance shall be removed, joints shall be cleaned, and the surface shall be dampened. *Construction joints over eight hours old shall be thoroughly cleaned with air and water prior to receiving shotcrete.*

(n) In-place fresh concrete that exhibits sags, sloughs, segregation, honeycombing, sand pockets, or other obvious defects shall be removed and replaced. *Shotcrete above sags and sloughs shall be removed and replaced while still plastic.*

*(q) Surface preparation: Concrete or masonry to receive shotcrete shall have the entire surface thoroughly cleaned and roughened by a mechanical method acceptable to the enforcement agency, and just prior to receiving shotcrete shall be thoroughly cleaned of all debris, dirt and dust. Concrete and masonry shall be brought to a saturated surface-dry (SSD) condition before shotcrete is deposited.*

(Renumber remaining sections)

…

**SECTION 1908A – SHOTCRETE**

…

Note to Publisher: Sections 1908.2 through 1908.10.3 of the previous version of the model code have been removed from the current version. Prior amendments associated with those deleted sections have been relocated herein to the associated sections to which they apply. Those prior amendments should no longer be printed in Section 1908A except as noted below.

**1908*A*.1 General.** Shotcrete shall be in accordance with the requirements of ACI 318 *and the provisions of ACI 506R.* ~~*The specified compressive strength of shotcrete shall not be less than 4,000 psi (27.6 MPa).*~~ *The use of a shotcrete mockup panel to qualify bar clearance dimensions in accordance with ACI 318 Section 25.2.7.1 or contact lap splices in accordance with ACI 318 Section 25.5.1.7 is subject to the approval of the building official.*

***[DSA-SS] Exception:*** *The reference to ACI 506R shall be to ACI 506.2, unless otherwise approved by the enforcing agent.*

~~*Concrete or masonry to receive shotcrete shall have the entire surface thoroughly cleaned and roughened by a mechanical method acceptable to the enforcement agency, and just prior to receiving shotcrete shall be thoroughly cleaned of all debris, dirt and dust. Concrete and masonry shall be brought to a saturated surface-dry (SSD) condition before shotcrete is deposited.*~~

…

***1908A.2 Tests and Inspections.*** *Preconstruction tests of one or more shotcrete mockup panels prepared in accordance with Section 1705A.3.9.2 are required. In addition to testing requirements in ACI 318, special inspection and testing shall be in accordance with Section 1705A.3.9.*

***1908A.~~11~~3 Forms and ground wires for shotcrete.*** *…*

~~***1908A.12 Placing.*** *Shotcrete shall be placed in accordance with ACI 506R. In addition to testing requirements in Section 1908A, special inspection and testing shall be in accordance with Section 1705A.19.*~~

> ~~***[DSA-SS] Exception:*** *The reference to ACI 506R shall be to ACI 506.2 and ACI 506R.*~~

…

**Item: #11**

**Chapter: CHAPTER 21A – MASONRY**

**…**

**SECTION 2105A - QUALITY ASSURANCE**

**2105A.1 General.** A quality assurance program shall be used to ensure that the constructed masonry is in compliance with the approved construction documents. The quality assurance program shall comply with the inspection and testing requirements of Chapter 17*A, TMS 602 and Sections 2105A.2 through 2105A.4.*

***2105A.2 Compressive strength, f'$_m$.*** *The specified compressive strength, f '$_m$, assumed in design shall be 2000 psi (13.79MPa) for all masonry construction using materials and details of construction required herein. Testing of the constructed masonry shall be provided in accordance with Section 2105A.5 or Section 2105A.6.*

> ***Exception:*** *Subject to the approval of the enforcement agency, higher values of f'$_m$ may be used in the design of reinforced grouted masonry and reinforced hollow-unit masonry. The approval shall be based on prism test results submitted by the architect or engineer which demonstrate the ability of the proposed construction to meet prescribed performance criteria for strength and stiffness. The design shall take into account the mortar joint depth. In no case shall the f'$_m$ assumed in design exceed 3,000 psi (20.7MPa).*

*Where an f'$_m$ greater than 2000 psi (13.79MPa) is approved, the architect or structural engineer shall establish a method of quality control of the masonry construction acceptable to the enforcement agency which shall be described in the contract specifications. ~~Refer to Section 1705A.4.3 for compliance verification requirements.~~* ~~(Stricken text relocated to Section 1705A.4.3)~~ *Compliance with the requirements for the specified strength of constructed masonry shall be provided using prism test method in accordance with Section 2105A.5. Substantiation for the specified compressive strength prior to the start of construction shall be obtained by using prism test method in Section 2105A.5 and Section 2105A.3.*

***2105A.3 Mortar and grout tests.*** **[OSHPD 1 & 4]** *These tests are to establish whether the masonry components meet the specified component strengths. At the beginning of all masonry work, at least one test sample of the mortar shall be taken on three successive working days and at least at one-week intervals thereafter. Samples of grout shall be taken for each mix design, each day grout is placed, and not less than every 5,000 square feet of masonry wall area. They shall meet the minimum strength requirement given in ASTM C270 Table 1 and ASTM C476/TMS 602 Section 2.2 for mortar and grout respectively. Additional samples shall be taken whenever any change in materials or job conditions occur, as determined by the building official. When the prism test method is used during construction, the tests in this section are not required.*

*Test specimens for mortar and grout shall be made as set forth in ASTM C1586 and ASTM C1019.*

*Exception~~s~~:*

*~~1.~~ For nonbearing nonshear masonry walls not exceeding total wall height of 12 feet above top of foundation, mortar tests shall be permitted to be limited to those at the beginning of masonry work for each mix design.*

*…[Reserved DSA]*

***2105A.4 Masonry core testing.*** **[OSHPD 1 & 4]** ~~(Stricken text relocated to Section 1705A.4.5)~~

*Not less than two cores shall be taken from each building for each 5,000 square feet (465 m2) of the masonry wall area or fraction thereof. The approved agency shall perform or observe the coring of the masonry walls and sample locations shall be subject to approval of the registered design professional.*

*Core samples shall comply with the following:*

*1. Cored no sooner than 7 days after grouting of the selected area;*

*2. Be a minimum of 3 ¾ inches in nominal diameter; and*

*3. Sampled in such a manner as to exclude any masonry unit webs, mortar joint, or reinforcing steel. If all cells contain reinforcement, alternate core locations or means to detect void or delamination shall be selected by the registered design professional and approved by the building official.*

*Visual examination of all cores shall be made by an approved agency and the condition of the cores reported as required by the California Administrative Code. Shear test both joints between the grout core and the outside wythes or face shell of the masonry 28 days after grouting of the sample area using a shear test apparatus acceptable to the enforcement agency. Core samples shall not be soaked before testing. Core samples to be tested shall be stored in sealed plastic bags or non-absorbent containers immediately after coring and for at least 5 days prior to testing. The average unit shear value for each pair of cores (4 shear tests) from each 5,000 square feet of wall area (or less) on the cross section of core shall not be less than 2.5 √f 'm psi.*

*All cores shall be submitted to an approved agency for examination, even where the core specimens failed during the cutting operation. The approved agency shall report the location where each core was taken, the findings of their visual examination of each core, identify which cores were selected for shear testing, and the results of the shear tests.*

***Exceptions:***

*1. Core sampling and testing is not required for nonbearing nonshear masonry walls, not exceeding a total wall height of 12 feet above top of foundation, built with single-wythe hollow unit concrete masonry that attaches opposite face shells using webs cast as single unit, when designed using an $f'_m$ not exceeding 2,000 psi (13.79 MPa).*

*2. An infrared thermographic survey or other nondestructive test procedures, shall be permitted to be approved as an alternative system to detect voids or delamination in grouted masonry in-lieu of core sampling and testing.*

***2105A.5 Masonry prism method testing.*** **[OSHPD 1 & 4]** *Prism test method performed prior to the start or during construction shall be in accordance with TMS 602 Section 1.4 B.3. Prism test method* *performed on constructed walls shall be in accordance with TMS 602 Section 1.4 B.4.*

***2105A.6 Unit strength method testing.*** *Unit strength method testing shall be performed in accordance with TMS 602 Section 1.4 B.2.*

…

**Item: #12**

**Chapter: CHAPTER 22A – STEEL**

**…**

**SECTION 2211A - COLD-FORMED STEEL LIGHT-FRAME CONSTRUCTION**

**…**

**2211*A*.1.3 Truss design.** Cold-formed steel trusses shall comply with the additional provisions of Sections 2211*A*.1.3.1 through 2211*A*.1.3.3.

(The following item is an existing amendment that was missed in the printed version of the 2019 CBC and should be added back into the 2022 CBC.) *Complete engineering analysis and truss design drawings shall accompany the construction documents submitted to the enforcement agency for approval. When load testing is required, the test report shall be submitted with the truss design drawings and engineering analysis to the enforcement agency.*

…

***SECTION 2213A - TESTING AND FIELD VER/FICATION***

...

***2213A.1 Tests of High-strength Bolts, Nuts and Washers***. *High-strength bolts, nuts and washers shall be sampled and tested ~~by an approved agency for conformance with the requirements of applicable ASTM standards~~ in accordance with Section 1705A.2.6.*

***[Reserved for OSHPD]***

***2213A.2 Tests of end-welded studs.*** *End-welded studs shall be tested in accordance with ~~the requirements of the AWS D1.1, Sections 7.7 and 7.8~~ Section 1705A.2.5.*

**ITEM #13**

**CHAPTER 24**

## GLASS AND GLAZING

…

***SECTION 2410 - [DSA-SS, DSA-SS/CC, OSHPD 1, 1R, 2, 4* & *5] STRUCTURAL SEALANT GLAZING (SSG)***

…

***2410.1.2 Testing and inspection.*** *~~Qualification~~ Testing and inspection of SSG shall satisfy the following requirements: ~~this section. Quality assurance inspection and testing shall be in accordance with Section 1705A.13.2 and 1705A.14.2.1, respectively.~~*

*~~a.~~1.* *The seismic drift capability of SSG shall be determined by tests in accordance with AAMA 501.6~~,~~ and AAMA 501.4 ~~and ASCE 7, Section 13.5.9.2~~. Analysis as an alternative to testing is not acceptable for the purposes of satisfying the seismic drift requirements of the SSG system.*

***Exception: [DSA-SS, DSA-SS/CC]** …*

*~~b.~~2.* *The applicability of the specific AAMA 501.6 and AAMA 501.4 testing shall be subject to approval by the building official.*

*~~c.~~3.* *The panel test specimens used in the AAMA 501.6 and AAMA 501.4 testing shall include all glass types (annealed, heat strengthened, laminated, tempered) and insulated glass units that comprise more than 5 percent of the total glass curtain wall area used in the building.*

*~~d.~~4.* *AAMA 501.4 test specimen shall include the same materials, sections, connections, and attachment details to the test apparatus as used in the building.*

*~~e.~~5.* *Serviceability tests of SSG test specimen shall be performed in accordance with AAMA 501.4 after seismic displacement tests to the design story drift.*

*~~f.~~6.* *The window wall system using structural sealant by different manufacturer/product category shall be qualified in accordance with AAMA 501.6 and AAMA 501.4 testing for the seismic drift required. ~~Analysis as an alternative to testing is not acceptable for the purposes of satisfying the seismic drift requirements of the SSG system.~~*

*~~g.~~7.* *Where unitized SSG is used with horizontal stack joints at each floor level and split vertical mullions that can move independently, only a story height single unit need ~~to~~ be tested under AAMA 501.6. Where continuous horizontal bands of SSG are used in the building, either two or four sided, the aspect ratio (height-to-length) of the test specimen shall be less than 1.0, contain not less than two interior vertical joints and all joints (vertical in the case of two sided), including the perimeter of the glass, shall be glazed with SSG.*

~~*h.*~~*8.* *Where SSG continues around corners, the AAMA 501.4 test specimen shall include one corner panel to verify the kinematics of the corner condition under seismic drift.*

~~*I*~~*9* ***[OSHPD 1 & 4]*** *Quality assurance and inspection requirements shall include formalized post-installation tests using the point load testing procedure in accordance with ASTM C1392. The point load tests shall be done after the initial installation.* ~~(Stricken text relocated to Section 1705A.14.2.1)~~

~~*j.*~~*10.* ***[OSHPD 1 & 4]*** *Where the SSG is field assembled, hand pull tab tests in accordance with ASTM C1401, Section X2.1, one test every 100 linear feet, but not less than one test for each building elevation view shall be required.* (~~Stricken text relocated to Section 1705A.14.2.1)~~

*Existing AAMA 501.4 and 501.6 test results satisfying the requirements of this section shall be permitted, in lieu of project specific tests, when approved by the building official.*

***2410.1.3 Monitoring.*** *Short- and long-term periodic performance monitoring shall be provided in accordance with ASTM C1401, C1392 and C1394. Inspection frequencies recommended in ASTM C*~~*1392*~~*1394 Section 5.1 shall be followed.*

…

**Item: #14**

**Chapter: CHAPTER 35 – REFERENCED STANDARDS**

…

**ASCE/SEI**

7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Structures *with Supplement No. 1,* *2 and 3*

***104.11,*** 202, Table 1504.1.1, *Table* ***1504.B, 1510.7.1,*** 1602.l, ***1603A.2,*** 1604.3, ***1604A.4,*** Table 1604.3, 1604.5, Table 1604.5, 1604.8.2, 1604.9, 1605.1, 1605.2.1, 1605.3.1, 1605.3.1.2, 1605.3.2, 1605.3.2.l, 1607.8.1, 1607.8.1.1, 1607.8.1.2, 1607.9, 1607.13.1, 1607.13.3.1, 1608.1, 1608.2, 1608.3, 1609.1.1, 1609.2, 1609.3, 1609.5.1, 1609.5.3, 1611.2, 1612.2, 1613.1, 1613.2.2, 1613.2.3, 1613.2.5, Table 1613.2.3(1), Table 1613.2.3(2), 1613.2.5.1, 1613.2.5.2, 1613.3, 1614.1, 1615.1, ***1613A, 1617A, 1617.9, 1617.10, 1617.2,*** 1705.12, 1705.12.1.1, 1705.12.1.2, 1705.12.4, 1705.13.1.1, 1705.13.1.2, 1705.13.2, 1705.13.3, 1705.13.4, 1709.5, ***1803A.6,*** 1803.5.12, 1808.3.1, 1809.13, 1810.3.6.1, 1810.3.8.3.2, 1810.3.8.3.3, 1810.3.9.4, 1810.3.11.2, 1810.3.12, 1901.2, 1905.1.1, 1905.1.2, 1905.1.7, 1905.1.8, 2205.2.1.1, 2205.2.1.2, 2205.2.2, 2206.2.1, 2209.1, 2209.2, 2210.2, 2211.1.1.1, ***2212A.l.l, 2212A.2.4,*** Table 2304.6.1, Table 2306.3(3), Table 2308.7.5, 2404.1, ***2410.1.1, 2410.1.2,*** 2505.1, 2505.2, 2506.2.1

…

**ASTM**

…

C1394-20 Standard Guide for In-Situ Structural Silicon Glazing Evaluation
2410.1.3

…

**NEHRP**

*NEHRP Recommended Seismic Provisions for New Building and Other Structures, Volume 1, FEMA P-2082-1, September 2020.*
1617A.1.3

…

**Notation:**
Authority: Health and Safety Code, Sections 1275, 18928, 129790, and 129850
Reference: Health and Safety Code, Section 129850

**INITIAL EXPRESS TERMS**
**FOR PROPOSED BUILDING STANDARDS**
**OF THE OFFICE OF STATEWIDE HEALTH PLANNING AND DEVELOPMENT**
**REGARDING THE 2021 CALIFORNIA EXISTING BUILDING CODE**
**CALIFORNIA CODE OF REGULATIONS, TITLE 24, PART 10**
**(OSHPD 07/21)**

**LEGEND for EXPRESS TERMS (Based on model codes - Parts 2, 2.5, 3, 4, 5, 9, 10)**

1. Model Code language appears upright
2. Existing California amendments appear in *italic*
3. Amended model code or new California amendments appear *underlined & italic*
4. Repealed model code language appears ~~upright and in strikeout~~
5. Repealed California amendments appear in ~~*italic and strikeout*~~
6. Ellipsis (...) indicate existing text remains unchanged

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## INITIAL EXPRESS TERMS

**Item: #1**

**Chapter: CHAPTER 3*A* – PROVISIONS FOR ALL COMPLIANCE METHODS**

…

**Section: SECTION 303*A* Reserved**

**Section: SECTION 304~~3~~*A* STRUCTURAL DESIGN LOADS AND EVALUATION AND DESIGN PROCEDURES**

(Renumber previous sections)

…

***304~~3~~A.3.4.5 SPC-4D using ASCE 41.*** *Structures shall be deemed to comply with the SPC-4D requirements of Table 2.5.3, Chapter 6 of the California Administrative Code, when all of the following are satisfied:*

1. *Damage control structural performance level (S- 2) in accordance with Section 2.3.1.2.1 of ASCE 41 at BSE-1 E; and*
2. *Collapse Prevention Structural Performance Level (S-5) in accordance with Section 2.3.1.5 of ASCE 41 at BSE-2E; and*
3. *Items identified in Chapter 6, Article 10 of the California Administrative Code satisfy the requirements of Position Retention nonstructural performance level (N-B) in accordance with Section 2.3.2.2 at BSE-1E.*

***304A.3.4.5.1*** *~~-~~Replace ~~Exception~~ ASCE 41-13 § 7.2.13.2 ~~Separation Exemptions: Add the following exemption~~ with the following:*

~~*3. Seismic separation is deemed to comply with SPC-4D requirements and a pounding analysis is not required where either A) or B) apply:*~~

A. ~~The~~ Where the adjacent building was constructed using the 1989 or later edition of the California Building Code and built under OSHPD jurisdiction, the minimum building separation distance specified in Section 7.2.13.1 need not be evaluated for Structural Performance Level Damage Control or lower.

B. ~~The~~ Where adjacent ~~building~~ structure or building evaluated ~~meets the SPC building separation requirements in accordance with the California Administrative Code (CAC), Chapter 6, Section 3.4 and all the following are met~~ is not less than half as tall and adjacent structure has floors/levels that matches those of the building being evaluated, the following exceptions apply:

   1) ~~Where the structural resisting system of the adjacent building is different, the mass of the more flexible building is no greater than 50 percent of the mass of the stiffer building~~ For Structural Performance Level of Life Safety or lower, the seismic separation between the adjacent structure need not be evaluated.

   2) For Structural Performance Level of Damage Control, buildings need not meet the minimum separation distance specified in Section 7.2.13.1. where either a) or b) applies:
      a) Adjacent structure is more than 2 inches times the number of stories away from the building being evaluated at all floor levels.
      b) The adjacent building does not have any of the following structural deficiencies as defined in the California Administrative Code, Chapter 6, Article 3:
         1) Load path (3.1)
         2) Weak story (3.3.1)
         3) Soft story (3.3.2)
         4) Vertical discontinuity (3.3.5) or
         5) Torsion (3.3.6)

C. Where an approved pounding analysis procedure that accounts for the change in dynamic response of the structures caused by impact is used, the evaluated and retrofitted buildings need not meet the minimum separation distance specified in Section 7.2.13.1. Such analysis shall demonstrate that:

   1) The structures are capable of transferring forces resulting from impact for diaphragms located at the same elevation; or

   2) The structures are capable of resisting all required vertical and lateral forces considering the loss of any elements or components damaged by impact of the structures.

**Item: #2**

**Chapter: CHAPTER 3*A* – PROVISIONS FOR ALL COMPLIANCE METHODS**

**Section: SECTION 304~~3~~*A* STRUCTURAL DESIGN LOADS AND EVALUATION AND DESIGN PROCEDURES**

(Renumber previous sections)

…

***304~~3~~A.3.5.8 ASCE 41 Section 8.4.1.1. Replace ASCE 41 Section 8.4.1.1 as follows:***
*Prescriptive Expected Capacities. Not permitted by OSHPD.*

***304~~3~~A.3.5.9 ASCE 41 Section 8.4.2.3.2.1 Modify ASCE 41 Section 8.4.2.3.2.1 as follows:***

**8.4.2.3.2.1 Foundation Modeled as a Fixed Base** If the base of the structure is assumed to be completely rigid, the foundation soil shall be classified as deformation controlled. Component actions shall be determined by Eq. (7-34). Acceptance criteria shall be based on Eq. (7-36), m-factors for foundation soil shall be 1.5 for Immediate Occupancy, 3.0 for Life Safety, and 4.0 for Collapse Prevention, and the use of upper-bound component capacities shall be permitted. Where overturning results in an axial uplift force demand from linear analysis, this uplift shall be considered deformation controlled, and an m-factor of 1.5 for Immediate Occupancy, 3.0 for Life Safety, and 4.0 for Collapse Prevention applied to the expected restoring dead load shall be used.

*Alternatively, when seismic evaluation is performed for foundation after global analysis of the superstructure is complete, both overturning and axial seismic pseudo force demands are permitted to be divided by the m-factors above, provided the foundation is analyzed as a beam on Winkler springs (soil does not resist tension). The vertical spring stiffness values may be determined either from Figure 8-2 or Equation 8-11, or as provided by the geotechnical engineer. Acceptance criteria for soil bearing shall be considered met, based on one of the following methods either A or B:*

A) *Soil spring reactions are limited by the ultimate soil bearing capacity and the foundation system is stable under the applied loads.*
B) *The resisting soil pressure distribution under the footing is triangular such that the maximum soil bearing pressure at any point of the footing is less than the ultimate soil bearing capacity.*
*Subject to the approval of the authority having jurisdiction, higher soil pressures may be permitted when appropriately justified.*

*The evaluation of the foundation structural element shall be considered as force controlled in accordance with the material chapters using the bearing pressure distribution under the footing from the same method used for the soil bearing acceptance criteria.*

**8.4.2.3.2.2 Foundation Interface Modeled as a Flexible Base** Where the foundation flexibility is included in the mathematical model and is modeled using linear elastic foundation soil representation, the foundation soil shall be classified as deformation-controlled. Component actions shall be determined by Eq. (7-34). For rectangular or I-shaped footings, acceptability of foundation overturning shall be based on the m-factors in Table 8-3. Where global overturning results in an uplift force on the foundation, the expected dead load action on that portion of the foundation being uplifted shall be multiplied by the appropriate m-factor from Table 8-3 and shall be greater than the absolute axial tension demand on the foundation.

The m-factors in Table 8-3 depend on $A_c/A_f$, $b/L_c$, and the missing area ratio $(A_{rect} - A_f)/A_{rect}$, where $A_c$ is defined in Section 8.4.2.3.1. The idealized footing configurations and corresponding parameters are defined in Fig. 8-3. The parameter b is defined as the width of rectangular footings and the flange width of I-shaped footings. The parameter $L_c$ is defined as the length of the contact area and equal to $A_c/b$. The extent of the I-shape shall be quantified by the missing area ratio. For I-shaped footings, the parameter $A_{rect}$ is equal to the area of the smallest rectangle that covers the footing footprint, and $A_f$ is the actual footing area.

*Alternatively, superstructure pseudo force overturning demands to the foundation are permitted to be divided by the appropriate m-factors above and applied to the mathematical model representing the foundation system only, re-analyzed as a beam on Winkler springs (soil does not resist tension). Acceptance criteria for soil bearing shall be considered met, based on one of the following methods either A or B:*

A) *Soil spring reactions are limited by the ultimate soil bearing capacity and the foundation system is stable under the applied loads.*
B) *The resisting soil pressure distribution under the footing is triangular and the maximum soil bearing pressure at any point of the footing is less than the ultimate soil bearing capacity.*
*Subject to the approval of the authority having jurisdiction, higher soil pressures may be permitted when appropriately justified.*

*The evaluation of the foundation structural element shall be considered as force controlled in accordance with the material chapters using the bearing pressure distribution under the footing from the same method used for the soil bearing acceptance criteria.*

…
(renumber remaining sections)

…
***304~~3~~A.3.5.13 ASCE 41 Section 10.12.3 Modify ASCE 41 Section 10.12.3 as follows:***

**10.12.3 Evaluation of Existing Condition** Allowable soil capacities (subgrade modulus, bearing pressure, and passive pressure) and foundation displacements for the selected performance level shall be as prescribed in Chapter 8 or as established with project specific data. All components of existing foundation systems and all new material, components, or components required for retrofit shall be evaluated as force-controlled actions. However, the capacity of the foundation components need not exceed 1.25
times the capacity of the supported vertical structural component or element (column or wall).

> *Exception: Component actions that are deformation controlled are permitted to use their expected strengths for the acceptance criteria.*

…
(renumber remaining sections)

**Item: #3**

**Chapter: CHAPTER 16 - REFERENCED STANDARDS**

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***ASCE/SEI*** ***American Society of Civil Engineers***

***Structural Engineering Institute***

***1801 Alexander Bell Drive***

***Reston, VA 20191-4400***

*7-16* **Minimum Design Loads and Associated Criteria for Buildings and Other Structures** with Supplement No. 1 ***[OSHPD 1, 1R, 2, 4 & 5]*** *and 3*

304.2, *304~~3~~A.2,* 304.3.1, *501A.3, 502A.5, 503A.13,* 503.4, 503.12, 503.13, 805.3, 805.4

***41—13: Seismic Evaluation and Retrofit of Existing Buildings***

*304~~3~~A.2, 304~~3~~A.3.4, 304~~3~~A.3.5*

**41-17: Seismic Evaluation and Retrofit of Existing Buildings** ***[OSHPD 1R, 2, 4 & 5]*** *with Supplement No. 1*

304.3.1, Table 304.3.1, 304.3.2, Table 304.3.2

*…*

NOTATION:

Authority: Health and Safety Code Section 130005(g) & 130021

Reference:  Health and Safety Code Section 1275, 129790, 129850 & 130005(g)

…