2007/2008 INTERNATIONAL BUILDING CODE
Structural Code Development Committee

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INTERNATIONAL BUILDING CODE STRUCTURAL COMMITTEE HEARING RESULTS

S1-07/08

Committee Action: **Disapproved**

Committee Reason: The proposal would create a conflict between ASTM D448 and SPRI RP4 standards. Also under ASTM D 1863 ninety percent of aggregate would be finer than three-quarters on an inch.

Assembly Action: **None**

S2-07/08

Committee Action: **Approved as Submitted**

Committee Reason: The code change improves clarity by removing unnecessary text from the definition of the term ballast.

Assembly Action: **None**

S3-07/08

Committee Action: **Disapproved**

Committee Reason: The proposal could lead to confusion over the application of roof live load criteria. The proposed definitions of landscaped roofs and roof garden may be necessary, but this potential confusion needs to be resolved.

Assembly Action: **None**

S4-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This rewording of Section 1503.1 is largely editorial and it appropriately removes the maintenance requirement from the building code.

Assembly Action: **None**

S5-07/08

Errata: Add standards to Part I of the code change:

Add standards to Chapter 35 as follows:

UL 2218-02 Standard for Impact Resistance of Prepared Roof Covering Materials
FM 4473-05 Specification Test Standard for Impact Resistance Testing of Rigid Roofing Materials by Impacting With Freezer Balls

Add Standards to Part II of the code change:

Add standards to Chapter 43 as follows:

UL 2218-02 Standard for Impact Resistance of Prepared Roof Covering Materials
FM 4473-05 Specification Test Standard for Impact Resistance Testing of Rigid Roofing Materials by Impacting With Freezer Balls

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standard FM 4473 (2005) indicated that, in the opinion of ICC Staff, the standard did not comply with ICC standards criteria, Section 3.6.2.6.

Analysis: Review of proposed new standard UL 2218-96 indicated that, in the opinion of ICC Staff, the standard did not comply with ICC standards criteria, Section 3.6.2.3.

PART I – IBC STRUCTURAL
Committee Action: Disapproved

Committee Reason: Disapproval was requested by the proponent. Some of the proposed terminology is confusing. In addition, references to testing sections should be corrected.

Assembly Action: None

PART II – IRC B/E Withdrawn by Proponent

S6-07/08

Errata: Add reference to footnote in heading of table as follows:

TABLE 1507.2.7.1(1)
CLASSIFICATION OF ASPHALT SHINGLES PER ASTM D 7158[a]

Correct Figure number in first column heading as follows:

TABLE 1507.2.7.1(2)
CLASSIFICATION OF ASPLALT SHINGLES PER ASTM D 3161

MAXIMUM BASIC WIND SPEED FROM FIGURE ~~R301.2(4)~~ 1609

PART I – IBC STRUCTURAL
Committee Action: Approved as Submitted

Committee Reason: Agreement with the proponent's reason which indicates that this code change provides clear scoping for the applicable test standard for wind resistance of asphalt shingles. This action will also be consistent with the IRC B & E committee's action.

Assembly Action: None

PART II – IRC B/E
Committee Action: Approved as Submitted

Committee Reason: This change makes testing to ASTM D 7158 as the default for asphalt shingles.

Assembly Action: None

S7-07/08

Committee Action: Approved as Submitted

Committee Reason: This code change removes a misplaced section that is now redundant, due to a code change approved in the previous code development cycle.

Assembly Action: None

S8-07/08

Errata: Indicate the year of the proposed standard as follows:

Single-Ply Roofing Institute
ANSI/SPRI WD-1-07 Wind Design Standard Practice for Roofing Assemblies

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standard SPRI WD-1-07 indicated that, in the opinion of ICC Staff, the standard did not comply with ICC standards criteria, Section 3.6.2.1.

Committee Action: **Disapproved**

Committee Reason: There is no verification available that shows the testing is accurate for the extrapolation method of the proposed reference standard. The standard also needs to be readily available.

Assembly Action: **None**

S9-07/08

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standard FM 4474-04 indicated that, in the opinion of ICC Staff, the standard did comply with ICC standards criteria.

Committee Action: **Approved as Submitted**

Committee Reason: Agreement with the proponent's reason which indicates that the new FM 4474 standard provides an acceptable substitute for FM 4450 and FM 4470.

Assembly Action: **None**

S10-07/08

Committee Action: **Disapproved**

Committee Reason: Based on the varying sizes of stone and the excessive amounts of stone that can be there below the nominal size that is specified. In addition the removal of the reference to Section 1504.8 is a concern, since Table 1504.8 was added to the code to go beyond the standard. It would be wrong to remove it because of an apparent conflict. Referencing SPRI RP-4 solely, could create a conflict by referring to an outdated edition of ASCE 7. Also regarding the use of the "approved design method", it's not clear that Building Officials would want to approve such designs when they're brought to them.

Assembly Action: **None**

S11-07/08

Committee Action: **Disapproved**

Committee Reason: The proposed section on landscaped roofs is poorly worded. A direct reference to Section 1504.1 should be provided. This is also consistent with the committee's disapproval of S3-07/08.

Assembly Action: **None**

S12-07/08

Committee Action: **Disapproved**

Committee Reason: The current wording already restricts the use of aggregate on roofs in hurricane-prone regions, regardless of what the aggregate is used for. It appears that the proposed clarifying verbiage may allow something to fall through a crack.

Assembly Action: **Approved as Submitted**

S13-07/08

Committee Action: **Disapproved**

Committee Reason: The committee's disapproval on this code change is for reasons similar to S14-07/08. Such proposals should separate the treatment of ballast from that of aggregate.

Assembly Action: **None**

S14-07/08

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standard ASTM D 1863 indicated that, in the opinion of ICC Staff, the standard did comply with ICC standards criteria.

Committee Action: Disapproved

Committee Reason: It is unclear how a building official can determine that the required 50 percent of the total aggregate is embedded in the flood coat. Furthermore regardless of the actual embedded percentage, there would seem to be a problem posed by the percentage of aggregate that is not embedded.

Assembly Action: None

S15-07/08

PART I – IBC STRUCTURAL
Committee Action: Disapproved

Committee Reason: See related code change proposal S82-07/08.

Assembly Action: None

PART II – IRC B/E
Committee Action: Approved as Submitted

Committee Reason: This change clarifies the code in regard to the measurement of mean roof height. Currently there is no reference in the code specific where to measure the mean roof height from. This change provides the reference.

Assembly Action: None

S16-07/08

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standard SPRI RP-14 indicated that, in the opinion of ICC Staff, the standard did not comply with ICC standards criteria, Section 3.6.3.1.

Committee Action: Disapproved

Committee Reason: The code change was disapproved because the proposed reference standard is only a draft. The proponent can resubmit when the standard is ready.

Assembly Action: None

S17-07/08

This code change was heard by the IBC Fire Safety Code Development Committee.

Committee Action: Disapproved

Committee Reason: The committee felt that the testing criteria related to the slope and airflow should be revised as part of the development of the referenced standards rather than in the code. Further, the committee indicated that no data was provided to support the revisions to slope and airflow requirements. Lastly, the committee felt that verifying these criteria would be a burden to the code official.

Assembly Action: None

S18-07/08

PART I – IBC FIRE SAFETY
Committee Action: Approved as Submitted

Committee Reason: The committee agreed that based on the wide range of materials and configurations of concrete and clay roofing products available, all roof assemblies with clay or concrete tile roof covering should be tested in accordance with ASTM E108 or UL 790 to substantiate a Class A roof assembly, unless these products are installed on a noncombustible deck.

Assembly Action: None

PART II – IRC B/E
Committee Action: Disapproved

Committee Reason: Based on proponent's request for disapproval in favor of S20-07/08, Part II.

Assembly Action: None

S19-07/08

PART I – IBC FIRE SAFETY
Committee Action: Approved as Modified

Modify the proposal as follows:

1505.2 Class A roof assemblies. Class A roof assemblies are those that are effective against severe fire test exposure. Class A roof assemblies and roof coverings shall be listed and identified as Class A by any approved testing agency. Class A roof assemblies shall be permitted for use in buildings or structures of all types of construction.

Exceptions:

1. Class A roof assemblies include those with coverings of brick, masonry, slate, clay or concrete roof tile, or an exposed concrete roof deck.
2. Class A roof assemblies also include ferrous or copper shingles or sheets, metal sheets and shingles, installed on non-combustible decks or ferrous, copper or metal sheets installed without a roof deck <u>on non-combustible framing.</u>

Committee Reason: The committee agreed that metal sheets and shingles installed on a non-combustible deck and metal sheet roofing installed directly on non-combustible framing without a roof deck achieves a Class A roof assembly, based on data that indicated that flaming of combustible decks/framing caused premature failure of a roof assembly. The modification clarifies that metal sheet roofing that is installed without a roof deck still requires installation on framing that is non-combustible in order to qualify as a Class A roof covering.

Assembly Action: None

PART II – IRC B/E
Committee Action: Disapproved

Committee Reason: Based on proponent's request for disapproval in favor of S20-07/08, Part II. Also, ferrous, copper or metal sheets without a roof deck may not be a Class A roof. This issue needs to be reworked.

Assembly Action: None

S20-07/08

PART I – IBC FIRE SAFETY
Committee Action: Approved as Submitted

Committee Reason: Based on data indicating that certain thicknesses and styles of clay and concrete roofing tiles do not qualify as Class A roof assemblies when installed on combustible decks, the committee agreed that these products should be limited to Class A when installed only on non-combustible decks. When installed on combustible decks, these products need to be tested in accordance with ASTM E108 or UL 790 to substantiate a Class A roof assembly

Assembly Action: None

PART II – IRC B/E
Committee Action: **Approved as Submitted**

Committee Reason: This change improves the code by requiring these roof materials to be installed on a non-combustible deck. If the materials meet the Class A test they can be installed without a non-combustible deck.

Assembly Action: **None**

S21-07/08

Withdrawn by Proponent

S22-07/08

This code change was heard by the IBC Fire Safety Code Development Committee.

Committee Action: **Disapproved**

Committee Reason: Based on incorrect terminology (fire resistance should be fire classification) and the lack of testing criteria addressing roof gardens and landscaped roofs.

Assembly Action: **None**

S23-07/08

This code change was heard by the IBC Fire Safety Code Development Committee.

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standard SPRI VF-1 indicated that, in the opinion of ICC Staff, the standard did not comply with ICC standards criteria, Section 3.6.3.1.

Committee Action: **Disapproved**

Committee Reason: Based on the proponent's request. The proponent requested disapproval from the committee based on the fact that the proposed referenced standard (ANSI/SPRI VF-1, Fire Design Standard Guidelines for Vegetative Roofs) was not yet completed or published.

Assembly Action: **None**

S24-07/08

This code change was heard by the IBC Fire Safety Code Development Committee.

Committee Action: **Disapproved**

Committee Reason: The committee voted for disapproval based on terminology and enforcement problems, and the overly restrictive requirement for all exposed roof areas to be Class A roof assemblies. The term "Green roofs" should probably be "landscaped roof." The enforcement problems arise from the lack of testing criteria for the landscaped roof portions and lack of labeling requirements.

Assembly Action: **None**

S25-07/08

PART I – IBC STRUCTURAL
Committee Action: **Approved as Submitted**

Committee Reason: This proposal clarifies the original intent of Section 1507.2.5 by removing unnecessary wording.

Assembly Action: **None**

PART II – IRC B/E
Committee Action: Approved as Submitted

Committee Reason: This change eliminates redundancy in the code. The self seal strips or interlocking are already addressed in the test standards referenced in other sections of the code.

Assembly Action: None

S26-07/08

PART I – IBC STRUCTURAL
Committee Action: Disapproved

Committee Reason: it is inappropriate to equate sustained wind speeds used in testing to the three second gust design wind speed. This action is also consistent with the IRC B & E code development committee's action.

Assembly Action: None

PART II – IRC B/E
Committee Action: Disapproved

Committee Reason: The committee prefers the language in S6-07/08, Part II.

Assembly Action: None

S27-07/08

Committee Action: Approved as Submitted

Committee Reason: The increased valley lining width is consistent with current industry installation practices.

Assembly Action: None

S28-07/08

Committee Action: Disapproved

Committee Reason: The proposed exception has the potential to eliminate the minimum slope requirement. Justification has not been provided to support this.

Assembly Action: None

S29-07/08

PART I – IBC STRUCTURAL
Committee Action: Disapproved

Committee Reason: This proposal was poorly worded thus S30-07/08 was preferred. There was some confusion over whether screws would inadvertently be disallowed for securing metal roof panels with the proposed reference to a standard that is only for driven fasteners.

Assembly Action: None

PART II – IRC B/E
Committee Action: Disapproved

Committee Reason: The committee needs more information from the proponent. There is no guidance on how to apply ASTM F 1667. There is no confirmation that the added sizes are to apply to stainless steel fasteners only.

Assembly Action: None

S30-07/08

PART I – IBC STRUCTURAL
Committee Action: **Approved as Modified**

Modify proposal as follows:

1507.4.4 Attachment. Metal roof panels shall be secured to the supports in accordance with the approved manufacturer's fasteners. In the absence of manufacturer recommendations, the following fasteners shall be used:

1. Galvanized fasteners shall be used for steel roofs.
2. Copper, Brass, Bronze, copper alloy ~~and~~ or 300 series stainless-steel fasteners shall be used for copper roofs.
3. Stainless-steel fasteners are acceptable for all types of metal roofs.

Committee Reason: The proposal allows fasteners for cooper roofs that are possibly excluded by the current verbiage. The modification uses "or" in place of "and" since the fasteners listed are options.

Assembly Action: **None**

PART II – IRC B/E
Committee Action: **Approved as Submitted**

Committee Reason: This change adds acceptable fasteners for copper roofs in absence of manufacturer's recommendations.

Assembly Action: **None**

S31-07/08

Committee Action: **Disapproved**

Committee Reason: There is a concern that deleting the table for installation of wood shingles and shakes would result in the loss of some updates from recently approved code changes. Therefore the table should be retained at this time.

Assembly Action: **None**

S32-07/08

Committee Action: **Disapproved**

Committee Reason: There is a concern with the proposed exception referring to coal-tar surfacing which should be under Section 1507.15.

Assembly Action: **None**

S33-07/08

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standard ASTM D 6509-00 indicated that, in the opinion of ICC Staff, the standard did not comply with ICC standards criteria, Section 3.6.2.1.

Committee Action: **Approved as Modified**

Modify proposal as follows:

1507.11.2 Material standards. Modified bitumen roof coverings shall comply with CGSB 37-GP-56M, ASTM D 6162, ASTM D 6163, ASTM D 6164, ASTM D 6222, ASTM D 6223, ASTM D 6298 ~~and~~ or ASTM D 6509.

(Portions of proposal not shown remain unchanged)

Committee Reason: Including a reference to ASTM D 6509 improves the code by providing access to another product. The modification is editorial to keep the intent of the code the same.

Assembly Action: **None**

S34-07/08

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standard ASTM D 448 indicated that, in the opinion of ICC Staff, the standard did comply with ICC standards criteria.

Committee Action: **Approved as Submitted**

Committee Reason: The proposal adds a needed clarification for stone that is used as ballast.

Assembly Action: **None**

S35-07/08

PART I – IBC STRUCTURAL
Committee Action: **Approved as Modified**

Modify proposal as follows:

1507.14.2 Material standards. Spray-applied polyurethane foam insulation shall comply with Types III ~~and~~ or IV as defined in ASTM C 1029.

Committee Reason: This code change establishes product standards. The modification is editorial and makes it clear that the insulation should comply with one or the other rather than both.

Assembly Action: **None**

PART II – IRC B/E
Committee Action: **Approved as Submitted**

Committee Reason: This change provides the correct reference to the types of acceptable spray polyurethane insulation, as well as correcting an omission.

Assembly Action: **None**

S36-07/08

Committee Action: **Disapproved**

Committee Reason: The proposed exception for coal-tar surfaced roofs seems misplaced. Coal-tar surfacing does not belong under liquid-applied coatings.

Assembly Action: **None**

S37-07/08

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standard ASTM D 6947-07 indicated that, in the opinion of ICC Staff, the standard did comply with ICC standards criteria.

PART I – IBC STRUCTURAL
Committee Action: **Approved as Submitted**

Committee Reason: The addition of ASTM D 6947 clarifies code requirements for liquid-applied coatings.

Assembly Action: **None**

PART II – IRC B/E
Committee Action: **Approved as Submitted**

Committee Reason: This change allows more options for the roofing industry and provides more options for spray polyurethane foam.

Assembly Action: **None**

S38-07/08

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standard ASTM D 3409-93(2002)e1 indicated that, in the opinion of ICC Staff, the standard did comply with ICC standards criteria.

Analysis: Review of proposed new standards ASTM D 1187-97(2002)e1, D 2824-06 and D 6848-02 indicated that, in the opinion of ICC Staff, the standards did not comply with ICC standards criteria, Section 3.6.2.1.

Committee Action: **Disapproved**

Committee Reason: The proposed table may add value to the code, but three of the proposed new reference standards were not compliant with ICC requirements for referenced standards.

Assembly Action: **None**

S39-07/08

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standards ASTM D 3805-97(2003)e1 and D4261-05 indicated that, in the opinion of ICC Staff, the standards did not comply with ICC standards criteria, Section 3.6.2.1.

Committee Action: **Disapproved**

Committee Reason: The committee's disapproval is based on proposed standards that do not meet ICC requirements for referenced standards as well as questions on the applicability of the proposed standard to concrete surfacing.

Assembly Action: **None**

S40-07/08

Committee Action: **Disapproved**

Committee Reason: The sections that are cross referenced are for roof live loads under structural design. There was no justification provided for requiring that these roofs be designed by an accredited green roof professional, rather than another registered design professional.

Assembly Action: **None**

S41-07/08

Committee Action: **Disapproved**

Committee Reason: Section 1507.16 currently provides a cross reference to live load requirements for landscaped roofs. Landscaped roofs are not exempt from smoke and heat vent requirements in Section 910. There is no reason for adding such requirements in this section of the code.

Assembly Action: **None**

S42-07/08

PART I – IBC GENERAL

Committee Action: **Approved as Modified**

Modify the proposal as follows:

1509.2 Penthouses. A penthouse or penthouses in compliance with Sections ~~1509.2~~ 1509.2.1 through 1509.2.3 shall ~~not~~ be considered as a portion of the story below.

(Portions of proposal not shown remain unchanged)

Committee Reason: Clarifies application of the provisions for penthouses and removes requirements from the definition of penthouse. The modification simply revises the phrase “in compliance with section 1509.2” to “in compliance with Section 1509.2.1 through 1509.2.3” to clarify the applicability of the subsections and be consistent with code style. Additionally the term “not” in section 1509.2 was deleted as it would nullify the revisions the proponent intended.

Assembly Action: **None**

PART II – IBC MEANS OF EGRESS

Committee Action: **Disapproved**

Committee Reason: The proposal is too restrictive. The definition for means of egress deals with occupied spaces; therefore, since penthouses are considered unoccupied spaces, this could be a possible conflict. A penthouse typically has a very low or no occupants – the proposal would result in two means of egress being required for a penthouse. In addition, this requirement could result in at least two stairways to extend to the roof.

Assembly Action: **None**

S43-07/08

This code change was heard by the IBC General Code Development Committee.

Committee Action: **Disapproved**

Committee Reason: Sufficient justification was not provided to allow the additional use of fire retardant treated wood in Type IB construction for penthouses.

Assembly Action: **None**

S44-07/08

PART I – IBC STRUCTURAL

Committee Action: **Disapproved**

Committee Reason: There was a concern that the phrase “relies on adhesive sealants” could be interpreted as restricting reroofing for a much broader range of roof coverings than indicated in reason.

Assembly Action: **None**

PART II – IRC B/E

Committee Action: **Disapproved**

Committee Reason: This proposal does not address all areas of the country properly. There is no documentation in support that this is a problem. There is no guidance for how to determine if the roof covering relies on adhesives. This is written in poor code format.

Assembly Action: **None**

S45-07/08

This code change was heard by the IBC Fire Safety Code Development Committee.

Committee Action: Disapproved

Committee Reason: The committee felt that these proposed requirements for snow retention were too broad and based on subjective language in the proposed definitions such as “likely to cause great bodily harm”, “in the path of such discharge” and “methods that prevent hazardous roof discharge” would be unenforceable.

Assembly Action: None

S46-07/08

Committee Action: Disapproved

Committee Reason: There is a concern for possible unintended consequences in making definitions in the structural chapters applicable throughout the code.

Assembly Action: None

S47-07/08

Committee Action: Approved as Modified

Modify proposal as follows:

1603.1.6 Geotechnical information. The ~~soil classification and~~ design load-bearing values <u>of soils</u> shall be shown on the construction documents.

(Portions of proposal not shown remain unchanged)

Committee Reason: This proposal relocates information that is already in the code to a more appropriate section in Chapter 16, resulting in a clarification of requirements for construction documents. The modification removes “soils classification” in recognition that in some instances the allowable soil pressure may be determined without borings. Where a soils report is performed it is still part of the submittals required by Chapter 1.

Assembly Action: None

S48-07/08

Committee Action: Approved as Submitted

Committee Reason: The requirements for posting live loads do not belong in the structural design chapter. This proposal relocates them more appropriately to the administrative requirements in Chapter 1.

Assembly Action: None

S49-07/08

Committee Action: Disapproved

Committee Reason: There is obviously some confusion that needs to be cleared up. In Section 1604.2 the proposal would change the term “parts” to “portions” in the first sentence. But the second sentence which begins with “alternatively” leaves the term “parts” unchanged. It appears that under the alternative you could be referring to something different than the basic requirement.

Assembly Action: None

S50-07/08

PART I – IBC STRUCTURAL
Committee Action: Approved as Submitted

Committee Reason: This code change provides clarity because the deflection limit currently exists in ASTM E 1300. Adding the limitation to footnote "h" makes it more apparent to the code user.

Assembly Action: None

PART II – IRC B/E
Committee Action: Approved as Submitted

Committee Reason: The proposal provides clarification that the deflection limit for any edge of glass application shall not exceed L/175. The current code language does not specifically reference the L/175 limit from ASTM E 1300, "Standard Practice for Determining Load Resistance of Glass in Buildings", which is referenced in IBC Chapter 24 and is widely accepted in the glazing industry.

Assembly Action: None

S51-07/08

Committee Action: Approved as Submitted

Committee Reason: This proposal updates the steel deflection requirements for consistency with the current standards that are referenced for steel construction.

Assembly Action: None

S52-07/08

Committee Action: Approved as Modified

Modify proposal as follows:

1604.5 Occupancy category. Each buildings~~s~~ and structures~~s~~ shall be assigned an occupancy ~~categories~~ category in accordance with Table 1604.5.

1604.5.1 Multiple occupancies. Where a building or structure is occupied by two or more occupancies not included in the same occupancy category, it shall be assigned the classification of the highest occupancy category corresponding to the various occupancies. Where buildings or structures have two or more portions that are structurally separated, each portion shall be separately classified. Where a separated portion of a building or structure provides required access to, required egress from or shares life safety components with another portion having a higher occupancy category, both portions shall be assigned to the higher occupancy category.

Committee Reason: This code change provides terminology that is more consistent with Table 1604.5 and thus improves understanding of this provision. The modification makes further adjustments that are consistent with that intent.

Assembly Action: None

S53-07/08

Committee Action: Disapproved

Committee Reason: The application of the additional item under Category I was not clear and a proposed floor modification was not necessarily an improvement. The proponent is encouraged to address this in another manner via the public comment process.

Assembly Action: None

S54-07/08

Committee Action: **Disapproved**

Committee Reason: At the request of the proponent's representative, the code change was disapproved in hopes that a public comment can be submitted on this proposed change to Table 1604.5.

Assembly Action: **None**

S55-07/08

PART I – IBC STRUCTURAL
Committee Action: **Disapproved**

Committee Reason: The current definition of "Occupancy Category" makes the use of the term clear. This proposal could create confusion between the IBC and existing standards.

Assembly Action: **None**

PART II – IEBC
Committee Action: **Disapproved**

Committee Reason: Renaming occupancy category at this time could be potentially confusing, given that it will disagree with terminology used in ASCE7.

Assembly Action: **None**

S56-07/08

Committee Action: **Disapproved**

Committee Reason: The proposal was disapproved at the request of the proponent.

Assembly Action: **None**

S57-07/08

Committee Action: **Approved as Submitted**

Committee Reason: The minimum horizontal force of 280 pounds per foot that has been carried over from legacy codes is arbitrary and overly conservative.

Assembly Action: **None**

S58-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:

1604.8.3 Decks. Where supported by attachment to an exterior wall, decks shall be positively anchored to the primary structure and designed for both vertical and lateral loads as applicable. Such attachment shall not be accomplished by the use of toenails or nails subject to withdrawal. Where positive connection to the primary building structure cannot be verified during inspection, decks shall be self-supporting. In addition to the normal downward acting dead and live load reactions, ~~decks with cantilevered framing members,~~ connections to exterior walls or other framing members <u>of decks with cantilevered framing members</u> shall be designed and constructed to resist uplift resulting from the full live load specified in Table 1607.1 or snow load specified in Section 1608, whichever is greater, acting on the cantilevered portion of the deck, and no live load or snow load on the remaining portion of the ~~span~~ <u>deck</u>.

Committee Reason: This code change improves the code by clarifying the current required loading condition that affects the critical portion of the deck. The modification is editorial and further clarifies the intention of this code section.

Assembly Action: **None**

S59-07/08

Committee Action: **Disapproved**

Committee Reason: The committee feels that these provisions should not be related solely to occupancy category, since location may also be of interest. Group R-2 buildings should not be considered iconic structures. The forces driving these criteria are unknown (i.e. unanticipated loads) and it is not clear that these criteria should apply to all buildings. The committee would support the development of these requirements in a standard or as a technical document that can be applied where the Building Official needs to.

Assembly Action: **None**

S60-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:

1605.1 (Supp) General. Buildings and other structures and portions thereof shall be designed to resist:

1. The load combinations specified in Section 1605.2, 1605.3.1 or 1605.3.2,
2. The load combinations specified in Chapters 18 through 23, and
3. The load combinations with overstrength factor specified in Section 12.4.3.2 of ASCE 7 where required by Section 12.2.5.2, 12.3.3.3 or 12.10.2.1 of ASCE 7. With the simplified procedure of ASCE 7 Section 12.14, the load combinations with overstrength factor of Section 12.14.3.2 of ASCE 7 shall be used.

Applicable loads shall be considered, including both earthquake and wind, in accordance with the specified load combinations. Each load combination shall also be investigated with one or more of the variable loads set to zero.

Where the load combinations with overstrength factor in Section 12.4.3.2 of ASCE 7 apply, they shall be used as follows:

1. The Basic Combinations for Strength Design with Overstrength Factor in lieu of Equations 16-5 and 16-7 in Section 1605.2.1.
2. The Basic Combinations for Allowable Stress Design with Overstrength Factor in lieu of Equations 16-12, 16-13 and 16-15 in Section 1605.3.1.
3. The Basic Combinations for Allowable Stress Design with Overstrength Factor in lieu of Equations 16-20 and 16-21 in Section 1605.3.2.

(Portions of proposal not shown remain unchanged)

Committee Reason: The code change provides correlation with the ASCE 7 overstrength combinations. The modification clarifies when these load combinations are to be used.

Assembly Action: **None**

S61-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:

1605.1.1 Stability. Regardless of which load combinations are used to design for strength, where overall structure stability (such as stability against overturning, sliding, or buoyancy) is ~~checked using~~ being verified, use of the load combinations specified in Section 1605.2~~,~~ or 1605.3 shall be permitted. Where the load combinations specified in Section 1605.2 are used, strength reduction factors applicable to soil resistance shall be provided by a registered design professional. ~~qualified geotechnical engineer and consideration shall be given to acceptable behavior at service loads. Where structural elements are designed for strength using the~~

~~load combinations in 1605.2, it is permissible to check overall structure stability using the load combinations in 1605.3. Where the load combinations in 1605.3 are used to check overall structure stability, the dead load factor in each load combination shall be taken as 1.0 where the factors of safety in Section 1806.1 are applied.~~ <u>The stability of retaining wall shall be verified in accordance with Section 1807.2.3.</u>

Committee Reason: This proposal provides a clarification of the code with respect to stability. The modification improves the wording so that the intent is clearer and also coordinates with Chapter 18 requirements.

Assembly Action: None

S62-07/08

Committee Action: Approved as Submitted

Committee Reason: This proposal clarifies the notation for flood loads and makes the appropriate reference to ASCE 7.

Assembly Action: None

S63-07/08

Committee Action: Approved as Modified

Modify proposal as follows:

1605.3.1 Basic load combinations. Where allowable stress design (working stress design), as permitted by this code, is used, structures and portions thereof shall resist the most critical effects resulting from the following combinations of loads:

$D + F$	**(Equation 16-8)**
$D + H + F + L + T$	**(Equation 16-9)**
$D + H + F + (L_r \text{ or } S \text{ or } R)$	**(Equation 16-10)**
$D + H + F + 0.75(L + T) + 0.75\ (L_r \text{ or } S \text{ or } R)$	**(Equation 16-11)**
$D + H + F + (W \text{ or } 0.7E)$	**(Equation 16-12)**
$D + H + F + 0.75\ (W \text{ or } 0.7E) + 0.75L + 0.75\ (L_r \text{ or } S \text{ or } R)$	**(Equation 16-13)**
$0.6D + W + H$	**(Equation 16-14)**
$0.6D + 0.7E + H$	**(Equation 16-15)**

Exceptions:

1. Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m^2) or less and roof live loads<u> of 30 psf or less </u>need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.

Add the following new item:

1605.3.2 Alternative basic load combinations. In lieu of the basic load combinations specified in Section 1605.3.1, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following combinations. When using these alternative basic 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 wind loads are calculated in accordance with Chapter 6 of ASCE 7, the coefficient ω in the following equations shall be taken as 1.3. For other wind loads, ω shall be taken as 1. When 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. When 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.

$D + L + (L_r \text{ or } S \text{ or } R)$	**(Equation 16-16)**
$D + L + (\omega W)$	**(Equation 16-17)**
$D + L + \omega W + S/2$	**(Equation 16-18)**
$D + L + S + \omega W/2$	**(Equation 16-19)**
$D + L + S + E/1.4$	**(Equation 16-20)**
$0.9D + E/1.4$	**(Equation 16-21)**

Exceptions:

1. Crane hook loads need not be combined with roof live loads or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m^2) or less and roof live loads of 30 psf or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.

Committee Reason: This code changes brings consistency between strength load combinations and allowable stress load combinations. The modification limits applicability of the exception to roof live loads and also extends this allowance to the exception under the alternate basic allowable stress load combinations.

Assembly Action: **None**

S64-07/08 **Withdrawn by Proponent**

S65-07/08 **Withdrawn by Proponent**

S66-07/08

Committee Action: Approved as Modified

Modify proposal as follows:

1607.7.1.1 Concentrated load. Handrails and guards shall be able to resist a single concentrated load of 200 pounds (0.89 kN), applied in any direction at any point along the top, and ~~have attachment devices and supporting structure~~ to transfer this ~~loading to appropriate structural elements of the building~~ load through the supports to the structure. This load need not be assumed to act concurrently with the loads specified in ~~the preceding paragraph~~ Section 1607.7.1.

1607.7.2 Grab bars, shower seats and dressing room bench seats ~~benches~~. Grab bars ~~in accessible toilet and bathing facilities~~, shower seats ~~in accessible bathtubs and shower compartments~~ and dressing room bench seat systems ~~accessible benches in accordance with ICC A117.1~~ shall be designed to resist a single concentrated load of 250 pounds (1.11 kN) applied in any direction at any point.

1607.7.3 Vehicle barrier~~s~~ systems. Vehicle barrier systems for passenger ~~cars~~ vehicles shall be designed to resist a single load of 6,000 pounds (26.70 kN) applied horizontally in any direction to the barrier system and shall have anchorage or attachment capable of transmitting this load to the structure. For design of the system, the load shall be assumed to act at a minimum height of 1 foot, 6 inches (457 mm) above the floor or ramp surface on an area not to exceed 1 square foot (305 mm^2), and is not required to be assumed to act concurrently with any handrail or guard loadings specified in ~~the preceding paragraphs of~~ Section 1607.7.1. Garages accommodating trucks and buses shall be designed in accordance with an approved method that contains provision for traffic railings.

(Portions of proposal not shown remain unchanged)

Committee Reason: This is a clarification of the code, because it brings these revised sections into alignment with referenced standards. The modification also makes further improvements in the wording. The modification also retains the current wording of Section 1607.7.2 for grab bars because it was felt that grab bar design loads should apply to all grab bars regardless of the type of facility it is installed in.

Assembly Action: **None**

S67-07/08 **Withdrawn by Proponent**

S68-07/08

Committee Action: Approved as Submitted

Committee Reason: The change to wheel load area is consistent with the same requirement in ASCE 7.

Assembly Action: **None**

S69-07/08

Committee Action: Disapproved

Committee Reason: The committee disapproved this proposal at the recommendation of the proponent.

Assembly Action: None

S70-07/08

Committee Action: Approved as Modified

Modify proposal as follows:

1607.11.2.2 Special-purpose roofs. Roofs used for promenade purposes, roof gardens, assembly purposes or other special purposes shall be designed for a minimum live load as required in Table 1607.1. Such roof live loads are permitted to be reduced in accordance with 1607.9. Live loads of 100 psf (4.79 kN/m^2) <u>or more</u> at areas of roofs classified as Group A occupancies shall not be reduced.

(Portions of proposal not shown remain unchanged)

Committee Reason: This change clarifies the intent of the code's live load reduction for roofs with assembly occupancies. It is consistent with acceptance of code change S71-07/08. The modification covers areas where the live load happens to be greater than 100 psf.

Assembly Action: None

S71-07/08

Committee Action: Approved as Modified

Modify proposal as follows:

1607.11.1 Distribution of roof loads. Where uniform roof live loads are reduced to less than 20 psf (0.96 kN/m^2) in accordance with Section 1607.11.2.1 and are applied to the design of structural members arranged so as to create continuity, the reduced roof live load shall be applied to adjacent spans or to alternate spans, whichever produces the ~~greatest~~ <u>most</u> unfavorable <u>load</u> effect. See Section 1607.11.2 for minimum roof live loads and Section 7.5 of ASCE 7 for partial snow loading.

(Portions of proposal not shown remain unchanged)

Committee Reason: This proposal eliminates conflicts in the live load provisions and harmonizes them with the ASCE 7 standard. The modification is editorial to further clarify the required loading on continuous roof members.

Assembly Action: None

S72-07/08

PART I – IBC STRUCTURAL

Committee Action: Disapproved

Committee Reason: The application of the proposed loading to guard components is not clear. The 4 inch sphere requirement is for protection and the justification for combining it with this load requirement was not provided.

Assembly Action: None

PART II – IRC B/E

Committee Action: Disapproved

Committee Reason: The proposed wording would be in conflict with 25 years of baluster spacing, flexibility of infill components to include head and chest size. There was no justification given for the 50 pound load that was used. The existing language for guard opening limitations is preferred.

Assembly Action: None

S73-07/08

Committee Action: **Disapproved**

Committee Reason: Disapproved based the use of subjective terminology, particularly ductility – there are varying degrees of ductility. Designing a system that will absorb this kinetic energy is important, but as noted in the reason there is uncertainty on the appropriate vehicle weight. As worded, the proposal creates a liability without providing a clear means of assuring compliance. The proponent has provided valuable data on vehicle barriers. Other approaches that could be considered in future proposals could be increasing the design load or penalizing either precast concrete or the anchors.

Assembly Action: **None**

S74-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This proposal resolves a conflict in the live load reduction provision, by clearly stating the intention of these provisions.

Assembly Action: **None**

S75-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:

1607.9.1.4 Group A occupancies. Live loads of 100 psf (4.79 kN/m^2) <u>and at areas where fixed seats are located</u> shall not be reduced in Group A occupancies.

(Portions of proposal not shown remain unchanged)

Committee Reason: This code change makes adjustments in the live load requirements that further align the IBC with the ASCE 7 live load provisions. The modification clarifies that the restriction on live load reductions in Group A occupancies also includes areas with fixed seating.

Assembly Action: **None**

S76-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:

1607.9 Reduction in live loads. Except for uniform live loads at roofs ~~and marquees~~, all other minimum uniformly distributed live loads, L_o, in Table 1607.1 are permitted to be reduced in accordance with Section 1607.9.1 or 1607.9.2.

(Portions of proposal not shown remain unchanged)

Committee Reason: This proposal makes the roof live load requirements clearer. The modification removes marquees from Section 1607.9 to eliminate any confusion between that provisions and Section 1607.11.2.2.

Assembly Action: **None**

S77-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This proposal makes the alternate live load reduction method more consistent with the basic method, by adding an exception which can allow the reduction of high live loads.

Assembly Action: **None**

S78-07/08 **Withdrawn by Proponent**

S79-07/08

PART I – IBC STRUCTURAL
Committee Action: **Disapproved**

Committee Reason: While the new reference standard will replace an outdated standard, it is not yet complete. It is hoped that a public comment can be submitted to allow this standard to be referenced by the code.

Assembly Action: **None**

PART II – IRC B/E
Committee Action: **Approved as Submitted**

Committee Reason: This new standard, ICC-600 Standard for Residential Construction in High Wind Regions, will be a great improvement over the legacy standard SSTD 10. The standard provides a set of specifications that is consistent with the IBC and ASCE 7 wind loads, wind speed maps, and conventions.

Assembly Action: **None**

S80-07/08

Committee Action: **Approved as Submitted**

Committee Reason: The proposal adds an appropriate reference to the latest edition of the prescriptive cold-formed steel standard.

Assembly Action: **None**

S81-07/08

Committee Action: **Disapproved**

Committee Reason: The proposed standard has not been completed. It is hoped that a public comment can be submitted to allow this standard to be referenced by the code.

Assembly Action: **None**

S82-07/08

PART I – IBC STRUCTURAL
Committee Action: **Disapproved**

Committee Reason: Correcting the terminology for building height versus mean roof height needs to be carried out so as to avoid confusion. As proposed this may not be correct and could actually create conflicts.

Assembly Action: **None**

PART II – IRC B/E **Withdrawn by Proponent**

S83-07/08

Errata: Correct indicated Standards Organization as follows:

~~ANSI~~ Steel Door Institute
ANSI/SDI A250.13-XX Testing and Rating of Severe Windstorm Resistant Components for Swinging Door Assemblies.

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standard SDI A250.13 indicated that, in the opinion of ICC Staff, the standard did not comply with ICC standards criteria, Section 3.6.2.1.

Committee Action: **Disapproved**

Committee Reason: The newest edition of the standard is not yet completed, while the former edition did not meet ICC requirements for referenced standards.

Assembly Action: **None**

S84-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:

1609.6.1 Scope. As an alternate to ASCE 7 Section 6.5, the following provisions are permitted to be used to determine the wind effects on regularly shaped buildings, or other structures which meet all of the following conditions:

1. The building or other structure is less than or equal to 75 ~~100~~ feet (~~30480~~ 22 860 mm) in height, with a height to least width ratio of 4 or less, or the building or other structure has a fundamental frequency greater than or equal to 1 hertz.
2. The building or other structure is not sensitive to dynamic effects.
3. The building or other structure is not located on a site for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.
4. The building shall meet the requirements of a simple diaphragm building as defined in ASCE 7 Section 6.2.

TABLE 1609.6.2(2)
NET PRESSURE COEFFICIENTS, C_{net}[a, b, c]

STRUCTURE OR PART THEREOF	DESCRIPTION		C_{net} FACTOR	
1. Main Wind Force Resisting Frames and Systems	WALLS:		Enclosed	Part Enclosed
	Windward Wall		0.43	0.11
	Leeward Wall		~~-0.53~~ -0.51	-0.83
	Side Wall		-0.66	-0.97
	Parapet Wall	Windward	1.28	1.28
		Leeward	-0.85	-0.85
	ROOFS:		Enclosed	Part Enclosed
	Wind perpendicular to ridge			
	Leeward roof or flat roof		-0.66	-0.97
	Windward roof slopes:			
	Slope < 2:12 (10°)	Case 1	-1.09	-1.41
		Case 2	-0.28	-0.60
	Slope = 4:12 (18°)	Case 1	-0.73	-1.04
		Case 2	-0.05	-0.37
	Slope = 5:12 (23°)	Case 1	-0.58	-0.90
		Case 2	0.03	-0.29
	Slope = 6:12 (27°)	Case 1	-0.47	-0.78
		Case 2	~~0.20~~ 0.06	~~0.51~~ -0.25
	Slope = 7:12 (30°)	Case 1	-0.37	-0.68
		Case 2	~~0.30~~ 0.07	~~0.61~~ -0.25
	Slope 9:12 (37°)	Case 1	-0.27	-0.58
		Case 2	~~0.31~~ 0.14	~~0.63~~ -0.18
	Slope 12:12 (45°)		~~0.37~~ 0.14	~~0.68~~ -0.18
	Wind parallel to ridge and flat roofs		-1.09	-1.41

(Portions of proposal not shown remain unchanged)

Committee Reason: This proposal provides a needed alternative that addresses concerns over the complexity of the ASCE 7 Method 2 wind analysis. Exam committees have considered this complexity to be the primary cause in the disparity in answers given to test questions on Method 2. Adding this now is similar to the approach taken when adding the simplified wind method to the SBC and the 2000 IBC, which prompted the ASCE 7 committee to get it into their document. With the floor modifications this proposal is superior to S85-07/08. The modification places additional limitations on buildings that can qualify for this method of analysis, by eliminating flexible buildings and requiring simple diaphragm building. These address some of the concerns that were raised in connection with S85-07/08.

There were still some concerns with assumptions regarding internal pressurization. The resulting values are not necessarily conservative for Components and Cladding as well as MWFRS for some nonsymmetrical configurations. The same comment concerning the use of ASCE 7 figures that is noted on S85-07/08 applies to this change as well. It would have been preferable to include necessary figures in this proposal. There is also a concern with diaphragms. The method should probably be limited to rigid diaphragms. It is possible that horizontal truss elements could be unconservatively designed on the windward side because of the preference given to maximum negative values in the selection of these coefficients. The same concern in hurricane prone regions discussed in S85-07/08 applies to this proposal as well. The scope of application is too broad. There are possible issues down the road if we've begun using these values and ASCE 7 introduces a version of this method which provides different values.

Assembly Action: None

S85-07/08

Committee Action: Disapproved

Committee Reason: This proposal was formatted more like the UBC and was considered a step backwards, since the legacy code groups previously determined what the new format should be and it makes sense to stick with that format. The objections to this item were not necessarily on technical grounds, but the committee preferred the format used in S84-07/08. The floor modification that was offered contained significant changes in coefficients which also raised a red flag. There are certain assumptions that are made to arrive at c_{net} values as to what conditions are governing that may not actually be true for a particular configuration. One example would be the MWFRS where the selection is for maximum negative suction pressure on leeward and a minimum positive pressure on the forward side. Unless the building is a rigid diaphragm, there may be an issue. If for instance, one had a building that utilizes subdiapragms, then depending on where the subdiaphragm is, that system which is part of the MWFRS would not be conservatively designed on the forward side.

There is an inherent assumption of pressurization such that when c_{net} values are combined the assumption is that the c_{pi} occurs at the same level under consideration, where that may not actually be the case. It's probably a safe assumption for a multistory building with a large degree of compartmentalization between levels, but in a larger building with greater volume that has openings for ventilation and other purposes that c_{pi} value would not be correct because it would have to be calculated at the point where the opening occurs.

There is a new exception for torsional analysis that is based on separation of vertical elements of the MWFRS that simply puts a prescriptive separation of those elements without any specification of the stiffness of those elements. A building could have a very stiff element on one side with a very limber one on the other and the arrangement would be extremely sensitive to torsion, but would be excluded from torsional analysis under this procedure. The exception does not appear to follow any of the torsional criteria in ASCE 7, including earthquake provisions, for determining torsional behavior. The ASCE 7 figures that are referenced have values for Cp so the user will have to look at the figure to determine the point of discontinuity, but ignore the values of Cp represented on those figures as well as the footnotes and that can cause confusion.

The c_{net} values do not utilize the h/L assumption for low, squat buildings where positive pressure can occur due to reattachment. The proponent's responses indicate that this was because the concern was negative pressure. But that may not be the governing case particularly in hurricane prone regions where you have high rainfall. You can have possible blockage of the roof drainage which also must be considered. So positive pressure can be significant in a hurricane event that is combined with high roof loading and this is not considered in this procedure, perhaps because it was developed for localities that do not experience hurricanes.

It also appears that the procedure for MWFRS is not conservative if the building is not symmetrical. For a shed roof with a high slope one would find that because c_{net} pressures are based on maximum suction on the leeward and minimum positive in order to make the base shear work out to be the same, the MWFRS or C & C might be under designed under these conditions because it does not represent the correct positive pressure on the windward side. The foregoing issues are examples why this proposal needs to go through the ASCE 7 process which will provide a more thorough review.

Assembly Action: None

S86-07/08

Committee Action: Disapproved

Committee Reason: The proposed definition of "new construction" seems like an attempt to circumvent "substantial improvements". This definition would also include repairs.

Assembly Action: None

S87-07/08

Committee Action: **Disapproved**

Committee Reason: This proposal is not consistent with the NFIP. The committee is hopeful that FEMA and HUD cooperating will result in a more suitable proposal at some point.

Assembly Action: **None**

S88-07/08

Committee Action: **Disapproved**

Committee Reason: While this may already be a part of the federal regulations, the committee felt it includes ill-defined terminology. Particularly the phrase "anticipated flood hazard areas" leaves too much to interpretation.

Assembly Action: **None**

S89-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This code change adds a straightforward reference to the ASCE 7 flood load requirements, which has been missing from Section 1612.

Assembly Action: **None**

S90-07/08

Withdrawn by Proponent

S91-07/08

Committee Action: **Disapproved**

Committee Reason: The committee agrees with the proponent's suggestion of using the least R-value is the preferred approach but this is a change that should be made through the ASCE 7 committee and not in the IBC.

Assembly Action: **None**

S92-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:

1613.6.6 Minimum distance for building separation. All buildings and structures shall be separated from adjoining structures. Separations shall allow for the maximum inelastic response displacement (δ_M). δ_M shall be determined at critical locations with consideration for both translational and torsional displacements of the structure using Equation 16-45.

$$\delta_M = \frac{C_d \delta_{max}}{I}$$ **(Equation 16-45)**

Where

C_d = Deflection amplification factor in Table 12.2-1 of ASCE 7.

δ_{max} = Maximum displacement defined in Section 12.8.4.3 of ASCE 7

<u>I = Importance factor in accordance with Section 11.5.1 of ASCE 7.</u>

Adjacent buildings on the same property shall be separated by a distance not less than δ_{MT}, determined by Equation 16-46.

$$\delta_{MT} = \sqrt{(\delta_{M1})^2 + (\delta_{M2})^2} \qquad \textbf{(Equation 16-46)}$$

Where
δ_{M1}, δ_{M2} = The maximum inelastic response displacements of the adjacent buildings in accordance with Equation 16-45.

Where a structure adjoins a property line not common to a public way, the structure shall also be set back from the property line by not less than the maximum inelastic response displacement, δ_M, of that structure.

Exceptions:

1. Smaller separations or property line setbacks shall be permitted when justified by rational analyses.
2. Buildings and structures assigned to the Seismic Design Category A, B or C.

Committee Reason: Adding a requirement for building separation fills a void in the current earthquake provisions. The modification introduces the importance factor into the equation for determining maximum inelastic response to be consistent with ASCE 7.

Assembly Action: None

S93-07/08

Committee Action: Approved as Submitted

Committee Reason: This proposal eliminates unnecessary ductwork bracing for ducts that are small or where minimum prescriptive hangers are considered adequate.

Assembly Action: None

S94-07/08

Committee Action: Approved as Submitted

Committee Reason: This code change fills a void in the current ASCE 7 provision, by clarifying that it also applies to special steel plate shear wall systems.

Assembly Action: None

S95-07/08

Errata: Add omitted text from Sec. 12.14.7.5 of ASCE 7 as follows:

1613.7.2 ASCE 7, Section 12.14.7.5. Modify ASCE 7, Section 12.14.7.5 to read as follows:

12.14.7.5 Anchorage of Concrete or Masonry Structural Walls. Concrete or masonry structural walls shall be anchored to all floors, roofs and members that provide out-of-plane lateral support for the wall or that are supported by the wall. The anchorage shall provide a positive direct connection between the wall and floor, roof or supporting member with the strength to resist horizontal forces specified in this section for structures with flexible diaphragms or of Section 13.3.1 (using a_p and R_p equal to 2.5) for structures with diaphragms that are not flexible.

(Portions of proposal not shown remain unchanged)

Committee Action: Disapproved

Committee Reason: The affected ASCE 7 provision is a structural integrity issue and the committee prefers to keep the current ASCE 7 provision intact.

Assembly Action: None

S96-07/08

Committee Action: **Approved as Submitted**

Committee Reason: The committee believes this is a reasonable change to the ASCE 7 requirement for anchorage of walls and it is consistent with the committee's action on S57-07/08.

Assembly Action: **None**

S97-07/08

Withdrawn by Proponent

S98-07/08

Committee Action: **Disapproved**

Committee Reason: The committee prefers that this proposal for diaphragm anchorage forces be addressed by the ASCE 7 committee.

Assembly Action: **None**

S99-07/08

Committee Action: **Disapproved**

Committee Reason: If needed the earthquake instrumentation requirements would be more appropriately placed in an appendix chapter, since this is something that jurisdictions can decide on locally.

Assembly Action: **None**

S100-07/08

Committee Action: **Disapproved**

Committee Reason: This proposal was disapproved for essentially the same reason as S99-07/08, although the treatment of existing buildings may be preferable in this proposal.

Assembly Action: **None**

S101-07/08

Committee Action: **Disapproved**

Committee Reason: There is a need for some structural integrity measures and some committee members feel this proposal would be a good step. However, it appears, as proposed, the current ACI 318 provisions for concrete have been extended to other materials without adequate explanation. The logic in doing so is lacking. These provisions would involve too many buildings that do not have integrity issues and there is no demonstrated need for enhancing these structures. There are also concerns about the consequences of requiring these provisions for buildings that are currently built all the time. There is some concern regarding how, or if, this analysis would relate to other required loading conditions – in particular, lateral loads.

Assembly Action: **None**

S102-07/08

This code change was heard by the IBC Fire Safety Code Development Committee.

Committee Action: **Approved as Submitted**

Committee Reason: The committee agreed that definitions for mastic and intumescent fire resistant coatings were appropriate additions to the code because requirements for application and special inspection of these coatings are currently specified in Section 1704.11.

Assembly Action: **None**

S103-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:

1703.1.1 Independence. An approved agency shall be objective, competent and independent from the contractor responsible for the work ~~who's work is~~ being inspected. The agency shall also disclose possible conflicts of interest so that objectivity can be confirmed.

Committee Reason: This proposal elaborates on the requirement for an approved agency to be independent by adding that it is with respect to the contractor whose work is being inspected. The modification rewords this to make it clearer.

Assembly Action: **None**

S104-07/08

Committee Action: **Disapproved**

Committee Reason: The proposed exception to special inspection for conventional light-frame construction may not be necessary, because exception 2 already covers this for the most part. It would be a good idea to resubmit this proposal with narrower limits on the proposed exception.

Assembly Action: **None**

S105-07/08

Withdrawn by Proponent

S106-07/08

Withdrawn by Proponent

S107-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:

1704.1 (Supp) General. Where application is made for construction as described in this section, the owner or the registered design professional in responsible charge acting as the owner's agent shall employ one or more approved agencies ~~or provide special inspection services~~ to perform inspections during construction on the types of work listed under Section 1704. These inspections are in addition to the inspections identified in Section 109.

The special inspector shall be a qualified person who shall demonstrate competence, to the satisfaction of the building official, for the inspection of the particular type of construction or operation requiring special inspection. The registered design professional in responsible charge and engineers of record involved in the design of the project are permitted to act as the approved agency and their personnel are permitted to ~~inspect~~ act as the special inspector for the work designed by them, provided those personnel meet the qualification requirements of this section to the satisfaction of the building official. The special inspector shall provide written documentation to the building official demonstrating their competence and relevant experience or training. Experience or training shall be considered relevant when the documented experience or training is related in complexity to the same type of special inspection activities for projects of similar complexity and material qualities. These qualifications are in addition to qualifications specified in other sections of this code.

Exceptions:

1. Special inspections are not required for work of a minor nature or as warranted by conditions in the jurisdiction as approved by the building official.
2. Special inspections are not required for building components unless the design involves the practice of professional engineering or architecture as defined by applicable state statutes and regulations governing the professional registration and certification of engineers or architects.
3. Unless otherwise required by the building official, special inspections are not required for occupancies in Group U that are accessory to a residential occupancy including, but not limited to, those listed in Section 312.1.

1704.1.2 Report requirement. Special inspectors shall keep records of inspections. The special inspector shall furnish inspection reports to the building official, and to the registered design professional in responsible charge. Reports shall indicate that work inspected was or was not completed in conformance to approved construction documents. Discrepancies shall be brought to the immediate attention of the ~~registered design professional in responsible charge and the~~ contractor for correction. If the discrepancies are not corrected, the discrepancies shall be brought to the attention of the building official and to the registered design professional in responsible charge prior to the completion of that phase of the work. A final report documenting required special inspections and correction of any discrepancies noted in the inspections shall be submitted at a point in time agreed upon prior to the start of work by the permit applicant and the building official.

Committee Reason: This code change clarifies the code by stating that the registered design professional is permitted to do special inspections where they can demonstrate their qualifications to the satisfaction of the building official. The modification addresses some objections that were raised by clarifying that the registered design profession must demonstrate qualifications to the building official like any other potential special inspector. Special inspections still must be coordinated with the other inspections required by Section 109. It was noted that experience with permitting this in Hawaii has been positive.

Assembly Action: None

S108-07/08 — Withdrawn by Proponent

S109-07/08

Committee Action: Approved as Submitted

Committee Reason: This is a clarification that makes the applicability of special inspections to approved fabricators clearer.

Assembly Action: None

S110-07/08

Committee Action: Approved as Modified

Modify proposal as follows:

1704.3 Steel construction. The special inspections for steel elements of buildings and structures shall be as required by Section 1704.3 and Table 1704.3.

Exceptions:

1. Special inspection of the steel fabrication process shall not be required where the fabricator does not perform any welding, thermal cutting or heating operation of any kind as part of the fabrication process. In such cases, the fabricator shall be required to submit a detailed procedure for material control that demonstrates the fabricator's ability to maintain suitable records and procedures such that, at any time during the fabrication process, the material specification, grade and mill test reports for the main stress-carrying elements are capable of being determined.
2. The special inspector need not be continuously present during welding of the following items, provided the materials, welding procedures and qualifications of welders are verified prior to the start of the work; periodic inspections are made of the work in progress; and a visual inspection of all welds is made prior to completion or prior to shipment of shop welding.

2.1. Single-pass fillet welds not exceeding 5/16 inch (7.9 mm) in size.
2.2. Floor and roof deck welding.
2.3. Welded studs when used for structural diaphragm.
2.4. Welded sheet steel for cold-formed steel ~~light frame construction~~ members.
2.5. Welding of stairs and railing systems.

1704.3.1.1 Cold-formed steel. Welding inspection and welding inspector qualification for cold-formed steel~~,~~ ~~including metal~~ floor and roof decks~~,~~ shall be in accordance with AWS D1.3.

Committee Reason: This code change makes an important clarification for cold-formed steel. AWS D 1.3 is the appropriate reference standard for welding of cold formed steel. The modification is an editorial correction to make the scope of the reference clearer and consistent with the current extent of special inspections.

Assembly Action: **None**

S111-07/08

Committee Action: **Approved as Modified**

Modify proposal by adding new item as follows:

1704.3.1.3 Reinforcing steel. Welding inspection and welding inspector qualification for reinforcing steel shall be in accordance with AWS D1.4 and ACI 318.

(Portions of proposal not shown remain unchanged)

Committee Reason: This code change clarifies the applicable reference standard for welding and bolting of structural steel. The modification adds a new section to provide appropriate guidance on welding inspections for reinforcing steel.

Assembly Action: **None**

S112-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:

TABLE 1704.3
REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]	IBC REFERENCE
1. Material verification of high-strength bolts, nuts and washers:				
a. Identification markings to conform to ASTM standards specified in the approved construction documents.	—	X	AISC 360, Section A3.3 and applicable ASTM material standards	—
b. Manufacturer's certificate of compliance required	—	X	—	—
2. Inspection of high-strength bolting:				
a. Snug-tight joints.	—	X		
b. Pretensioned and slip-critical joints using turn-of-nut with matchmarking, twist-off bolt, or direct tension indicator methods of installation.	—	X	AISC 360, Section M2.5	1704.3.3
c. Pretensioned and slip-critical joints using turn-of-nut without matchmarking or calibrated wrench methods of installation.	X	—		
3. Material verification of structural steel and cold-formed steel deck:				

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]	IBC REFERENCE
a. For structural steel, identification markings to conform to AISC 360	—	X	AISC 360, Section M5.5	
b. For other steel, identification markings to conform to ASTM standards specified in the approved construction documents.	—	X	Applicable ASTM material standards	
c. Manufacturer's certified ~~mill~~ test reports.	—	X		
4. Material verification of weld filler materials:				
a. Identification markings to conform to AWS specification in the approved construction documents.	—	X	AISC 360, Section A3.5 and Applicable AWS A5 documents	—
b. Manufacturer's certificate of compliance required.	—	X	—	—
5. Inspection of welding: a. Structural steel and cold-formed steel deck:				
1) Complete and partial joint penetration groove welds.	X	—	AWS D1.1	1704.3.1
2) Multipass fillet welds	X	—		
3) Single-pass fillet welds > 5/16"	X	—		
4) Plug and slot welds	X	—		
5) Single-pass fillet welds ≤ 5/16"	—	X		
6) Floor and roof deck welds.	—	X	AWS D1.3	
b. Reinforcing steel:				
1) Verification of weldability of reinforcing steel other than ASTM A706	—	X	AWS D1.4 or ACI 318: Section 3.5.2	—
2) Reinforcing steel-resisting flexural and axial forces in intermediate and special moment frames, and boundary elements of special reinforced concrete shear walls and shear reinforcement.	X	—		
3) Shear reinforcement.	X	—		
4) Other reinforcing steel	—	X		
6. Inspection of steel frame joint details for compliance with approved construction documents:		~~X~~		
a. Details such as bracing and stiffening.	—	X	—	1704.3.2
b. Member locations.	—	X		
c. Application of joint details at each connection.	—	X		

Committee Reason: This proposal revises the table for inspection of steel construction to include appropriate nomenclature, making the table more user friendly. Requirements for slip-critical bolts are made clearer. The modification provides clarification that limits the inspection of cold-formed steel to floor and roof decks.

Assembly Action: None

S113-07/08

Withdrawn by Proponent

S114-07/08

Committee Action: Disapproved

Committee Reason: The committee's disapproval was requested by the proponent's spokesperson.

Assembly Action: None

S115-07/08

Committee Action: Approved as Modified

Modify proposal as follows:

1704.3.4 Cold-formed steel trusses spanning 60 feet or greater. Where a cold formed steel truss clear span is 60 feet (18288mm) or greater, the special inspector shall verify that the temporary installation restraint/bracing and the permanent individual truss member restraint/bracing are installed in accordance with the approved truss submittal package.

1704.6.2 Metal-plate connected wood trusses spanning 60 feet or greater. Where a truss clear span is 60 feet (18288 mm) or greater, the special inspector shall verify that the temporary installation restraint/bracing and the permanent individual truss member restraint/bracing are installed in accordance with the approved ~~permit drawings~~ truss submittal package.

Committee Reason: This proposal adds special inspections for long span trusses that should be spelled out in the code. The modification makes an editorial change under the section for wood trusses to make the wording consistent with other code sections.

Assembly Action: None

S116-07/08

Committee Action: Disapproved

Committee Reason: Rather than clarifying inspection of bolts in concrete the proposed wording could make things more confusing. Confusion was created by an apparent mixed reference to allowable stress design and strength. There is merit to requiring special inspection for the various types of concrete anchors, but the confusion should be resolved.

Assembly Action: None

S117-07/08

Committee Action: Disapproved

Committee Reason: With the testimony on this proposal as well as S116-07/08, it was obvious that there are unresolved issues and no consensus on just how to incorporate the needed inspections of concrete anchors at this time. The addition of special inspections for drilled-in anchors has merit, but the requirements are not yet clear enough.

Assembly Action: None

S118-07/08

Errata: Revise first deleted section under item 2 to read as follows:

~~**1708.1 Masonry.** Testing and verification of masonry materials and assemblies prior to construction shall comply with the requirements of Sections 1708.1.1 through 1708.1.4, depending on the classification of the building or structure or nature of the occupancy, as defined by this code.~~

Committee Action: Approved as Modified

Modify proposal as follows:

1704.5 Masonry construction. Masonry construction shall be inspected and verified in accordance with the requirements of Sections 1704.5.1 through 1704.5.~~3~~ 2, depending on the classification of the building or structure or nature of the occupancy, as defined by this code.

Exception: Special inspections shall not be required for:

1. Empirically designed masonry, glass unit masonry or masonry veneer designed by Section 2109, 2110 or Chapter 14, respectively, or by Chapter 5, 7 or 6 of ACI 530/ASCE 5/TMS 402, respectively, when they are part of structures classified as Occupancy Category I, II or III in accordance with Section 1604.5.

2. Masonry foundation walls constructed in accordance with Table 1805.5(1), 1805.5(2), 1805.5(3) or 1805.5(4).
3. Masonry fireplaces, masonry heaters or masonry chimneys installed or constructed in accordance with Section 2111, 2112 or 2113, respectively.

TABLE 1704.5.1
LEVEL 1 REQUIRED VERIFICATION AND INSPECTION OF MASONRY CONSTRUCTION

VERIFICATION AND INSPECTION	FREQUENCY OF INSPECTION		REFERENCE FOR CRITERIA		
	CONTINUOUS	PERIODIC	IBC SECTION	ACI 530/ASCE 5/TMS 402[a]	ACI 530.1/ASCE 6/TMS 602[a]
1. Compliance with required inspection provisions of the construction documents and the approved submittals shall be verified. ~~Verify compliance with the approved submittals~~	—	X	—	—	Art. 1.5

TABLE 1704.5.2
LEVEL 2 REQUIRED VERIFICATION AND INSPECTION OF MASONRY CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCE FOR CRITERIA		
			IBC SECTION	TMS 402/ACI 530/ ASCE 5[a]	TMS 602/ACI 530.1/ ASCE 6[a]
1. Compliance with required inspection provisions of the construction documents and the approved submittals ~~Verify compliance with the approved submittals~~	—	X	—	—	Art. 1.5
5 The following shall be verified to ensure compliance:					
a. Proportions of site-prepared mortar, grout and prestressing grout for bonded tendons.	—	X	—	—	Art. 2.6A
b. Placement of masonry units and construction of mortar joints.	—	X	—	—	Art. 3.3B
c. Placement of reinforcement, connectors and prestressing tendons and anchorages.	~~X~~	X	—	Sec. 1.13	Art. 3.4, 3.6A

Portions of proposal not shown do not change.

Committee Reason: This code change brings the IBC requirements for masonry inspections into better alignment with the referenced standards for masonry construction. The modification retains the current requirement for periodic inspection of placement of reinforcement, connectors and prestressing tendons and anchorages. It also makes appropriate wording changes for the purpose of consistency.

Assembly Action: None

S119-07/08

Committee Action: Disapproved

Committee Reason: The committee's disapproval was requested by the proponent's spokesperson.

Assembly Action: None

S120-07/08

Committee Action: Disapproved

Committee Reason: The committee's disapproval was requested by the proponent's spokesperson.

Assembly Action: None

S121-07/08

Committee Action: Approved as Submitted

Committee Reason: This is a straightforward change which makes it clear that compliance is based on the construction documents.

Assembly Action: None

S122-07/08

Errata: Revise Exception to Section1803.5 as follows:

> **Exception:** Compacted fill material ~~less than~~ 12 inches (305 mm) in depth or less need not comply with an approved report, provided it has been compacted to a minimum of 90 percent Modified Proctor in accordance with ASTM D 1557. The compaction shall be verified by ~~a qualified inspector approved by the building official~~ special inspection in accordance with Section 1704.7.

Committee Action: Approved as Modified

Modify proposal as follows:

1704.7 (Supp) Soils. Special inspections for existing site soil conditions, fill placement and load-bearing requirements shall be as required by this section and Table 1704.7. The approved geotechnical report, and the documents prepared by the registered design professional shall be used to determine compliance. During fill placement, the special inspector shall determine that proper materials and procedures are used in accordance with the provisions of the approved geotechnical report.

> **Exception:** Where Section 1803 does not require reporting of materials and procedures for fill placement, the special inspector shall verify that the in-place dry density of the compacted fill is ~~compacted to a minimum of~~ not less than 90 percent of the maximum dry density at optimum moisture content determined ~~Modified Proctor~~ in accordance with ASTM D 1557.

1803.5 Compacted fill material. Where footings will bear on compacted fill material, the compacted fill shall comply with the provisions of an approved report, which shall contain the following:

1. Specifications for the preparation of the site prior to placement of compacted fill material.
2. Specifications for material to be used as compacted fill.
3. Test method to be used to determine the maximum dry density and optimum moisture content of the material to be used as compacted fill.
4. Maximum allowable thickness of each lift of compacted fill material.
5. Field test method for determining the in-place dry density of the compacted fill.
6. Minimum acceptable in-place dry density expressed as a percentage of the maximum dry density determined in accordance with Item 3.
7. Number and frequency of field tests required to determine compliance with Item 6.

> **Exception:** Compacted fill material 12 inches (305 mm) in depth or less need not comply with an approved report, provided the in-place dry density is not less than ~~it has been compacted to a minimum of~~ 90 percent of the maximum dry density at optimum moisture content ~~Modified Proctor~~ in accordance with ASTM D 1557. The compaction shall be verified special inspection in accordance with Section 1704.7.

(Portions of proposal not shown remain unchanged)

Committee Reason: This proposal clarifies the requirements for special inspections of soil and fill. The modification rewords the exceptions for consistency with ASTM D 1557.

Assembly Action: None

S123-07/08

Committee Action: **Approved as Submitted**

Committee Reason: Recording concrete volume is a critical diagnostic tool. Adding it to the required special inspections for piers is consistent with current practice.

Assembly Action: **None**

S124-07/08

This code change was heard by the IBC Fire Safety Code Development Committee.

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standards ASTM E 2174-04 and E2393-04 indicated that, in the opinion of ICC Staff, the standards did not comply with ICC standards criteria, Section 3.6.2.1.

Committee Action: **Disapproved**

Committee Reason: The committee agreed that there was a lack of technical justification with respect to the exceptions of proposed Section 1704.15.1 for buildings less than 4 stories above grade and for installations by UL or FM certified contractors. The committee was also concerned that the proposed referenced standards (ASTM E2174 and E2393) contained nonmandatory language.

Assembly Action: **None**

S125-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:

1705.3 (Supp) Seismic resistance. The statement of special inspections shall include seismic requirements for the following cases:

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C, D, E or F, in accordance with Section 1613.

   <u>**Exception:** Requirements for the seismic-force resisting system are permitted to be excluded from the statement of special inspections for steel systems in structures assigned to Seismic Design Category C that are not specifically detailed for seismic resistance, with a response modification coefficient, R, of 3 or less, excluding cantilever column systems.</u>

2. Designated seismic systems in structures assigned to Seismic Design Category D, E or F.
3. The following additional systems and components in structures assigned to Seismic Design Category C:
   3.1. Heating, ventilating and air-conditioning (HVAC) ductwork containing hazardous materials and anchorage of such ductwork.
   3.2. Piping systems and mechanical units containing flammable, combustible or highly toxic materials.
   3.3. Anchorage of electrical equipment used for emergency or standby power systems.
4. The following additional systems and components in structures assigned to Seismic Design Category D:
   4.1. Systems required for Seismic Design Category C.
   4.2. Exterior wall panels and their anchorage.
   4.3. Suspended ceiling systems and their anchorage.
   4.4. Access floors and their anchorage.
   4.5. Steel storage racks and their anchorage, where the importance factor is equal to 1.5 in accordance with Section 15.5.3 of ASCE 7.
5. The following additional systems and components in structures assigned to Seismic Design Category E or F:
   5.1. Systems required for Seismic Design Categories C and D.
   5.2. Electrical equipment.

   Exception: Seismic requirements are permitted to be excluded from the statement of special inspections for structures designed and constructed in accordance with the following:

1. The structure consists of light-frame construction; the design spectral response acceleration at short periods, S_DS, is determined in Section 1613.5.4, does not exceed 0.5g; and the height of the structure does not exceed 35 feet (10 668 mm) above grade plane; or
2. The structure is constructed using a reinforced masonry structural system or reinforced concrete structural system; the design spectral response acceleration at short periods, *SDS*, as determined in Section 1613.5.4, does not exceed 0.5g, and the height of the structure does not exceed 25 feet (7620 mm) above grade plane; or
3. Detached one- or two-family dwellings not exceeding two stories above grade plane, provided the structure does not have any of the following plan or vertical irregularities in accordance with Section 12.3.2 of ASCE 7:
   3.1. Torsional irregularity.
   3.2. Nonparallel systems.
   3.3. Stiffness irregularity extreme soft story and soft story.
   3.4. Discontinuity in capacity weak story.
4. ~~Steel systems in structures that are assigned to Seismic Design Category C that are not specifically detailed for seismic resistance, with a response modification coefficient, R, of 3 or less, excluding cantilever column systems.~~

(Portions of proposal not shown remain unchanged)

Committee Reason: The committee agrees that the new exception is consistent with the intent of the current code and differentiates between designs under AISC 360 versus AISC 341. The modification puts the exception in a more appropriate location.

Assembly Action: **None**

S126-07/08

Committee Action: **Disapproved**

Committee Reason: The committee did not agree with the removal of roof cladding. Special inspection of cladding is important and should be retained.

Assembly Action: **None**

S127-07/08

Committee Action: **Approved as Submitted**

Committee Reason: There is no need for the section on contractor responsibility. Contractor responsibility is already implicit.

Assembly Action: **None**

S128-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:

1707.2 Structural steel. Special inspection for structural steel ~~members~~ shall be in accordance with the quality assurance plan requirements of AISC 341.

> **Exception:** Special inspections of structural steel in structures assigned to Seismic Design Category C that are not specifically detailed for seismic resistance, with a response modification coefficient, R, of 3 or less, excluding cantilever column systems.

Committee Reason: This proposal helps clarify special inspections for seismic resistance. The modification adds as exception that is consistent with the intent of the current code as well as the action taken on S125-07/08.

Assembly Action: **None**

S129-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:

1707.4 Cold-formed ~~steel and cold-formed~~ steel light frame construction. Periodic special inspection is required during welding operations of elements of the seismic-force-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the seismic-force-resisting system, including shear walls ~~struts~~, braces, diaphragms, collectors (drag struts) and hold-downs.

> **Exception:** Special inspection is not required for cold-formed steel light frame shear walls, braces, ~~shear panels and~~ diaphragms, collectors (drag struts), and hold downs ~~including screw attachment, bolting, anchoring and other fastening to other components of the seismic-force-resisting system,~~ where either of the following apply:
>
> 1. The sheathing is gypsum board or fiberboard.
> 2. The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c.).

Committee Reason: This proposal makes appropriate editorial corrections to special inspection requirements for cold-formed steel. The modification corrects the title to more accurately reflect the subject matter and makes the charging language consistent with the exception.

Assembly Action: **None**

S130-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This proposal is editorial and clearly defines the wall types in Exceptions 1 and 2.

Assembly Action: **None**

S131-07/08

Committee Action: **Disapproved**

Committee Reason: The proposed clarifications with respect to mechanical components with hazardous materials were deemed to be too broad compared to the current code requirement.

Assembly Action: **None**

S132-07/08

Committee Action: **Disapproved**

Committee Reason: The committee did not agree with removing special inspections during field gluing operations, since this is something that can't be inspected after the fact.

Assembly Action: **None**

S133-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:

1708.3 (Supp) Cold-formed steel light-frame construction. Periodic special inspection is required during welding operations of elements of the main wind-force-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the main wind-force-resisting system, including shear walls, ~~struts,~~ braces, diaphragms, collectors (drag struts) and hold downs.

Exception: Special inspection is not required for cold-formed steel light-frame shear walls, <u>braces,</u> ~~shear panels and~~ diaphragms, <u>collectors (drag struts) and hold downs</u> ~~including screw attachment, bolting, anchoring and other fastening to other components of the main wind-force-resisting system,~~ where either of the following apply:

1. The sheathing is gypsum board or fiberboard.
2. The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c.).

Committee Reason: This code change brings consistent and more concise language to the special inspection requirements for cold formed steel. The modification provides consistency between the exception and the charging language.

Assembly Action: None

S134-07/08

Committee Action: Disapproved

Committee Reason: It is not clear what type of labeling would be compliant were the proposed exception added to the code.

Assembly Action: None

S135-07/08

Committee Action: Approved as Submitted

Committee Reason: This proposal makes editorial changes to the requirements for testing for seismic resistance that help clarify the intent of this provision.

Assembly Action: None

S136-07/08

Committee Action: Approved as Modified

Modify proposal as follows:

1708.3 ~~Reinforcing and prestressing steel~~ <u>Concrete reinforcement</u>. ~~Certified mill test reports shall be provided for each shipment of reinforcement used to resist flexural and axial forces in reinforced concrete intermediate and special moment frames and boundary elements of special reinforced concrete and masonry shear walls.~~ Where reinforcement complying with ASTM A 615 is used to resist earthquake-induced flexural and axial forces in special moment frames<u>,</u> ~~and~~ special ~~reinforced concrete shear~~ <u>structural</u> walls <u>and coupling beams connecting special structural walls,</u> in structures assigned to Seismic Design Category <u>B,</u> C, D, E or F~~,~~ as determined in Section 1613, the ~~testing requirements of~~ <u>reinforcement shall comply with</u> Section ~~21.2.5~~ <u>21.1.5.2</u> of ACI 318 ~~shall be met~~. <u>Certified mill test reports shall be provided for each shipment of such reinforcement.</u> Where reinforcement complying with ASTM A 615 is to be welded, chemical tests shall be performed to determine weldability in accordance with Section 3.5.2 of ACI 318.

> ~~**Exception**: Certified mill test reports are not required to be provided for reinforcement complying with ASTM A 706.~~

Committee Reason: This code change to seismic testing of concrete reinforcement provides consistency with the ACI 318 standard. The modification makes further adjustments in the wording and eliminates the proposed exception to certified mill test reports.

Assembly Action: None

S137-07/08

Committee Action: Approved as Modified

Modify proposal as follows:

1708.4 Structural steel. Testing for structural steel shall be in accordance with the quality assurance plan requirements of AISC 341.

Exception: Testing for structural steel in structures assigned to Seismic Design Category C that are not specifically detailed for seismic resistance, with a response modification coefficient, R, of 3 or less, excluding cantilever column systems.

(Portions of proposal not shown remain unchanged)

Committee Reason: This proposal helps clarify structural steel testing requirements for seismic resistance. The modification adds as exception that is consistent with the intent of the current code as well as S125-07/08.

Assembly Action: None

S138-07/08

Committee Action: Approved as Submitted

Committee Reason: This code change clarifies the code requirement for seismic certification of designated seismic systems.

Assembly Action: None

S139-07/08

Committee Action: Approved as Submitted

Committee Reason: This proposal provides consistency with other occurrences of the term registered design professional.

Assembly Action: None

S140-07/08

Committee Action: Approved as Submitted

Committee Reason: This change is an editorial improvement that uses more accurate wording that is consistent with the earthquake provisions.

Assembly Action: None

S141-07/08

PART I – IBC STRUCTURAL
Committee Action: Disapproved

Committee Reason: The committee's disapproval was based on the proponent's request.

Assembly Action: None

PART II – IRC B/E
Committee Action: Disapproved

Committee Reason: This proposal would create a hardship on the side-hinge door industry. This is an onerous requirement for an unproven need. There is no evidence that this is a big water intrusion problem. This proposal goes too far in trying to solve the inconsistency between flashing and water intrusion. The proponent should work with the door industry and bring this back.

Assembly Action: None

S142-07/08

Committee Action: **Disapproved**

Committee Reason: Testimony indicates that this is a contentious issue and the committee feels there is too much uncertainty to remove the exemption for products meeting AAMA/WDMA/CSA 101/I.S.2/A440 at this time.

Assembly Action: **None**

S143-07/08

Errata: Correct indicated Standards Organization as follows:

~~ANSI~~ **<u>Steel Door Institute</u>**
ANSI/SDI A250.13-XX Testing and Rating of Severe Windstorm Resistant Components for Swinging Door Assemblies.

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standard ASDI A250.13 indicated that, in the opinion of ICC Staff, the standard did not comply with ICC standards criteria, 3.6.3.1.

Committee Action: **Disapproved**

Committee Reason: Disapproval was requested by the proponent because the proposed standard in not yet finalized.

Assembly Action: **None**

S144-07/08 **Withdrawn by Proponent**

S145-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This proposal improves consistency with the ASCE 7 standard when using allowable stress design.

Assembly Action: **None**

S146-07/08

Errata: Replace the entire proposal with the following:

Revise as follows:

106.1 [Supp] General. Submittal documents consisting of construction documents, statement of special inspections<u>, geotechnical report</u> and other data shall be submitted in one or more sets with each application for a permit. The construction documents shall be prepared by a registered design professional where required by the statutes of the jurisdiction in which the project is to be constructed. Where special conditions exist, the building official is authorized to require additional construction documents to be prepared by a registered design professional.

> **Exception:** The building official is authorized to waive the submission of construction documents and other data not required to be prepared by a registered design professional if it is found that the nature of the work applied for is such that review of construction documents is not necessary to obtain compliance with this code.

1610.1 General. Basement, foundation and retaining walls shall be designed to resist lateral soil loads. Soil loads specified in Table 1610.1 shall be used as the minimum design lateral soil loads unless ~~specified~~ <u>determined</u> otherwise ~~in~~ <u>by</u> a ~~soil~~ <u>geotechnical</u> investigation ~~report approved by the building official~~ <u>in accordance with Section 1803</u>. Basement walls and other walls in which horizontal movement is restricted at the top shall be designed for at-rest pressure. Retaining walls free to move and rotate at the top ~~are~~ shall be permitted to be designed for active pressure. Design lateral pressure from surcharge loads shall be added to the lateral earth pressure load. Design lateral pressure shall be increased if soils with expansion potential are present at the site.

> **Exception:** Basement walls extending not more than 8 feet (2438 mm) below grade and supporting flexible floor systems shall be permitted to be designed for active pressure.

SECTION ~~1802~~ 1803
~~FOUNDATION AND SOILS~~ GEOTECHNICAL INVESTIGATIONS

~~1802.1~~ 1803.1 General. ~~Foundation and soils~~ Geotechnical investigations shall be conducted in ~~conformance~~ accordance with ~~Sections 1802.2 through 1802.6~~ Section 1803.2 and reported in accordance with Section 1803.6. Where required by the building official ~~the classification and investigation of the soil~~ or where geotechnical investigations involve in-situ testing, laboratory testing, or engineering calculations, such investigations shall be ~~made~~ conducted by a registered design professional.

~~1802.2~~ 1803.2 ~~Where~~ Investigations required. ~~The owner or applicant shall submit a foundation and soils investigation to the building official where required in~~ Geotechnical investigations shall be conducted in accordance with Sections ~~1802.2.1 through 1802.2.7~~ 1803.3 through 1803.5.

> **Exception:** The building official ~~need not require~~ shall be permitted to waive the requirement for a ~~foundation or soils~~ geotechnical investigation where satisfactory data from adjacent areas is available that demonstrates an investigation is not necessary for any of the conditions in Sections ~~1802.2.1 through 1802.2.6~~ 1803.5.1 through 1803.5.6 and Sections 1803.5.10 and 1803.5.11.

~~**1802.3 Soil classification.** Where required, soils shall be classified in accordance with Section 1802.3.1 or 1802.3.2.~~

~~1802.4~~ 1803.3 Basis of investigation. Soil classification shall be based on observation and any necessary tests of the materials disclosed by borings, test pits or other subsurface exploration made in appropriate locations. Additional studies shall be made as necessary to evaluate slope stability, soil strength, position and adequacy of load-bearing soils, the effect of moisture variation on soil-bearing capacity, compressibility, liquefaction and expansiveness.

~~1802.4.1~~ 1803.3.1 ~~Exploratory boring~~ Scope of investigation. The scope of the ~~soil~~ geotechnical investigation including the number and types of borings or soundings, the equipment used to drill ~~and~~ or sample, the in-situ testing equipment and the laboratory testing program shall be determined by a registered design professional.

~~1802.5~~ 1803.4 ~~Soil boring and sampling~~ Qualified representative. The ~~soil boring and sampling~~ investigation procedure and apparatus shall be in accordance with generally accepted engineering practice. The registered design professional shall have a fully qualified representative on the site during all boring ~~and~~ or sampling operations.

1803.5 Investigated conditions. Geotechnical investigations shall be conducted as indicated in Sections 1803.5.1 through 1803.5.12.

~~1802.3.1~~ 1803.5.1 ~~General~~ Classification. ~~For the purposes of this chapter, the definition and classification of~~ Soil materials ~~for use in Table 1804.2~~ shall be classified in accordance with ASTM D 2487.

~~1802.2.1~~ 1803.5.2 Questionable soil. Where the classification, strength or compressibility of the soil is in doubt or where a load-bearing value superior to that specified in this code is claimed, the building official shall be permitted to require that ~~the necessary~~ a geotechnical investigation be ~~made~~ conducted. ~~Such investigation shall comply with the provisions of Sections 1802.4 through 1802.6.~~

~~1802.2.2~~ 1803.5.3 Expansive ~~soils~~ soil. In areas likely to have expansive soil, the building official shall require soil tests to determine where such soils do exist.

~~1802.3.2 Expansive soils.~~ Soils meeting all four of the following provisions shall be considered expansive, except that tests to show compliance with Items 1, 2 and 3 shall not be required if the test prescribed in Item 4 is conducted:

1. Plasticity index (PI) of 15 or greater, determined in accordance with ASTM D 4318.
2. More than 10 percent of the soil particles pass a No. 200 sieve (75 μm), determined in accordance with ASTM D 422.
3. More than 10 percent of the soil particles are less than 5 micrometers in size, determined in accordance with ASTM D 422.
4. Expansion index greater than 20, determined in accordance with ASTM D 4829.

~~1802.2.3~~ 1803.5.4 Ground-water table. A subsurface soil investigation shall be performed to determine whether the existing ground-water table is above or within 5 feet (1524 mm) below the elevation of the lowest floor level where such floor is located below the finished ground level adjacent to the foundation.

> **Exception:** A subsurface soil investigation to determine the location of the ground-water table shall not be required where waterproofing is provided in accordance with Section 1807.

~~**1802.2.4**~~ **1803.5.5 Deep ~~Pile and pier~~ foundations.** ~~Pile and pier foundations shall be designed and installed on the basis of a foundation investigation and report as specified in Sections 1802.4 through 1802.6 and Section 1808.2.1.~~ Where deep foundations will be used, a geotechnical investigation shall be conducted and shall include all of the following, unless sufficient data upon which to base the design and installation is otherwise available:

1. Recommended deep foundation types and installed capacities.
2. Recommended center-to-center spacing of deep foundation elements.
3. Driving criteria.
4. Installation procedures.
5. Field inspection and reporting procedures (to include procedures for verification of the installed bearing capacity where required).
6. Load test requirements.
7. Suitability of deep foundation materials for the intended environment.
8. Designation of bearing stratum or strata.
9. Reductions for group action, where necessary.

~~**1802.2.5**~~ **1803.5.6 Rock strata.** Where subsurface explorations at the project site indicate variations or doubtful characteristics in the structure of the rock upon which foundations are to be constructed, a sufficient number of borings shall be made to a depth of not less than 10 feet (3048 mm) below the level of the foundations to provide assurance of the soundness of the foundation bed and its load-bearing capacity.

1803.5.7 Excavation near foundations. Where excavation will remove lateral support from any foundation, an investigation shall be conducted to assess the potential consequences and address mitigation measures.

1803.5.8 Compacted fill material. Where shallow foundations will bear on compacted fill material more than 12 inches (305 mm) in depth, a geotechnical investigation shall be conducted and shall include all of the following:

1. Specifications for the preparation of the site prior to placement of compacted fill material.
2. Specifications for material to be used as compacted fill.
3. Test methods to be used to determine the maximum dry density and optimum moisture content of the material to be used as compacted fill.
4. Maximum allowable thickness of each lift of compacted fill material.
5. Field test method for determining the in-place dry density of the compacted fill.
6. Minimum acceptable in-place dry density expressed as a percentage of the maximum dry density determined in accordance with Item 3.
7. Number and frequency of field tests required to determine compliance with Item 6.

1803.5.9 Controlled low-strength material (CLSM). Where shallow foundations will bear on controlled low-strength material (CLSM), a geotechnical investigation shall be conducted and shall include all of the following:

1. Specifications for the preparation of the site prior to placement of the CLSM.
2. Specifications for the CLSM.
3. Laboratory or field test method(s) to be used to determine the compressive strength or bearing capacity of the CLSM.
4. Test methods for determining the acceptance of the CLSM in the field.
5. Number and frequency of field tests required to determine compliance with Item 4.

1803.5.10 Alternate setback and clearance. Where setbacks or clearances other than those required in Section 1805.3 are desired, the building official shall be permitted to require a geotechnical investigation by a registered design professional to demonstrate that the intent of Section 1805.3 would be satisfied. Such an investigation shall include consideration of material, height of slope, slope gradient, load intensity and erosion characteristics of slope material.

~~**1802.2.6**~~ **1803.5.11 Seismic Design ~~Category~~ Categories C through F.** ~~Where a~~ For structures ~~is determined~~ assigned to ~~be in~~ Seismic Design Category C, D, E, or F in accordance with Section 1613, ~~an~~ a geotechnical investigation shall be conducted, and shall include an evaluation of all the following potential geologic and seismic hazards ~~resulting from earthquake motions~~:

1. Slope instability.
2. Liquefaction.
3. Differential settlement. ~~and~~
4. Surface ~~rupture~~ displacement due to faulting or lateral spreading.

~~**1802.2.7**~~ **1803.5.12 Seismic Design ~~Category~~ Categories D~~, E or~~ through F.** ~~Where the~~ For structures ~~is determined~~ assigned to ~~be in~~ Seismic Design Category D, E or F~~,~~ in accordance with Section 1613, the ~~soils~~ geotechnical investigation ~~requirements for Seismic Design Category C, given in~~ required by Section ~~1802.2.6~~ 1803.5.11~~, shall be met, in addition to the following. The investigation~~ shall also include:

1. ~~A~~ The determination of lateral pressures on basement and retaining walls due to earthquake motions.
2. The potential for liquefaction and soil strength loss evaluated for site peak ground accelerations, magnitudes, and source characteristics consistent with the design earthquake ground motions. Peak ground acceleration shall be permitted to be determined based on a site-specific study taking into account soil amplification effects, as specified in Chapter 21 of ASCE 7, or, in the absence of such a study, peak ground accelerations shall be assumed equal to S_{DS} / 2.5, where S_{DS} is determined in accordance with Section 11.4 of ASCE 7.

3. An assessment of potential consequences of ~~any~~ liquefaction and soil strength loss, including estimation of differential settlement, lateral movement ~~or~~, lateral loads on foundations, reduction in foundation soil-bearing capacity, increases in lateral pressures on retaining walls and flotation of buried structures.
4. ~~and shall address~~ Discussion of mitigation measures~~. Such measures shall be given consideration in the design of the structure and can include, but are~~ such as, but not limited to, ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements and forces, or any combination of these measures and how they shall be considered in the design of the structure. ~~The potential for liquefaction and soil strength loss shall be evaluated for site peak ground acceleration magnitudes and source characteristics consistent with the design earthquake ground motions. Peak ground acceleration shall be determined from a site-specific study taking into account soil amplification effects, as specified in Chapter 21 of ASCE 7.~~

   ~~**Exception:** A site-specific study need not be performed provided that peak ground acceleration equal to S_{DS} / 2.5 is used, where S_{DS} is determined in accordance with Section 21.2.1 of ASCE 7.~~

~~1802.6~~ 1803.6 ~~Reports~~ Reporting. The soil classification and design load-bearing capacity shall be shown on the construction document. Where geotechnical investigations are required ~~by the building official~~, a written report of the investigations shall be submitted ~~that includes~~ to the building official by the owner or authorized agent at the time of permit application. This geotechnical report shall include, but need not be limited to, the following information:

1. A plot showing the location of ~~test borings and/or excavations~~ the soil investigations.
2. A complete record of the soil boring and penetration test logs and soil samples.
3. A record of the soil profile.
4. Elevation of the water table, if encountered.
5. Recommendations for foundation type and design criteria, including but not limited to: bearing capacity of natural or compacted soil; provisions to mitigate the effects of expansive soils; mitigation of the effects of liquefaction, differential settlement and varying soil strength; and the effects of adjacent loads.
6. Expected total and differential settlement.
7. ~~Pile and pier~~ Deep foundation information in accordance with Section ~~1808.2.2~~ 1803.5.5.
8. Special design and construction provisions for ~~footings or~~ foundations of structures founded on expansive soils, as necessary.
9. Compacted fill material properties and testing in accordance with Section ~~1803.5~~ 1803.5.8.
10. Controlled low-strength material properties and testing in accordance with Section 1803.5.9.

SECTION ~~1803~~ 1804
EXCAVATION, GRADING AND FILL

~~1803.1~~ 1804.1 ~~Excavations~~ Excavation near ~~footings or~~ foundations. ~~Excavations~~ Excavation for any purpose shall not remove lateral support from any ~~footing or~~ foundation without first underpinning or protecting the ~~footing or~~ foundation against settlement or lateral translation.

~~1803.2~~ 1804.2 Placement of backfill. The excavation outside the foundation shall be backfilled with soil that is free of organic material, construction debris, cobbles and boulders or with a controlled low-strength material (CLSM). The backfill shall be placed in lifts and compacted, in a manner that does not damage the foundation or the waterproofing or dampproofing material.

Exception: Controlled low-strength material need not be compacted.

~~1803.3~~ 1804.3 Site grading. The ground immediately adjacent to the foundation shall be sloped away from the building at a slope of not less than one unit vertical in 20 units horizontal (5-percent slope) for a minimum distance of 10 feet (3048 mm) measured perpendicular to the face of the wall. If physical obstructions or lot lines prohibit 10 feet (3048 mm) of horizontal distance, a 5-percent slope shall be provided to an approved alternate method of diverting water away from the foundation. Swales used for this purpose shall be sloped a minimum of 2 percent where located within 10 feet (3048 mm) of the building foundation. Impervious surfaces within 10 feet (3048 mm) of the building foundation shall be sloped a minimum of 2 percent away from the building.

Exception: Where climatic or soil conditions warrant, the slope of the ground away from the building foundation ~~is~~ shall be permitted to be reduced to not less than one unit vertical in 48 units horizontal (2-percent slope).

The procedure used to establish the final ground level adjacent to the foundation shall account for additional settlement of the backfill.

~~1803.4~~ 1804.4 Grading and fill in flood hazard areas. In flood hazard areas established in Section 1612.3, grading and/or fill shall not be approved:

1. Unless such fill is placed, compacted and sloped to minimize shifting, slumping and erosion during the rise and fall of floor water and, as applicable, wave action.
2. In floodways, unless it has been demonstrated through hydrologic and hydraulic analyses performed by a registered design professional in accordance with standard engineering practice that the proposed grading or fill, or both, will not result in any increase in flood levels during the occurrence of the design flood.

3. In flood hazard areas subject to high-velocity wave action, unless such fill is conducted and/or placed to avoid diversion of water and waves toward any building or structure.
4. Where design flood elevations are specified but floodways have not been designated, unless it has been demonstrated that the cumulative effect of the proposed floor hazard area encroachment, when combined with all other existing and anticipated floor hazard area encroachment, will not increase the design flood elevation more than one foot (305 mm) at any point.

~~1803.5~~ 1804.5 Compacted fill material. Where ~~footings~~ shallow foundations will bear on compacted fill material, the compacted fill shall comply with the provisions of an approved geotechnical report~~, which shall contain the following:~~ as set forth in Section 1803.

1. ~~Specifications for the preparation of the site prior to placement of compacted fill material.~~
2. ~~Specifications for material to be used as compacted fill.~~
3. ~~Test method to be used to determine the maximum dry density and optimum moisture content of the material to be used as compacted fill.~~
4. ~~Maximum allowable thickness of each lift of compacted fill material.~~
5. ~~Field test method for determining the in-place dry density of the compacted fill.~~
6. ~~Minimum acceptable in-place dry density expressed as a percentage of the maximum dry density determined in accordance with Item 3.~~
7. ~~Number and frequency of field tests required to determine compliance with Item 6.~~

Exception: Compacted fill material less than 12 inches (305 mm) in depth need not comply with an approved report, provided it has been compacted to a minimum of 90 percent Modified Proctor in accordance with ASTM D 1557. The compaction shall be verified by a qualified inspector approved by the building official.

~~1803.6~~ 1804.6 Controlled low-strength material (CLSM). Where ~~footings~~ shallow foundations will bear on controlled low-strength material (CLSM), the CLSM shall comply with the provisions of an approved geotechnical report, ~~which shall contain the following:~~ as set forth in Section 1803.

1. ~~Specifications for the preparation of the site prior to placement of the CLSM.~~
2. ~~Specifications for the CLSM.~~
3. ~~Laboratory or field test method(s) to be used to determine the compressive strength or bearing capacity of the CLSM.~~
4. ~~Test methods for determining the acceptance of the CLSM in the field.~~
5. ~~Number and frequency of field tests required to determine compliance with Item 4.~~

1805.3.5 Alternate setback and clearance. Alternate setbacks and clearances are permitted, subject to the approval of the building official. The building official ~~is~~ shall be permitted to require ~~an~~ a geotechnical investigation ~~and recommendation of a registered design professional to demonstrate that the intent of this section has been satisfied. Such an investigation shall include consideration of material, height of slope, slope gradient, load intensity and erosion characteristics of slope material~~ as set forth in Section 1803.5.10.

1808.2.2 General. Pier and pile foundations shall be designed and installed on the basis of a ~~foundation~~ geotechnical investigation as ~~defined~~ set forth in Section ~~1802~~ 1803 ~~unless sufficient data upon which to base the design and installation is available~~.

~~The investigation and report provisions of Section 1802 shall be expanded to include, but not be limited to, the following:~~

1. ~~Recommended pier or pile types and installed capacities.~~
2. ~~Recommended center-to-center spacing of piers or piles.~~
3. ~~Driving criteria.~~
4. ~~Installation procedures.~~
5. ~~Field inspection and reporting procedures (to include procedures for verification of the installed bearing capacity where required).~~
6. ~~Pier or pile load test requirements.~~
7. ~~Durability of pier or pile materials.~~
8. ~~Designation of bearing stratum or strata.~~
9. ~~Reductions for group action, where necessary.~~

1808.2.8.4 Allowable frictional resistance. The assumed frictional resistance developed by any pier or uncased cast-in-place pile shall not exceed one-sixth of the bearing value of the soil material at minimum depth as set forth in Table 1804.2, up to a maximum of 500 psf (24 kPa), unless a greater value is allowed by the building official after a ~~soil~~ geotechnical investigation as specified in Section ~~1802~~ 1803 is submitted or a greater value is substantiated by a load test in accordance with Section 1808.2.8.3. Frictional resistance and bearing resistance shall not be assumed to act simultaneously unless ~~recommended~~ determined by a ~~soil~~ geotechnical investigation ~~as specified~~ in accordance with Section ~~1802~~ 1803.

1808.2.10 Use of higher allowable pier or pile stresses. Allowable stresses greater than those specified for piers or for each pile type in Sections 1809 and 1810 are permitted where supporting data justifying such higher stresses is filed with the building official. Such substantiating data shall include:

1. A ~~soils~~ geotechnical investigation in accordance with Section ~~1802~~ 1803.
2. Pier or pile load tests in accordance with Section 1808.2.8.3, regardless of the load supported by the pier or pile.

The design and installation of the pier or pile foundation shall be under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pier or pile foundations who shall certify to the building official that the piers or piles as installed satisfy the design criteria.

3304.1.4 Fill supporting foundations. Fill to be used to support the foundations of any building or structure shall comply with Section ~~1803.5~~ 1804.5. Special inspections of compacted fill shall be in accordance with Section 1704.7.

J101.1 Scope. The provisions of this chapter apply to grading, excavation and earthwork construction, including fills and embankments. Where conflicts occur between the technical requirements of this chapter and the ~~soils~~ geotechnical report, the ~~soils~~ geotechnical report shall govern.

J104.3 Soils report. A ~~soils~~ geotechnical report prepared by a registered design ~~professionals~~ professional shall be provided ~~which~~. The report shall ~~identify~~ contain at least the following:

1. The nature and distribution of existing soils;
2. Conclusions and recommendations for grading procedures;
3. Soil design criteria for any structures or embankments required to accomplish the proposed grading; and
4. Where necessary, slope stability studies, and recommendations and conclusions regarding site geology.

Exception: A ~~soils~~ geotechnical report is not required where the building official determines that the nature of the work applied for is such that a report is not necessary.

J106.1 Maximum slope. The slope of cut surfaces shall be no steeper than is safe for the intended use, and shall be no steeper than 2 horizontal to 1 vertical (50 percent) unless the ~~applicant~~ owner or authorized agent furnishes a ~~soils~~ geotechnical report justifying a steeper slope.

Exceptions:

1. A cut surface may be at a slope of 1.5 horizontal to 1 vertical (67 percent) provided that all the following are met:
   - 1.1. It is not intended to support structures or surcharges.
   - 1.2. It is adequately protected against erosion.
   - 1.3. It is no more than 8 feet (2438 mm) in height.
   - 1.4. It is approved by the building official.
   - 1.5. Ground-water is not encountered.
2. A cut surface in bedrock shall be permitted to be at a slope of 1 horizontal to 1 vertical (100 percent).

J107.1 General. Unless otherwise recommended in the ~~soils~~ geotechnical report, fills shall ~~conform to~~ comply with the provisions of this section.

J107.6 Maximum slope. The slope of fill surfaces shall be no steeper than is safe for the intended use. Fill slopes steeper than 2 horizontal to 1 vertical (50 percent) shall be justified by ~~soils reports~~ a geotechnical report or engineering data.

Committee Action: Approved as Modified

Modify proposal as follows:

1803.5.4 Ground-water table. A subsurface soil investigation shall be performed to determine whether the existing ground-water table is above or within 5 feet (1524 mm) below the elevation of the lowest floor level where such floor is located below the finished ground level adjacent to the foundation.

Exception: A subsurface soil investigation to determine the location of the ground-water table shall not be required where waterproofing is provided in accordance with Section ~~1807~~ 1805.

1803.5.10 Alternate setback and clearance. Where setbacks or clearances other than those required in Section ~~1805.3~~ 1808.7 are desired, the building official shall be permitted to require a geotechnical investigation by a registered design professional to demonstrate that the intent of Section ~~1805.3~~ 1808.7 would be satisfied. Such an investigation shall include consideration of material, height of slope, slope gradient, load intensity and erosion characteristics of slope material.

1803.5.12 Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E or F in accordance with Section 1613, the geotechnical investigation required by Section 1803.5.11 shall also include:

1. The determination of lateral pressures on ~~basement~~ foundation walls and retaining walls due to earthquake motions.
2. The potential for liquefaction and soil strength loss evaluated for site peak ground accelerations, magnitudes, and source characteristics consistent with the design earthquake ground motions. Peak ground acceleration shall be permitted to be determined based on a site-specific study taking into account soil amplification effects, as specified in Chapter 21 of ASCE 7, or, in the absence of such a study, peak ground accelerations shall be assumed equal to S_{DS} / 2.5, where S_{DS} is determined in accordance with Section 11.4 of ASCE 7.
3. An assessment of potential consequences of liquefaction and soil strength loss, including estimation of differential settlement, lateral movement, lateral loads on foundations, reduction in foundation soil-bearing capacity, increases in lateral pressures on retaining walls and flotation of buried structures.
4. Discussion of mitigation measures such as, but not limited to, ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements and forces, or any combination of these measures and how they shall be considered in the design of the structure.

(Portions of proposal not shown remain unchanged)

Committee Reason: This proposal improves the code by removing redundancy and reorganizing the Chapter 18 provisions. The modification corrects section references to correlate with other approved changes and provides an editorial change to foundation walls for consistency in terminology.

Assembly Action: None

S147-07/08

Errata: Revise Exception to Section 1804.2 as follows:

> **Exception:** A presumptive load-bearing capacity ~~is~~ shall be permitted to be used where the building official deems the load-bearing capacity of mud, organic silt or unprepared fill is adequate for the support of lightweight ~~and~~ or temporary structures.

Committee Action: Approved as Modified

Modify proposal as follows:

1806.2 Presumptive load-bearing values. The load-bearing values used in design for supporting soils near the surface shall not exceed the values specified in Table 1806.2 unless data to substantiate the use of higher values are submitted and approved. Where the building official has reason to doubt the classification, strength, or compressibility of the soil, the requirements of Section ~~1802.2.1~~ 1803.5.2 shall be satisfied.

Presumptive load-bearing values shall apply to materials with similar physical characteristics and dispositions. Mud, organic silt, organic clays, peat or unprepared fill shall not be assumed to have a presumptive load-bearing capacity unless data to substantiate the use of such a value are submitted.

> **Exception:** A presumptive load-bearing capacity shall be permitted to be used where the building official deems the load-bearing capacity of mud, organic silt or unprepared fill is adequate for the support of lightweight or temporary structures.

1806.3.4 Increase for poles. Isolated poles for uses such as flagpoles or signs and poles used to support buildings that are not adversely affected by a 0.5 inch (12.7 mm) motion at the ground surface due to ~~wind or earthquake~~ short-term lateral loads shall be permitted to be designed using lateral bearing pressures equal to two times the tabular values.

(Portions of proposal not shown remain unchanged)

Committee Reason: This code change makes editorial changes that provide clarification of the current requirements. The modification correlates section references with other approved code changes and reinstates the original wording “short term lateral” loads which was more general than had been proposed.

Assembly Action: None

S148-07/08

PART I – IBC STRUCTURAL
Committee Action: Disapproved

Committee Reason: The proposal offered increased allowable pressures that were unsubstantiated. There also were concerns with undefined terms.

Assembly Action: None

PART II – IRC B/E
Committee Action: Disapproved

Committee Reason: The proposal would increase some allowable foundation pressures 30%. There was no technical data submitted to substantiate the allowable increase.

Assembly Action: None

S149-07/08

Errata: Replace the entire proposal with the following:

Revise as follows:

1610.1 General. ~~Basement,~~ Foundation walls and retaining walls shall be designed to resist lateral soil loads. Soil loads specified in Table 1610.1 shall be used as the minimum design lateral soil loads unless specified otherwise in a soil investigation report approved by the building official. ~~Basement~~ Foundation walls and other walls in which horizontal movement is restricted at the top shall be designed for at-rest pressure. Retaining walls free to move and rotate at the top ~~are~~ shall be permitted to be designed for active pressure. Design lateral pressure from surcharge loads shall be added to the lateral earth pressure load. Design lateral pressure shall be increased if soils ~~with expansion potential are present at the site~~ at the site are expansive. Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 1807.4.2 and 1807.4.3.

Exception: ~~Basement~~ Foundation walls extending not more than 8 feet (2438 mm) below grade and ~~supporting~~ laterally supported at the top by flexible ~~floor systems~~ diaphragms shall be permitted to be designed for active pressure.

TABLE 1610.1
~~SOIL~~ LATERAL SOIL LOAD

DESCRIPTION OF BACKFILL MATERIAL [c]	UNIFIED SOIL CLASSIFICATION	DESIGN LATERAL SOIL LOAD [a] (pounds per square foot per foot of depth) Active pressure	At-rest pressure
Well-graded, clean gravels; gravel-sand mixes	GW	30	60
Poorly graded clean gravels; gravel-sand mixes	GP	30	60
Silty gravels, poorly graded gravel-sand mixes	GM	40	60
Clayey gravels, poorly graded gravel-and-clay mixes	GC	45	60
Well-graded, clean sands; gravelly sand mixes	SW	30	60
Poorly graded clean sands; sand-gravel mixes	SP	30	60
Silty sands, poorly graded sand-silt mixes	SM	45	60
Sand-silt clay mix with plastic fines	SM-SC	45	100
Clayey sands, poorly graded sand-clay mixes	SC	60	100
Inorganic silts and clayey silts	ML	45	100
Mixture of inorganic silt and clay	ML-CL	60	100
Inorganic clays of low to medium plasticity	CL	60	100
Organic silts and silt clays, low plasticity	OL	Note b	Note b
Inorganic clayey silts, elastic silts	MH	Note b	Note b
Inorganic clays of high plasticity	CH	Note b	Note b
Organic clays and silty clays	OH	Note b	Note b

For SI: 1 pound per square foot per foot of depth = 0.157 kPa/m, 1 foot = 304.8 mm.

a. Design lateral soil loads are given for moist conditions for the specified soils at their optimum densities. Actual field conditions shall govern. Submerged or saturated soil pressures shall include the weight of the buoyant soil plus the hydrostatic loads.
b. Unsuitable as backfill material.
c. The definition and classification of soil materials shall be in accordance with ASTM D 2487.

SECTION 1807
FOUNDATION WALLS, RETAINING WALLS, AND EMBEDDED POSTS AND POLES

1807.1 Foundation walls. Foundation walls shall be designed and constructed in accordance with Sections 1807.1.1 through 1807.1.6. Foundation walls shall be supported by foundations designed in accordance with Section 1808.

1807.1.1 Design lateral soil loads. Foundation walls shall be designed for the lateral soil loads set forth in Section 1610.

1807.1.2 Unbalanced backfill height. Unbalanced backfill height is the difference in height between the exterior finish ground level and the lower of the top of the concrete footing that supports the foundation wall or the interior finish ground level. Where an interior concrete slab on grade is provided and is in contact with the interior surface of the foundation wall, the unbalanced backfill height shall be permitted to be measured from the exterior finish ground level to the top of the interior concrete slab.

~~1805.5.1.3~~ 1807.1.3 Rubble stone foundation walls. Foundation walls of rough or random rubble stone shall not be less than 16 inches (406 mm) thick. Rubble stone shall not be used for foundation~~s~~ walls for structures ~~in~~ assigned to Seismic Design Category C, D, E or F.

~~1805.4.6~~ 1807.1.4 Permanent wood foundation~~s~~ systems. Permanent wood foundation systems shall be designed and installed in accordance with AF&PA Technical Report No. 7. Lumber and plywood shall be treated in accordance with AWPA U1 (Commodity Specification A, Use Category 4B and Section 5.2) and shall be identified in accordance with Section 2303.1.8.1.

~~1805.5~~ 1807.1.5 Concrete and masonry foundation walls. Concrete and masonry foundation walls shall be designed in accordance with Chapter 19 or 21, ~~respectively~~ as applicable.

Exception: Concrete and masonry foundation walls ~~that are laterally supported at the top and bottom within the parameters of Tables 1805.5(1) through 1805.5(5) are~~ shall be permitted to be designed and constructed in accordance with ~~Sections 1805.5.1 through 1805.5.5~~ Section 1807.1.6.

1807.1.6 Prescriptive design of concrete and masonry foundation walls. Concrete and masonry foundation walls that are laterally supported at the top and bottom shall be permitted to be designed and constructed in accordance with this section.

~~**1805.5.1**~~ **1807.1.6.1 Foundation wall thickness.** ~~The minimum thickness of concrete and masonry foundation walls shall comply with Sections 1805.5.1.1 through 1805.5.1.3.~~

~~**1805.5.1.1 Thickness at top of foundation wall.**~~ The thickness of prescriptively designed foundation walls shall not be less than the thickness of the wall supported, except that foundation walls of at least 8 inch (203 mm) nominal width ~~are~~ shall be permitted to support brick-veneered frame walls and 10-inch-wide (254 mm) cavity walls provided the requirements of Section ~~1805.5.1.2~~ 1807.1.6.2 or 1807.1.6.3 are met. ~~Corbeling of masonry shall be in accordance with Section 2104.2. Where an 8-inch (203 mm) wall is corbeled, the top corbel shall not extend higher than the bottom of the floor framing and shall be a full course of headers at least 6 inches (152 mm) in length or the top course bed joint shall be tied to the vertical wall projection. The tie shall be W2.8 (4.8 mm) and spaced at a maximum horizontal distance of 36 inches (914 mm). The hollow space behind the corbelled masonry shall be filled with mortar or grout.~~

~~**1805.5.1.2 Thickness based on soil loads, unbalanced backfill height and wall height.** The thickness of foundation walls shall comply with the requirements of Table 1805.5(5) for concrete walls, Table 1805.5(1) for plain masonry walls or Table 1805.5(2), 1805.5(3) or 1805.5(4) for masonry walls with reinforcement. When using the tables, masonry shall be laid in running bond and the mortar shall be Type M or S.~~

~~Unbalanced backfill height is the difference in height between the exterior finish ground level and the lower of the top of the concrete footing that supports the foundation wall or the interior finish ground level. Where an interior concrete slab on grade is provided and is in contact with the interior surface of the foundation wall, the unbalanced backfill height is permitted to be measured from the exterior finish ground level to the top of the interior concrete slab.~~

~~**1805.5.2 Foundation wall materials.** Concrete foundation walls constructed in accordance with Table 1805.5(5) shall comply with Section 1805.5.2.1. Masonry foundation walls constructed in accordance with Table 1805.5(1), 1805.5(2), 1805.5(3) or 1805.5(4) shall comply with Section 1805.5.2.2.~~

~~**1805.5.2.1**~~ **1807.1.6.2 Concrete foundation walls**. Concrete foundation walls shall comply with the following:

1. The thickness shall comply with the requirements of Table 1807.1.6.2.
2. The size and spacing of vertical reinforcement shown in Table ~~1805(5)~~ 1807.1.6.2 is based on the use of reinforcement with a minimum yield strength of 60,000 psi (414 Mpa). Vertical reinforcement with a minimum yield strength of 40,000 psi (276 Mpa) or 50,000 psi (345 Mpa) ~~is~~ shall be permitted, provided the same size bar is used and the spacing shown in the table is reduced by multiplying the spacing by 0.67 or 0.83, respectively.
3. Vertical reinforcement, when required, shall be placed nearest the inside face of the wall a distance, d, from the outside face (soil face) of the wall. The distance, d, is equal to the wall thickness, t, minus 1.25 inches (32 mm) plus one-half the bar diameter, d_b, [$d = t - (1.25 + d_b / 2)$]. The reinforcement shall be placed within a tolerance of ± 3/8 inch (9.5 mm) where d is less than or equal to 8 inches (203 mm) or ± 1/2 inch (12.7 mm) where d is greater than 8 inches (203 mm).
4. In lieu of the reinforcement shown in Table ~~1805.5(5)~~ 1807.1.6.2, smaller reinforcing bar sizes with closer spacings that provide an equivalent cross-sectional area of reinforcement per unit length ~~are~~ shall be permitted.
5. Concrete cover for reinforcement measured from the inside face of the wall shall not be less than 3/4 inch (19.1 mm). Concrete cover for reinforcement measured from the outside face of the wall shall not be less than 1.5 inches (38 mm) for No. 5 bars and smaller, and not less than 2 inches (51 mm) for larger bars.
6. Concrete shall have a specified compressive strength, f_c', of not less than 2,500 psi (17.2 MPa) at 28 days.
7. The unfactored axial load per linear foot of wall shall not exceed $1.2\ t\ f_c'$ where t is the specified wall thickness in inches.

~~**1805.5.5.1**~~ **1807.1.6.2.1 Seismic requirements** ~~**for concrete foundation walls**~~**.** Based on the seismic design category assigned to the structure in accordance with Section 1613, concrete foundation walls designed using Table ~~1805.5(5)~~ 1807.1.6.2 shall be subject to the following limitations:

1. Seismic Design Categories A and B. No additional seismic requirements, except provide not less than two No. 5 bars around window and door openings. Such bars shall extend at least 24 inches (610 mm) beyond the corners of the openings.
2. Seismic Design Categories C, D, E and F. Tables shall not be used except as allowed for plain concrete members in Section 1908.1.15.

TABLE ~~1805.5(5)~~ 1807.1.6.2
CONCRETE FOUNDATION WALLS [b, c]

MAXIMUM WALL HEIGHT (feet)	MAXIMUM UNBALANCED BACKFILL HEIGHT[e] (feet)	VERTICAL REINFORCEMENT AND SPACING (inches)								
		Design lateral soil load[a] (psf per foot of depth)								
		30[d]			45[d]			60		
		Minimum wall thickness (inches)								
		7.5	9.5	11.5	7.5	9.5	11.5	7.5	9.5	11.5
5	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
6	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	PC	PC	PC
7	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5 at 48"	PC	PC
	7	PC	PC	PC	#5 at 46"	PC	PC	#6 at 48"	PC	PC
8	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5 at 43"	PC	PC
	7	PC	PC	PC	#5 at 41"	PC	PC	#6 at 43"	PC	PC
	8	#5 at 47"	PC	PC	#6 at 43"	PC	PC	#6 at 32"	#6 at 44"	PC
9	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5 at 39"	PC	PC
	7	PC	PC	PC	#5 at 37"	PC	PC	#6 at 38"	#5 at 37"	PC
	8	#5 at 41"	PC	PC	#6 at 38"	#5 at 37"	PC	#7 at 39"	#6 at 39"	#4 at 48"
	9[d]	#6 at 46"	PC	PC	#7 at 41"	#6 at 41"	PC	#7 at 31"	#7 at 41"	#6 at 39"
10	4	PC	PC	PC	PC	PC	PC	PC	PC	PC
	5	PC	PC	PC	PC	PC	PC	PC	PC	PC
	6	PC	PC	PC	PC	PC	PC	#5 at 37"	PC	PC
	7	PC	PC	PC	#6 at 48"	PC	PC	#6 at 35"	#6 at 48"	PC
	8	#5 at 38"	PC	PC	#7 at 47"	#6 at 47"	PC	#7 at 35"	#7 at 47"	#6 at 45"
	9[d]	#6 at 41"	#4 at 48"	PC	#7 at 37"	#7 at 48"	#4 at 48"	#6 at 22"	#7 at 37"	#7 at 47"
	10[d]	#7 at 45"	#6 at 45"	PC	#7 at 31"	#7 at 40"	#6 at 38"	#6 at 22"	#7 at 30"	#7 at 38"

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.

a. For design lateral soil loads ~~for different classes of soil~~, see Section 1610.
b. Provisions for this table are based on design and construction requirements specified in Section ~~1805.5.2.1~~ 1807.1.6.2.
c. "PC" means plain concrete.
d. Where unbalanced backfill height exceeds 8 feet and design lateral soil loads from Table 1610.1 are used, the requirements for 30 and 45 psf per foot of depth are not applicable. See Section 1610.
e. For height of unbalanced backfill, see Section ~~1805.5.1.2~~ 1807.1.2.

~~1805.5.2.2~~ 1807.1.6.3 Masonry foundation walls. Masonry foundation walls shall comply with the following:

1. The thickness shall comply with the requirements of Table 1807.1.6.3(1) for plain masonry walls or Table 1807.1.6.3(2), 1807.1.6.3(3) or 1807.1.6.3(4) for masonry walls with reinforcement.
2. Vertical reinforcement shall have a minimum yield strength of 60,000 psi (414 Mpa).
3. The specified location of the reinforcement shall equal or exceed the effective depth distance, *d*, noted in Tables ~~1805.5(2), 1805.5(3) and 1805.5(4)~~ 1807.1.6.3(2), 1807.1.6.3(3) and 1807.1.6.3(4) and shall be measured from the face of the exterior (soil) side of the wall to the center of the vertical reinforcement. The reinforcement shall be placed within the tolerances specified in ACI 530.1/ASCE 6/TMS 402, Article 3.4 B7 of the specified location.
4. Grout shall comply with Section 2103.12.
5. Concrete masonry units shall comply with ASTM C 90.
6. Clay masonry units shall comply with ASTM C 652 for hollow brick, except compliance with ASTM C 62 or C 216 ~~is~~ shall be permitted ~~when~~ where solid masonry units are installed in accordance with Table ~~1805.5(1)~~ 1807.1.6.3(1) for plain masonry.
7. Masonry units shall be laid in running bond and installed with Type M or S mortar in accordance with Section 2103.8.
8. The unfactored axial load per linear foot of wall shall not exceed $1.2\ t\ f'_m$ where t is the specified wall thickness in inches and f'_m is the specified compressive strength of masonry in pounds per square inch.
9. At least 4 inches (102 mm) of solid masonry shall be provided at girder supports at the top of hollow masonry unit foundation walls.
10. Corbeling of masonry shall be in accordance with Section 2104.2. Where an 8-inch (203 mm) wall is corbeled, the top corbel shall not extend higher than the bottom of the floor framing and shall be a full course of headers at least 6 inches (152 mm) in length or the top course bed joint shall be tied to the vertical wall projection. The tie shall be W2.8 (4.8 mm) and spaced at a maximum horizontal distance of 36 inches (914 mm). The hollow space behind the corbelled masonry shall be filled with mortar or grout.

~~1805.5.3~~ 1807.1.6.3.1 Alternative foundation wall reinforcement. In lieu of the reinforcement provisions for masonry foundation walls in Table ~~1805.5(2), 1805.5(3) or 1805.5(4)~~ 1807.1.6.3(2), 1807.1.6.3(3) or 1807.1.6.3(4), alternative reinforcing bar sizes and spacings having an equivalent cross-sectional area of reinforcement per linear foot (mm) of wall ~~are~~ shall be permitted to be used, provided the spacing of reinforcement does not exceed 72 inches (1829 mm) and reinforcing bar sizes do not exceed No. 11.

~~**1805.5.4 Hollow masonry walls.** At least 4 inches (102 mm) of solid masonry shall be provided at girder supports at the top of hollow masonry unit foundation walls.~~

~~**1805.5.5 Seismic requirements.** Tables 1805.5(1) through 1805.5(5) shall be subject to the following limitations in Sections 1805.5.5.1 and 1805.5.5.2 based on the seismic design category assigned to the structure as defined in Section 1613.~~

~~**1805.5.5.2**~~ **1807.1.6.3.2 Seismic requirements** ~~**for masonry foundation walls**~~**.** Based on the seismic design category assigned to the structure in accordance with Section 1613, masonry foundation walls designed using Tables ~~1805.5(1) through 1805.5(4)~~ 1807.1.6.3(1) through 1807.1.6.3(4) shall be subject to the following limitations:

1. Seismic Design Categories A and B. No additional seismic requirements.
2. Seismic Design Category C. A design using Tables ~~1805.5(1) through 1805.5(4)~~ 1807.1.6.3(1) through 1807.1.6.3(4) is subject to the seismic requirements of Section 2106.4.
3. Seismic Design Category D. A design using Tables ~~1805.2(2) through 1805.5(4)~~ 1807.1.6.3(2) through 1807.1.6.3(4) is subject to the seismic requirements of Section 2106.5.
4. Seismic Design Categories E and F. A design using Tables ~~1805.2(2) through 1805.5(4)~~ 1807.1.6.3(2) through 1807.1.6.3(4) is subject to the seismic requirements of Section 2106.6.

(Renumber remaining sections)

TABLE ~~1805.5(1)~~ 1807.1.6.3(1)
PLAIN MASONRY FOUNDATION WALLS [a, b, c]

MAXIMUM WALL HEIGHT (feet)	MAXIMUM UNBALANCED BACKFILL HEIGHT[e] (feet)	MINIMUM NOMINAL WALL THICKNESS (inches) ~~Soil classes and~~ Design lateral soil load[a] (psf per foot ~~below natural grade~~ of depth)		
		~~GW, GP, SW and SP soils~~ 30[f]	~~GM, GC, SM, SM-SC and ML soils~~ 45[f]	~~SC, ML-CL and Inorganic CL soils~~ 60
7	4 (or less)	8	8	8
	5	8	10	10
	6	10	12	10 (solid[c])
	7	12	10 (solid[c])	10 (solid[c])
8	4 (or less)	8	8	8
	5	8	10	12
	6	10	12	12 (solid[c])
	7	12	12 (solid[c])	Note d
	8	10 (solid[c])	12 (solid[c])	Note d
9	4 (or less)	8	8	8
	5	8	10	12
	6	12	12	12 (solid[c])
	7	12 (solid[c])	12 (solid[c])	Note d
	8	12 (solid[c])	Note d	Note d
	9[f]	Note d	Note d	Note d

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.

a. For design lateral soil loads, see Section 1610. ~~Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.~~

b. Provisions for this table are based on design and construction requirements specified in Section ~~1805.5.2.2~~ 1807.1.6.3.

c. Solid grouted hollow units or solid masonry units.

d. A design in compliance with Chapter 21 or reinforcement in accordance with Table ~~1805.5(2)~~ 1807.1.6.3(2) is required.

e. For height of unbalanced backfill, see Section ~~1805.5.1.2~~ 1807.1.2.

f. Where unbalanced backfill height exceeds 8 feet and design lateral soil loads from Table 1610.1 are used, the requirements for 30 and 45 psf per foot of depth are not applicable. See Section 1610.

TABLE ~~1805.5(2)~~ 1807.1.6.3(2)
8-INCH MASONRY FOUNDATION WALLS WITH REINFORCEMENT WHERE d ≥ 5 INCHES [a, b, c]

MAXIMUM WALL HEIGHT (feet-inches)	MAXIMUM UNBALANCED BACKFILL HEIGHT[d] (feet-inches)	VERTICAL REINFORCEMENT ~~Soil classes and~~ Design lateral soil load[a] (psf per foot ~~below natural grade~~ of depth)		
		~~GW, GP, SW and SP soils~~ 30[e]	~~GM, GC, SM, SM-SC and ML soils~~ 45[e]	~~SC, ML-CL and Inorganic CL soils~~ 60
7-4	4-0 (or less)	#4 at 48" o.c.	#4 at 48" o.c.	#4 at 48" o.c.
	5-0	#4 at 48" o.c.	#4 at 48" o.c.	#4 at 48" o.c.
	6-0	#4 at 48" o.c.	#5 at 48" o.c.	#5 at 48" o.c.
	7-4	#5 at 48" o.c.	#6 at 48" o.c.	#7 at 48" o.c.
8-0	4-0 (or less)	#4 at 48" o.c.	#4 at 48" o.c.	#4 at 48" o.c.
	5-0	#4 at 48" o.c.	#4 at 48" o.c.	#4 at 48" o.c.
	6-0	#4 at 48" o.c.	#5 at 48" o.c.	#5 at 48" o.c.
	7-0	#5 at 48" o.c.	#6 at 48" o.c.	#7 at 48" o.c.
	8-0	#5 at 48" o.c.	#6 at 48" o.c.	#7 at 48" o.c.

MAXIMUM WALL HEIGHT (feet-inches)	MAXIMUM UNBALANCED BACKFILL HEIGHT[d] (feet-inches)	VERTICAL REINFORCEMENT		
		~~Soil classes and~~ Design lateral soil load[a] (psf per foot ~~below natural grade~~ of depth)		
		~~GW, GP, SW and SP soils~~ 30[e]	~~GM, GC, SM, SM-SC and ML soils~~ 45[e]	~~SC, ML-CL and Inorganic CL soils~~ 60
8-8	4-0 (or less)	#4 at 48" o.c.	#4 at 48" o.c.	#4 at 48" o.c.
	5-0	#4 at 48" o.c.	#4 at 48" o.c.	#5 at 48" o.c.
	6-0	#4 at 48" o.c.	#5 at 48" o.c.	#6 at 48" o.c.
	7-0	#5 at 48" o.c.	#6 at 48" o.c.	#7 at 48" o.c.
	8-8[e]	#6 at 48" o.c.	#7 at 48" o.c.	#8 at 48" o.c.
9-4	4-0 (or less)	#4 at 48" o.c.	#4 at 48" o.c.	#4 at 48" o.c.
	5-0	#4 at 48" o.c.	#4 at 48" o.c.	#5 at 48" o.c.
	6-0	#4 at 48" o.c.	#5 at 48" o.c.	#6 at 48" o.c.
	7-0	#5 at 48" o.c.	#6 at 48" o.c.	#7 at 48" o.c.
	8-0	#6 at 48" o.c.	#7 at 48" o.c.	#8 at 48" o.c.
	9-4[e]	#7 at 48" o.c.	#8 at 48" o.c.	#9 at 48" o.c.
10-0	4-0 (or less)	#4 at 48" o.c.	#4 at 48" o.c.	#4 at 48" o.c.
	5-0	#4 at 48" o.c.	#4 at 48" o.c.	#5 at 48" o.c.
	6-0	#4 at 48" o.c.	#5 at 48" o.c.	#6 at 48" o.c.
	7-0	#5 at 48" o.c.	#6 at 48" o.c.	#7 at 48" o.c.
	8-0	#6 at 48" o.c.	#7 at 48" o.c.	#8 at 48" o.c.
	9-0[e]	#7 at 48" o.c.	#8 at 48" o.c.	#9 at 48" o.c.
	10-0[e]	#7 at 48" o.c.	#9 at 48" o.c.	#9 at 48" o.c.

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.

a. For design lateral soil loads, see Section 1610. ~~Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.~~

b. Provisions for this table are based on design and construction requirements specified in Section ~~1805.5.2.2~~ 1807.1.6.3.

c. For alternative reinforcement, see Section ~~1805.5.3~~ 1807.1.6.3.1.

d. For height of unbalanced backfill, see Section ~~1805.5.1.2~~ 1807.1.2.

e. Where unbalanced backfill height exceeds 8 feet and design lateral soil loads from Table 1610.1 are used, the requirements for 30 and 45 psf per foot of depth are not applicable. See Section 1610.

TABLE ~~1805.5(3)~~ 1807.1.6.3(3)
10-INCH MASONRY FOUNDATION WALLS WITH REINFORCEMENT WHERE d ≥ 6.75 INCHES [a, b, c]

MAXIMUM WALL HEIGHT (feet-inches)	MAXIMUM UNBALANCED BACKFILL HEIGHT[d] (feet-inches)	VERTICAL REINFORCEMENT		
		~~Soil classes and~~ Design lateral soil load[a] (psf per foot ~~below natural grade~~ of depth)		
		~~GW, GP, SW and SP soils~~ 30[e]	~~GM, GC, SM, SM-SC and ML soils~~ 45[e]	~~SC, ML-CL and Inorganic CL soils~~ 60
7-4	4-0 (or less)	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	5-0	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	6-0	#4 at 56" o.c.	#4 at 56" o.c.	#5 at 56" o.c.
	7-4	#4 at 56" o.c.	#5 at 56" o.c.	#6 at 56" o.c.
8-0	4-0 (or less)	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	5-0	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	6-0	#4 at 56" o.c.	#4 at 56" o.c.	#5 at 56" o.c.
	7-0	#4 at 56" o.c.	#5 at 56" o.c.	#6 at 56" o.c.
	8-0	#5 at 56" o.c.	#6 at 56" o.c.	#7 at 56" o.c.
8-8	4-0 (or less)	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	5-0	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	6-0	#4 at 56" o.c.	#4 at 56" o.c.	#5 at 56" o.c.
	7-0	#4 at 56" o.c.	#5 at 56" o.c.	#6 at 56" o.c.
	8-8[e]	#5 at 56" o.c.	#7 at 56" o.c.	#8 at 56" o.c.
9-4	4-0 (or less)	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	5-0	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	6-0	#4 at 56" o.c.	#5 at 56" o.c.	#5 at 56" o.c.
	7-0	#4 at 56" o.c.	#5 at 56" o.c.	#6 at 56" o.c.
	8-0	#5 at 56" o.c.	#6 at 56" o.c.	#7 at 56" o.c.
	9-4[e]	#6 at 56" o.c.	#7 at 56" o.c.	#8 at 56" o.c.
10-0	4-0 (or less)	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	5-0	#4 at 56" o.c.	#4 at 56" o.c.	#4 at 56" o.c.
	6-0	#4 at 56" o.c.	#5 at 56" o.c.	#5 at 56" o.c.
	7-0	#5 at 56" o.c.	#6 at 56" o.c.	#7 at 56" o.c.
	8-0	#5 at 56" o.c.	#7 at 56" o.c.	#8 at 56" o.c.
	9-0[e]	#6 at 56" o.c.	#7 at 56" o.c.	#9 at 56" o.c.
	10-0[e]	#7 at 56" o.c.	#8 at 56" o.c.	#9 at 56" o.c.

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.

a. For design lateral soil loads, see Section 1610. ~~Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.~~

b. Provisions for this table are based on design and construction requirements specified in Section ~~1805.5.2.2~~ 1807.1.6.3.
c. For alternative reinforcement, see Section ~~1805.5.3~~ 1807.1.6.3.1.
d. For height of unbalanced backfill, see Section ~~1805.5.1.2~~ 1807.1.2.
e. Where unbalanced backfill height exceeds 8 feet and design lateral soil loads from Table 1610.1 are used, the requirements for 30 and 45 psf per foot of depth are not applicable. See Section 1610.

TABLE ~~1805.5(4)~~ 1807.1.6.3(4)
12-INCH MASONRY FOUNDATION WALLS WITH REINFORCEMENT WHERE d ≥ 8.75 INCHES [a, b, c]

MAXIMUM WALL HEIGHT (feet-inches)	MAXIMUM UNBALANCED BACKFILL HEIGHT[d] (feet-inches)	VERTICAL REINFORCEMENT		
		~~Soil classes and~~ Design lateral soil load[a] (psf per foot ~~below natural grade~~ of depth)		
		~~GW, GP, SW and SP soils~~ 30[e]	~~GM, GC, SM, SM-SC and ML soils~~ 45[e]	~~SC, ML-CL and Inorganic CL soils~~ 60
7-4	4-0 (or less)	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	5-0	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	6-0	#4 at 72" o.c.	#4 at 72" o.c.	#5 at 72" o.c.
	7-4	#4 at 72" o.c.	#5 at 72" o.c.	#6 at 72" o.c.
8-0	4-0 (or less)	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	5-0	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	6-0	#4 at 72" o.c.	#4 at 72" o.c.	#5 at 72" o.c.
	7-0	#4 at 72" o.c.	#5 at 72" o.c.	#6 at 72" o.c.
	8-0	#5 at 72" o.c.	#6 at 72" o.c.	#7 at 72" o.c.
8-8	4-0 (or less)	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	5-0	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	6-0	#4 at 72" o.c.	#4 at 72" o.c.	#5 at 72" o.c.
	7-0	#4 at 72" o.c.	#5 at 72" o.c.	#6 at 72" o.c.
	8-8[e]	#5 at 72" o.c.	#7 at 72" o.c.	#8 at 72" o.c.
9-4	4-0 (or less)	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	5-0	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	6-0	#4 at 72" o.c.	#5 at 72" o.c.	#5 at 72" o.c.
	7-0	#4 at 72" o.c.	#5 at 72" o.c.	#6 at 72" o.c.
	8-0	#5 at 72" o.c.	#6 at 72" o.c.	#7 at 72" o.c.
	9-4[e]	#6 at 72" o.c.	#7 at 72" o.c.	#8 at 72" o.c.
10-0	4-0 (or less)	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	5-0	#4 at 72" o.c.	#4 at 72" o.c.	#4 at 72" o.c.
	6-0	#4 at 72" o.c.	#5 at 72" o.c.	#5 at 72" o.c.
	7-0	#4 at 72" o.c.	#6 at 72" o.c.	#6 at 72" o.c.
	8-0	#5 at 72" o.c.	#6 at 72" o.c.	#7 at 72" o.c.
	9-0[e]	#6 at 72" o.c.	#7 at 72" o.c.	#8 at 72" o.c.
	10-0[e]	#7 at 72" o.c.	#8 at 72" o.c.	#9 at 72" o.c.

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot per foot = 0.157 kPa/m.
a. For design lateral soil loads, see Section 1610. ~~Soil classes are in accordance with the Unified Soil Classification System and design lateral soil loads are for moist soil conditions without hydrostatic pressure.~~
b. Provisions for this table are based on design and construction requirements specified in Section ~~1805.5.2.2~~ 1807.1.6.3.
c. For alternative reinforcement, see Section ~~1805.5.3~~ 1807.1.6.3.1.
d. For height of unbalanced backfill, see Section ~~1805.5.1.2~~ 1807.1.2.
e. Where unbalanced backfill height exceeds 8 feet and design lateral soil loads from Table 1610.1 are used, the requirements for 30 and 45 psf per foot of depth are not applicable. See Section 1610.

~~SECTION 1806~~
~~RETAINING WALLS~~

1807.2 Retaining walls. Retaining walls shall be designed in accordance with Sections 1807.2.1 through 1807.2.3.

~~1806.1~~ 1807.2.1 General. Retaining walls shall be designed to ensure stability against overturning, sliding, excessive foundation pressure and water uplift.

1807.2.2 Design lateral soil loads. Retaining walls shall be designed for the lateral soil loads set forth in Section 1610.

1807.2.3 Safety factor. Retaining walls shall be designed ~~for~~ to resist the lateral action of soil to produce sliding and overturning with a safety factor of 1.5 ~~against lateral sliding and overturning~~. The load combinations of Section 1605.3 shall not apply to these requirements.

~~1805.7~~ 1807.3 ~~Designs employing lateral bearing~~ Embedded posts and poles. Designs to resist both axial and lateral loads employing posts or poles as columns embedded in earth or embedded in concrete footings in the earth shall ~~conform to the requirements of~~ be in accordance with Sections ~~1805.7.1 through 1805.7.3~~ 1807.3.1 through 1807.3.3.

~~1805.7.1~~ 1807.3.1 Limitations. The design procedures outlined in this section are subject to the following limitations:

1. The frictional resistance for structural walls and slabs on silts and clays shall be limited to one-half of the normal force imposed on the soil by the weight of the footing or slab.
2. Posts embedded in earth shall not be used to provide lateral support for structural or nonstructural materials such as plaster, masonry or concrete unless bracing is provided that develops the limited deflection required.

Wood poles shall be treated in accordance with AWPA U1 for sawn timber posts (Commodity Specification A, Use Category 4B), and for round timber posts (Commodity Specification B, Use Category 4B).

~~**1805.7.2**~~ **1807.3.2** **Design criteria.** The depth to resist lateral loads shall be determined ~~by~~ using the design criteria established in Sections ~~1805.7.2.1 through 1805.7.2.3~~ 1807.3.2.1 through 1807.3.2.3, or by other methods approved by the building official.

~~**1805.7.2.1**~~ **1807.3.2.1** **Nonconstrained.** The following formula shall be used in determining the depth of embedment required to resist lateral loads where no lateral constraint is provided at the ground surface, such as by a rigid floor or rigid ground surface pavement, and where no lateral constraint is provided above the ground surface, such as by a structural diaphragm.

$$d = 0.5\,A\,\{1 + [1 + (4.36\,h / A)]^{1/2}\} \qquad \textbf{(Equation 18-1)}$$

where:

A = 2.34 P / S_1 b.
b = Diameter of round post or footing or diagonal dimension of square post or footing, feet (m).
d = Depth of embedment in earth in feet (m) but not over 12 feet (3658 mm) for purpose of computing lateral pressure.
h = Distance in feet (m) from ground surface to point of application of "P."
P = Applied lateral force in pounds (kN).
S_1 = Allowable lateral soil-bearing pressure as set forth in Section 1804.3 based on a depth of one-third the depth of embedment in pounds per square foot (psf) (kPa).

~~**1805.7.2.2**~~ **1807.3.2.2** **Constrained.** The following formula shall be used to determine the depth of embedment required to resist lateral loads where lateral constraint is provided at the ground surface, such as by a rigid floor or pavement.

$$d^2 = 4.25\,(P\,h / S_3\,b) \qquad \textbf{(Equation 18-2)}$$

or alternatively

$$d^2 = 4.25\,(M_g / S_3\,b) \qquad \textbf{(Equation 18-3)}$$

where:

M_g = Moment in the post at grade, in foot-pounds (kN-m).
S_3 = Allowable lateral soil-bearing pressure as set forth in Section 1804.3 based on a depth equal to the depth of embedment in pounds per square foot (kPa).

~~**1805.7.2.3**~~ **1807.3.2.3** **Vertical load.** The resistance to vertical loads shall be determined ~~by~~ using the ~~allowable soil-bearing~~ vertical foundation pressure set forth in Table 1804.2.

~~**1805.7.3**~~ **1807.3.3** **Backfill.** The backfill in the annular space around columns not embedded in poured footings shall be by one of the following methods:

1. Backfill shall be of concrete with an ultimate strength of 2,000 psi (13.8 MPa) at 28 days. The hole shall not be less than 4 inches (102 mm) larger than the diameter of the column at its bottom or 4 inches (102 mm) larger than the diagonal dimension of a square or rectangular column.
2. Backfill shall be of clean sand. The sand shall be thoroughly compacted by tamping in layers not more than 8 inches (203 mm) in depth.
3. Backfill shall be of controlled low-strength material (CLSM).

Delete without substitution:

~~**1805.5.6 Foundation wall drainage.** Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 1807.4.2 and 1807.4.3.~~

~~**1805.6 Foundation plate or sill bolting.** Wood foundation plates or sills shall be bolted or strapped to the foundation or foundation wall as provided in Chapter 23.~~

Committee Action: Approved as Modified

Modify proposal as follows:

1610.1 General. Foundation walls and retaining walls shall be designed to resist lateral soil loads. Soil loads specified in Table 1610.1 shall be used as the minimum design lateral soil loads unless specified otherwise in a soil investigation report approved by the building official. Foundation walls and other walls in which horizontal movement is restricted at the top shall be designed for at-rest pressure. Retaining walls free to move and rotate at the top shall be permitted to be designed for active pressure. Design lateral pressure from surcharge loads shall be added to the lateral earth pressure load. Design lateral pressure shall be increased if soils at the site are expansive. Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections ~~1807.4.2 and 1807.4.3~~ 1805.4.2 and 1805.4.3.

Exception: Foundation walls extending not more than 8 feet (2438 mm) below grade and laterally supported at the top by flexible diaphragms shall be permitted to be designed for active pressure.

1807.1.6.2.1 Seismic requirements. Based on the seismic design category assigned to the structure in accordance with Section 1613, concrete foundation walls designed using Table 1807.1.6.2 shall be subject to the following limitations:

1. Seismic Design Categories A and B. No additional seismic requirements, except provide reinforcement around openings in accordance with Section 1909.6.3. ~~not less than two No. 5 bars around window and door openings. Such bars shall extend at least 24 inches (610 mm) beyond the corners of the openings.~~
2. Seismic Design Categories C, D, E and F. Tables shall not be used except as allowed for plain concrete members in Section 1908.1.15.

1807.2.3 Safety factor. Retaining walls shall be designed to resist the lateral action of soil to produce sliding and overturning with a minimum safety factor of 1.5 in each case. The load combinations of Section 1605~~.3~~ shall not apply to ~~these requirements~~ this requirement. Instead, design shall be based on 0.7 times nominal earthquake loads, 1.0 times other nominal loads, and investigation with one or more of the variable loads set to zero.

1807.3.2.1 Nonconstrained. The following formula shall be used in determining the depth of embedment required to resist lateral loads where no lateral constraint is provided at the ground surface, such as by a rigid floor or rigid ground surface pavement, and where no lateral constraint is provided above the ground surface, such as by a structural diaphragm.

$d = 0.5\,A\,\{1 + [1 + (4.36\,h / A)]^{1/2}\}$ **(Equation 18-1)**

where:

A = 2.34 P / S_1 b.
b = Diameter of round post or footing or diagonal dimension of square post or footing, feet (m).
d = Depth of embedment in earth in feet (m) but not over 12 feet (3658 mm) for purpose of computing lateral pressure.
h = Distance in feet (m) from ground surface to point of application of "P."
P = Applied lateral force in pounds (kN).
S_1 = Allowable lateral soil-bearing pressure as set forth in Section ~~1804.3~~ 1806.2 based on a depth of one-third the depth of embedment in pounds per square foot (psf) (kPa).

1807.3.2.2 Constrained. The following formula shall be used to determine the depth of embedment required to resist lateral loads where lateral constraint is provided at the ground surface, such as by a rigid floor or pavement.

$d^2 = 4.25\,(P\,h / S_3\,b)$ **(Equation 18-2)**

or alternatively

$d^2 = 4.25\,(M_g / S_3\,b)$ **(Equation 18-3)**

where:

M_g = Moment in the post at grade, in foot-pounds (kN-m).
S_3 = Allowable lateral soil-bearing pressure as set forth in Section ~~1804.3~~ 1806.2 based on a depth equal to the depth of embedment in pounds per square foot (kPa).

1807.3.2.3 Vertical load. The resistance to vertical loads shall be determined using the vertical foundation pressure set forth in Table ~~1804.2~~ 1806.2.

(Portions of proposal not shown remain unchanged)

Committee Reason: The approval is consistent with other committee actions on proposals that reformat Chapter 18. The modification provides correlation of section references with other approved code changes. The modification to Section 1807.1.6.2.1 provides coordination with Chapter 19 requirements. The modification to Section 1807.2.3 makes editorial changes, clarifies that 0.7 times the earthquake load should be used and coordinates this section with Section 1605.

Assembly Action: None

S150-07/08

Errata: Replace the entire proposal with the following:

Revise as follows:

1801.2 Design basis. Allowable bearing pressures, allowable stresses and design formulas provided in this chapter shall be used with the allowable stress design load combinations specified in Section 1605.3. The quality and design of materials used structurally in excavations~~, footings~~ and foundations shall ~~conform to~~ comply with the requirements specified in Chapters 16, 19, 21, 22 and 23 of this code. Excavations and fills shall also comply with Chapter 33.

SECTION ~~1805~~ 1808
~~FOOTINGS AND~~ FOUNDATIONS

1808.1 General. Foundations shall be designed and constructed in accordance with Sections 1808.2 through 1808.9. Shallow foundations shall also satisfy the requirements of Section 1809. Deep foundations shall also satisfy the requirements of Section 1810.

~~**1805.4 Footings.** Footings shall be designed and constructed in accordance with Sections 1805.4.1 through 1805.4.6.~~

~~1805.4.1~~ 1808.2 Design for capacity and settlement. ~~Footings~~ Foundations shall be so designed that the allowable bearing capacity of the soil is not exceeded, and that differential settlement is minimized. ~~The minimum width of footings shall be 12 inches (305 mm).~~ ~~Footings~~ Foundations in areas with expansive soils shall be designed in accordance with the provisions of Section ~~1805.8~~ 1808.6.

~~1805.4.1.1~~ 1808.3 Design loads. ~~Footings~~ Foundations shall be designed for the most unfavorable effects due to the combinations of loads specified in Section 1605.2 or 1605.3. The dead load is permitted to include the weight of foundations~~, footings~~ and overlying fill. Reduced live loads, as specified in Sections 1607.9 and 1607.11, ~~are~~ shall be permitted to be used in the design of ~~footings~~ foundations.

~~1801.2.1~~ 1808.3.1 ~~Foundation design for~~ Seismic overturning. Where ~~the foundation is~~ foundations are proportioned using the load combinations of Section 1605.2, and the computation of seismic overturning ~~moment~~ effects is by ~~the~~ Equivalent Lateral Force ~~method~~ Analysis or ~~the~~ Modal Analysis ~~method~~, the proportioning shall be in accordance with Section 12.13.4 of ASCE 7.

~~1805.4.1.2~~ 1808.4 Vibratory loads. Where machinery operations or other vibrations are transmitted through the foundation, consideration shall be given in the ~~footing~~ foundation design to prevent detrimental disturbances of the soil.

~~1805.2.3~~ 1808.5 Shifting or moving soils. Where it is known that the shallow subsoils are of a shifting or moving character, ~~footings~~ foundations shall be carried to a sufficient depth to ensure stability.

~~1805.8~~ 1808.6 Design for expansive soils. ~~Footings or~~ Foundations for buildings and structures founded on expansive soils shall be designed in accordance with Section ~~1805.8.1 or 1805.8.2~~ 1808.6.1 or 1808.6.2.

> **Exception:** ~~Footing or~~ Foundation design need not comply with Section ~~1805.8.1 or 1805.8.2~~ 1808.6.1 or 1808.6.2 where one of the following conditions is satisfied:
>
> 1. The soil is removed in accordance with Section ~~1805.8.3, nor where~~ 1808.6.3; or
> 2. The building official approves stabilization of the soil in accordance with Section ~~1805.8.4~~ 1808.6.4.

~~1805.8.1~~ 1808.6.1 Foundations. ~~Footings or~~ Foundations placed on or within the active zone of expansive soils shall be designed to resist differential volume changes and to prevent structural damage to the supported structure. Deflection and racking of the supported structure shall be limited to that which will not interfere with the usability and serviceability of the structure.

Foundations placed below where volume change occurs or below expansive soil shall comply with the following provisions:

1. Foundations extending into or penetrating expansive soils shall be designed to prevent uplift of the supported structure.
2. Foundations penetrating expansive soils shall be designed to resist forces exerted on the foundation due to soil volume changes or shall be isolated from the expansive soil.

~~1805.8.2~~ 1808.6.2 Slab-on-ground foundations. Moments, shears and deflections for use in designing slab-on-ground, mat or raft foundations on expansive soils shall be determined in accordance with *WRI/CRSI Design of Slab-on-ground Foundations* or *PTI Standard Requirements for Analysis of Shallow Concrete Foundations on Expansive Soils*. Using the moments, shears and deflections determined above, nonprestressed slabs-on-ground, mat or raft foundations on expansive soils shall be designed in accordance with *WRI/CRSI Design of Slab-on-ground Foundations* and post-tensioned slab-on-ground, mat or raft foundations on expansive soils shall be designed in accordance with *PTI Standard Requirements for Design of Shallow Post-tensioned Concrete Foundations on Expansive Soils*. It shall be permitted to analyze and design such slabs by other methods that account for soil-structure interaction, the deformed shape of the soil support, the plate or stiffened plate action of the slab, as well as both center lift and edge lift conditions. Such alternative methods shall be rational and the basis for all aspects and parameters of the method shall be available for peer review.

~~1805.8.3~~ 1808.6.3 Removal of expansive soil. Where expansive soil is removed in lieu of designing ~~footings or~~ foundations in accordance with Section ~~1805.8.1 or 1805.8.2~~ 1808.6.1 or 1808.6.2, the soil shall be removed to a depth sufficient to ensure a constant moisture content in the remaining soil. Fill material shall not contain expansive soils and shall comply with Section 1803.5 or 1803.6.

Exception: Expansive soil need not be removed to the depth of constant moisture, provided the confining pressure in the expansive soil created by the fill and supported structure exceeds the swell pressure.

~~1805.8.4~~ 1808.6.4 Stabilization. Where the active zone of expansive soils is stabilized in lieu of designing ~~footings or~~ foundations in accordance with Section ~~1805.8.1 or 1805.8.2~~ 1808.6.1 or 1808.6.2, the soil shall be stabilized by chemical, dewatering, presaturation or equivalent techniques.

~~1805.3~~ 1808.7 ~~Footings~~ Foundations on or adjacent to slopes. The placement of buildings and structures on or adjacent to slopes steeper than one unit vertical in three units horizontal (33.3-percent slope) shall ~~conform to~~ comply with Sections ~~1805.3.1 through 1805.3.5~~ 1808.7.1 through 1808.7.5.

~~1805.3.1~~ 1808.7.1 Building clearance from ascending slopes. In general, buildings below slopes shall be set a sufficient distance from the slope to provide protection from slope drainage, erosion and shallow failures. Except as provided ~~for~~ in Section ~~1805.3.5~~ 1808.7.5 and Figure ~~1805.3.1~~ 1808.7.1, the following criteria will be assumed to provide this protection. Where the existing slope is steeper than one unit vertical in one unit horizontal (100-percent slope), the toe of the slope shall be assumed to be at the intersection of a horizontal plane drawn from the top of the foundation and a plane drawn tangent to the slope at an angle of 45 degrees (0.79 rad) to the horizontal. Where a retaining wall is constructed at the toe of the slope, the height of the slope shall be measured from the top of the wall to the top of the slope.

FIGURE ~~1805.3.1~~ 1808.7.1
FOUNDATION CLEARANCES FROM SLOPES

(No changes to Figure)

~~1805.3.2~~ 1808.7.2 ~~Footing~~ Foundation setback from descending slope surface. ~~Footings~~ Foundations on or adjacent to slope surfaces shall be founded in firm material with an embedment and set back from the slope surface sufficient to provide vertical and lateral support for the ~~footing~~ foundation without detrimental settlement. Except as provided for in Section ~~1805.3.5~~ 1808.7.5 and Figure ~~1805.3.1~~ 1808.7.1, the following setback is deemed adequate to meet the criteria. Where the slope is steeper than 1 unit vertical in 1 unit horizontal (100-percent slope), the required setback shall be measured from an imaginary plane 45 degrees (0.79 rad) to the horizontal, projected upward from the toe of the slope.

~~1805.3.3~~ 1808.7.3 Pools. The setback between pools regulated by this code and slopes shall be equal to one-half the building footing setback distance required by this section. That portion of the pool wall within a horizontal distance of 7 feet (2134 mm) from the top of the slope shall be capable of supporting the water in the pool without soil support.

~~1805.3.4~~ 1808.7.4 Foundation elevation. On graded sites, the top of any exterior foundation shall extend above the elevation of the street gutter at point of discharge or the inlet of an approved drainage device a minimum of 12 inches (305 mm) plus 2 percent. Alternate elevations are permitted subject to the approval of the building official, provided it can be demonstrated that required drainage to the point of discharge and away from the structure is provided at all locations on the site.

~~1805.3.5~~ 1808.7.5 Alternate setback and clearance. Alternate setbacks and clearances are permitted, subject to the approval of the building official. The building official is permitted to require an investigation and recommendation of a registered design professional to demonstrate that the intent of this section has been satisfied. Such an investigation shall include consideration of material, height of slope, slope gradient, load intensity and erosion characteristics of slope material.

~~1805.4.2~~ 1808.8 Concrete ~~footings~~ foundations. The design, materials and construction of concrete ~~footings~~ foundations shall comply with Sections ~~1805.4.2.1 through 1805.4.2.6~~ 1808.8.1 through 1808.8.6 and the provisions of Chapter 19.

Exception: Where ~~a specific design is not provided,~~ concrete footings supporting walls of light-frame construction are ~~permitted to be~~ designed in accordance with Table ~~1805.4.2~~ 1809.7, a specific design in accordance with Chapter 19 is not required.

~~1805.4.2.1~~ 1808.8.1 Concrete or grout strength and mix proportioning. Concrete or grout in ~~footings~~ foundations shall have a specified compressive strength ($f'c$) ~~of~~ not less than ~~2,500 pounds per square inch (psi) (17 237 kPa) at 28 days~~ the largest applicable value indicated in Table 1808.8.1.

~~**1810.1.1 Materials.** Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 2,500 psi (17.24 MPa).~~ Where concrete is placed through a funnel hopper at the top of a deep foundation element ~~the pile~~, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 6 inches (152 mm). Where concrete or grout is to be pumped, the mix design including slump shall be adjusted to produce a pumpable mixture ~~concrete~~.

TABLE 1808.8.1
MINIMUM SPECIFIED COMPRESSIVE STRENGTH, f'_c, OF CONCRETE OR GROUT

CONDITION	SPECIFIED COMPRESSIVE STRENGTH, $f'c$
1. Foundations for structures assigned to Seismic Design Category A, B, or C	2,500 psi (17.24 MPa)
2a. Foundations for Group R or U occupancies of light-framed construction, two stories or less in height, assigned to Seismic Design Category D, E, or F	2,500 psi (17.24 MPa)
2b. Foundations for other structures assigned to Seismic Design Category D, E, or F	3,000 psi (20.68 MPa)
3. Precast nonprestressed driven piles	3,000 psi (20.68 MPa)
5. Socketed drilled shafts	4,000 psi (27.58 MPa)
6. Micropiles	4,000 psi (27.58 MPa)
7. Precast prestressed driven piles	5,000 psi (34.48 MPa)

1808.8.2 Concrete cover. The concrete cover provided for prestressed and nonprestressed reinforcement in foundations shall be no less than that specified in Table 1808.8.2. 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 the concrete surface.

TABLE 1808.8.2
MINIMUM CONCRETE COVER

FOUNDATION ELEMENT OR CONDITION	MINIMUM COVER
1. Shallow foundations	In accordance with Section 7.7 of ACI 318
2. Precast nonprestressed deep foundation elements [a]	
Exposed to seawater	3 inches (76 mm)
Not manufactured under plant conditions	2 inches (51 mm)
Manufactured under plant control conditions	In accordance with Section 7.7.3 of ACI 318
3. Precast prestressed deep foundation elements	
Exposed to seawater	2.5 inches (64 mm)
Other	In accordance with Section 7.7.3 of ACI 318
4. Cast-in-place deep foundation elements not enclosed by a steel pipe, tube, or permanent casing	2.5 inches (64 mm)
5. Cast-in-place deep foundation elements enclosed by a steel pipe, tube, or permanent casing	1 inch (25 mm)
6. Structural steel core within a steel pipe, tube, or permanent casing	2 inches (51 mm)

a. Longitudinal bars spaced less than 1.5 inches (38 mm) clear distance apart shall be considered bundled bars for which the minimum concrete cover shall be equal to that for the equivalent diameter of the bundled bars.

~~**1805.4.2.4**~~ **1808.8.3 Placement of concrete.** Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized foundation. Concrete ~~footings~~ shall not be placed through water unless a tremie or other method approved by the building official is used. Where placed under or in the presence of water, the concrete shall be deposited by approved means to ensure minimum segregation of the mix and negligible turbulence of the water. Where depositing concrete from the top of a deep foundation element, the concrete shall be chuted directly into smooth-sided pipes or tubes or poured in a rapid and continuous operation through a funnel hopper centered at the top of the element.

~~**1805.4.2.5**~~ **1808.8.4 Protection of concrete.** Concrete ~~footings~~ foundations shall be protected from freezing during depositing and for a period of not less than five days thereafter. Water shall not be allowed to flow through the deposited concrete.

~~**1805.4.2.6**~~ **1808.8.5 Forming of concrete.** Concrete ~~footings~~ foundations are permitted to be cast against the earth where, in the opinion of the building official, soil conditions do not require ~~forming~~ form work. Where ~~forming~~ form work is required, it shall be in accordance with Chapter 6 of ACI 318.

~~**1805.9**~~ **1808.8.6 Seismic requirements.** See Section 1908 for additional requirements for ~~footings and~~ foundations of structures assigned to Seismic Design Category C, D, E or F.

For structures assigned to Seismic Design Category D, E or F, provisions of ACI 318, Sections 21.10.1 ~~to 21.10.3~~ through 21.10.4, shall apply ~~when~~ where not in conflict with the provisions of ~~Section 1805~~ Sections 1808 through 1810. ~~Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.~~

Exceptions:

1. ~~Group R or U occupancies of light-framed construction and two stories or less above grade plane are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2 MPa) at 28 days.~~
1. Detached one- and two-family dwellings of light-frame construction and two stories or less above grade plane are not required to comply with the provisions of ACI 318, Sections 21.10.1 through ~~to 21.10.3~~ 21.10.4.
2. Section 21.10.4.4(a) of ACI 318 shall not apply.

~~1812.7~~ 1808.9 Vertical masonry foundation elements. ~~Where the unsupported height of foundation piers exceeds six times the least dimension, the allowable working stress on piers of unit masonry shall be reduced~~ Vertical masonry foundation elements that are not foundation piers as defined in Section 2102.1 shall be designed as piers, walls, or columns, as applicable, in accordance with ACI 530/ASCE 5/TMS 402.

1809
SHALLOW FOUNDATIONS

~~1805.1~~ 1809.1 [Supp] General. ~~Footings and~~ Shallow foundations shall be designed and constructed in accordance with Sections ~~1805.1 through 1805.9~~ 1809.2 through 1809.13.

1809.2 Supporting soils. ~~Footings and~~ Shallow foundations shall be built on undisturbed soil, compacted fill material or controlled low-strength material (CLSM). Compacted fill material shall be placed in accordance with Section 1803.5. CLSM shall be placed in accordance with Section 1803.6.

1809.3 Stepped footings. The top surface of footings shall be level. The bottom surface of footings ~~is~~ shall be permitted to have a slope not exceeding one unit vertical in 10 units horizontal (10-percent slope). Footings shall be stepped where it is necessary to change the elevation of the top surface of the footing or where the surface of the ground slopes more than one unit vertical in 10 units horizontal (10-percent slope).

~~1805.2~~ 1809.4 Depth and width of footings. The minimum depth of footings below the undisturbed ground surface shall be 12 inches (305 mm). Where applicable, the ~~depth of footings~~ requirements of Section 1809.5 shall also ~~conform to Sections 1805.2.1 through 1805.2.3~~ be satisfied. The minimum width of footings shall be 12 inches (305 mm).

~~1805.2.1~~ 1809.5 Frost protection. Except where otherwise protected from frost, foundation walls~~, piers~~ and other permanent supports of buildings and structures shall be protected from frost by one or more of the following methods:

1. Extending below the frost line of the locality;
2. Constructing in accordance with ASCE-32; or
3. Erecting on solid rock.

Exception: Free-standing buildings meeting all of the following conditions shall not be required to be protected:

1. ~~Classified in~~ Assigned to Occupancy Category I, in accordance with Section 1604.5;
2. Area of 600 square feet (56 m^2) or less for light-frame construction or 400 square feet (37 m^2) or less for other than light-frame construction; and
3. Eave height of 10 feet (3048 mm) or less.

~~Footings~~ Shallow foundations shall not bear on frozen soil unless such frozen condition is of a permanent character.

~~1805.2.2~~ 1809.6 ~~Isolated~~ Location of footings. Footings on granular soil shall be so located that the line drawn between the lower edges of adjoining footings shall not have a slope steeper than 30 degrees (0.52 rad) with the horizontal, unless the material supporting the higher footing is braced or retained or otherwise laterally supported in an approved manner or a greater slope has been properly established by engineering analysis.

1809.7 Prescriptive footings for light-frame construction. Where a specific design is not provided, concrete or masonry-unit footings supporting walls of light-frame construction shall be permitted to be designed in accordance with Table 1809.7.

TABLE ~~1805.4.2~~ 1809.7
PRESCRIPTIVE FOOTINGS SUPPORTING WALLS OF LIGHT-FRAME CONSTRUCTION [a, b, c, d, e]

NUMBER OF FLOORS SUPPORTED BY THE FOOTING [f]	WIDTH OF FOOTING (inches)	THICKNESS OF FOOTING (inches)
1	12	6
2	15	6
3	18	8 [g]

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm.

a. Depth of footings shall be in accordance with Section ~~1805.2~~ 1809.4.
b. The ground under the floor ~~is~~ shall be permitted to be excavated to the elevation of the top of the footing.
c. Interior-stud-bearing walls ~~are~~ shall be permitted to be supported by isolated footings. The footing width and length shall be twice the width shown in this table, and footings shall be spaced not more than 6 feet on center.

d. See Section 1908 for additional requirements for concrete footings of structures assigned to Seismic Design Category C, D, E or F.
e. For thickness of foundation walls, see Section 1805.5.
f. Footings ~~are~~ shall be permitted to support a roof in addition to the stipulated number of floors. Footings supporting roof only shall be as required for supporting one floor.
g. Plain concrete footings for Group R-3 occupancies ~~are~~ shall be permitted to be 6 inches thick.

~~1805.4.2.3~~ 1809.8 Plain concrete footings. The edge thickness of plain concrete footings supporting walls of other than light-frame construction shall not be less than 8 inches (203 mm) where placed on soil.

Exception: For plain concrete footings supporting Group R-3 occupancies, the edge thickness is permitted to be 6 inches (152 mm), provided that the footing does not extend beyond a distance greater than the thickness of the footing on either side of the supported wall.

~~1805.4.3~~ 1809.9 Masonry-unit footings. The design, materials and construction of masonry-unit footings shall comply with Sections ~~1805.4.3.1 and 1805.4.3.2~~ 1809.9.1 and 1809.9.2, and the provisions of Chapter 21.

Exception: Where a specific design is not provided, masonry-unit footings supporting walls of light-frame construction ~~are~~ shall be permitted to be designed in accordance with Table ~~1805.4.2~~ 1809.7.

~~1805.4.3.1~~ 1809.9.1 Dimensions. Masonry-unit footings shall be laid in Type M or S mortar complying with Section 2103.8 and the depth shall not be less than twice the projection beyond the wall, pier or column. The width shall not be less than 8 inches (203 mm) wider than the wall supported thereon.

~~1805.4.3.2~~ 1809.9.2 Offsets. The maximum offset of each course in brick foundation walls stepped up from the footings shall be 1.5 inches (38 mm) where laid in single courses, and 3 inches (76 mm) where laid in double courses.

~~1805.5.7~~ 1809.10 Pier and curtain wall foundations. Except in Seismic Design Categories D, E and F, pier and curtain wall foundations ~~are~~ shall be permitted to be used to support light-frame construction not more than two stories above grade plane, provided the following requirements are met:

1. All load-bearing walls shall be placed on continuous concrete footings bonded integrally with the exterior wall footings.
2. The minimum actual thickness of a load-bearing masonry wall shall not be less than 4 inches (102 mm) nominal or 3.625 inches (92 mm) actual thickness, and shall be bonded integrally with piers spaced 6 feet (1829 mm) on center (o.c.).
3. Piers shall be constructed in accordance with Chapter 21 and the following:
   3.1. The unsupported height of the masonry piers shall not exceed 10 times their least dimension.
   3.2. Where structural clay tile or hollow concrete masonry units are used for piers supporting beams and girders, the cellular spaces shall be filled solidly with concrete or Type M or S mortar.

   Exception: Unfilled hollow piers ~~are~~ shall be permitted where the unsupported height of the pier is not more than four times its least dimension.

   3.3. Hollow piers shall be capped with 4 inches (102 mm) of solid masonry or concrete or the cavities of the top course shall be filled with concrete or grout.
4. The maximum height of a 4-inch (102mm) load-bearing masonry foundation wall supporting wood frame walls and floors shall not be more than 4 feet (1219 mm) in height.
5. The unbalanced fill for 4-inch (102 mm) foundation walls shall not exceed 24 inches (610 mm) for solid masonry, nor 12 inches (305mm) for hollow masonry.

~~1805.4.4~~ 1809.11 Steel grillage footings. Grillage footings of structural steel shapes shall be separated with approved steel spacers and be entirely encased in concrete with at least 6 inches (152 mm) on the bottom and at least 4 inches (102 mm) at all other points. The spaces between the shapes shall be completely filled with concrete or cement grout.

~~1805.4.5~~ 1809.12 Timber footings. Timber footings ~~are~~ shall be permitted for buildings of Type V construction and as otherwise approved by the building official. Such footings shall be treated in accordance with AWPA U1 (Commodity Specification A, Use Category 4B). Treated timbers are not required where placed entirely below permanent water level, or where used as capping for wood piles that project above the water level over submerged or marsh lands. The compressive stresses perpendicular to grain in untreated timber footings supported upon treated piles shall not exceed 70 percent of the allowable stresses for the species and grade of timber as specified in the AF&PA NDS.

(Renumber Section 1805.4.6)

~~1805.4.2.2~~ 1809.13 Footing seismic ties. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613, individual spread footings founded on soil defined in Section 1613.5.2 as Site Class E or F shall be interconnected by ties. Ties shall be capable of carrying, in tension or compression, a force equal to the product of the larger footing load times the seismic coefficient S_{DS} divided by 10 unless it is demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade.

1808.2.23.2 Seismic Design Category D, E or F. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613, the requirements for Seismic Design Category C given in Section 1808.2.23.1 shall be met, in addition to the following. ~~Provisions of ACI 318, Section 21.10.4, shall apply when not in conflict with the provisions of Sections 1808 through 1812. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.~~

~~Exceptions:~~

1. ~~Group R or U occupancies of light-framed construction and two stories or less above grade plane are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2 MPa) at 28 days.~~
2. ~~Detached one- and two-family dwellings of light-frame construction and two stories or less in height are not required to comply with the provisions of ACI 318, Section 21.10.4.~~
3. ~~Section 21.10.4.4(a) of ACI 318 need not apply to concrete piles.~~

1809.2.3.1 Materials. Prestressing steel shall conform to ASTM A 416. ~~Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 5,000 psi (34.48 MPa).~~

1810.3.4 Reinforcement. For piles installed with a hollow-stem auger, where full-length longitudinal steel reinforcement is placed without lateral ties, the reinforcement shall be placed through the hollow stem of the auger prior to filling the pile with concrete. ~~All pile reinforcement shall have a concrete cover of not less than 2.5 inches (64 mm).~~

Exception: Where physical constraints do not allow the placement of the longitudinal reinforcement prior to filling the pile with concrete or where partial-length longitudinal reinforcement is placed without lateral ties, the reinforcement is allowed to be placed after the piles are completely concreted but while concrete is still in a semifluid state.

1810.5.4 Reinforcement. ~~Reinforcement shall not be placed within 1 inch (25 mm) of the steel shell.~~ Reinforcing shall be required for unsupported pile lengths or where the pile is designed to resist uplift or unbalanced lateral loads.

1810.6.4 Reinforcement. Reinforcement steel shall conform to Section 1810.1.2. ~~Reinforcement shall not be placed within 1 inch (25 mm) of the steel casing.~~

1810.7.2 Materials. Pipe and steel cores shall conform to the material requirements in Section 1809.3. Pipes shall have a minimum wall thickness of 3/8 inch (9.5 mm) and shall be fitted with a suitable steel-driving shoe welded to the bottom of the pipe. ~~Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 4,000 psi (27.58 MPa). The concrete mix shall be designed and proportioned so as to produce a cohesive workable mix with a slump of 4 inches to 6 inches (102 mm to 152 mm).~~

1810.7.4 Structural core. The gross cross-sectional area of the structural steel core shall not exceed 25 percent of the gross area of the caisson. ~~The minimum clearance between the structural core and the pipe shall be 2 inches (51 mm).~~ Where cores are to be spliced, the ends shall be milled or ground to provide full contact and shall be full-depth welded.

1810.7.6 Installation. The rock socket and pile shall be thoroughly cleaned of foreign materials before filling with concrete. Steel cores shall be bedded in cement grout at the base of the rock socket. ~~Concrete shall not be placed through water except where a tremie or other approved method is used.~~

1810.8.2 [Supp] Materials. ~~Grout shall have a specified compressive strength (f'_c) of not less than 4,000 psi (27.58 Mpa). The grout mix shall be designed and proportioned so as to produce a pumpable mixture.~~ Reinforcement shall consist of deformed reinforcing bars in accordance with ASTM A 615 Grade 60 or 75 or ASTM A 722 Grade 150.

The steel pipe shall have a minimum wall thickness of 3/16 inch (4.8 mm). Splices shall comply with Section 1808.2.7. The steel pipe shall have a minimum yield strength exceeding 45,000 psi (310 MPa) and a minimum elongation of 15 percent as shown by mill certifications or two coupon test samples per 40,000 pounds (18 160 kg) of pipe.

Delete without substitution:

~~**1809.2.2.1 Materials.** Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 3,000 psi (20.68 MPa).~~

~~**1809.2.2.5 Concrete cover.** Reinforcement for piles that are not manufactured under plant conditions shall have a concrete cover of not less than 2 inches (51 mm).~~

~~Reinforcement for piles manufactured under plant control conditions shall have a concrete cover of not less than 1.25 inches (32 mm) for No. 5 bars and smaller, and not less than 1.5 inches (38 mm) for No. 6 through No. 11 bars except that longitudinal bars spaced less than 1.5 inches (38 mm) clear distance apart shall be considered bundled bars for which the minimum concrete cover shall be equal to that for the equivalent diameter of the bundled bars. Reinforcement for piles exposed to seawater shall have a concrete cover of not less than 3 inches (76 mm).~~

~~**1809.2.3.5 Concrete cover.** Prestressing steel and pile reinforcement shall have a concrete cover of not less than 1-1/4 inches (32 mm) for square piles of 12 inches (305 mm) or smaller size and 1-1/2 inches (38 mm) for larger piles, except that for piles exposed to seawater, the minimum protective concrete cover shall not be less than 2-1/2 inches (64 mm).~~

~~**1810.1.3 Concrete placement.** Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. Concrete shall not be placed through water except where a tremie or other approved method is used. When depositing concrete from the top of the pile, the concrete shall not be chuted directly into the pile but shall be poured in a rapid and continuous operation through a funnel hopper centered at the top of the pile.~~

~~**1810.2.5 Concrete cover.** The minimum concrete cover shall be 2-1/2 inches (64 mm) for uncased shafts and 1 inch (25 mm) for cased shafts.~~

~~**1810.4.4 Concrete cover.** Pile reinforcement shall have a concrete cover of not less than 2.5 inches (64 mm), measured from the inside face of the drive casing or mandrel.~~

~~**1810.6.5 Placing concrete.** The placement of concrete shall conform to Section 1810.1.3, but is permitted to be chuted directly into smooth sided pipes and tubes without a centering funnel hopper.~~

~~**1812.3 Materials.** Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 2,500 psi (17.24 MPa). Where concrete is placed through a funnel hopper at the top of the pier, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 6 inches (152 mm). Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.~~

~~**1812.5 Concrete placement.** Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. Concrete shall not be placed through water except where a tremie or other approved method is used. When depositing concrete from the top of the pier, the concrete shall not be chuted directly into the pier but shall be poured in a rapid and continuous operation through a funnel hopper centered at the top of the pier.~~

~~**1812.10 Dewatering.** Where piers are carried to depths below water level, the piers shall be constructed by a method that will provide accurate preparation and inspection of the bottom, and the depositing or construction of sound concrete or other masonry in the dry.~~

Committee Action: Approved as Modified

Modify proposal as follows:

1808.6.3 Removal of expansive soil. Where expansive soil is removed in lieu of designing foundations in accordance with Section 1808.6.1 or 1808.6.2, the soil shall be removed to a depth sufficient to ensure a constant moisture content in the remaining soil. Fill material shall not contain expansive soils and shall comply with Section ~~1803.5 or 1803.6~~ <u>1804.5 or 1804.6</u>.

Exception: Expansive soil need not be removed to the depth of constant moisture, provided the confining pressure in the expansive soil created by the fill and supported structure exceeds the swell pressure.

TABLE 1808.8.1
MINIMUM SPECIFIED COMPRESSIVE STRENGTH, f'_c, OF CONCRETE OR GROUT

FOUNDATION ELEMENT OR CONDITION	SPECIFIED COMPRESSIVE STRENGTH, $f'c$
1. Foundations for structures assigned to Seismic Design Category A, B, or C	2,500 psi
2a. Foundations for Group R or U occupancies of light-framed construction, two stories or less in height, assigned to Seismic Design Category D, E, or F	2,500 psi
2b. Foundations for other structures assigned to Seismic Design Category D, E, or F	3,000 psi
3. Precast nonprestressed driven piles	~~3,000~~ <u>4,000</u> psi
5. Socketed drilled shafts	4,000 psi
6. Micropiles	4,000 psi
7. Precast prestressed driven piles	5,000 psi

1808.8.2 Concrete cover. The concrete cover provided for prestressed and nonprestressed reinforcement in foundations shall be no less than ~~that~~ <u>the largest applicable value</u> specified in Table 1808.8.2. <u>Longitudinal bars spaced less than 1.5 inches (38 mm) clear distance apart shall be considered bundled bars for which the concrete cover provided shall also be no less than that required by Section 7.7.4 of ACI 318.</u> 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 the concrete surface.

TABLE 1808.8.2
MINIMUM CONCRETE COVER

FOUNDATION ELEMENT OR CONDITION	MINIMUM COVER
1. Shallow foundations	In accordance with Section 7.7 of ACI 318
2. Precast nonprestressed deep foundation elements [a] Exposed to seawater Not manufactured under plant conditions Manufactured under plant control conditions	 3 inches 2 inches In accordance with Section 7.7.3 of ACI 318
3. Precast prestressed deep foundation elements Exposed to seawater Other	 2.5 inches In accordance with Section 7.7.3 of ACI 318

FOUNDATION ELEMENT OR CONDITION	MINIMUM COVER
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

a. ~~Longitudinal bars spaced less than 1.5 inches (38 mm) clear distance apart shall be considered bundled bars for which the minimum concrete cover shall be equal to that for the equivalent diameter of the bundled bars.~~

1808.8.3 Placement of concrete. Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized foundation. Concrete shall not be placed through water unless a tremie or other method approved by the building official is used. Where placed under or in the presence of water, the concrete shall be deposited by approved means to ensure minimum segregation of the mix and negligible turbulence of the water. Where depositing concrete from the top of a deep foundation element, the concrete shall be chuted directly into smooth-sided pipes or tubes or ~~poured~~ placed in a rapid and continuous operation through a funnel hopper centered at the top of the element.

1809.2 Supporting soils. Shallow foundations shall be built on undisturbed soil, compacted fill material or controlled low-strength material (CLSM). Compacted fill material shall be placed in accordance with Section ~~1803.5~~ 1804.5. CLSM shall be placed in accordance with Section ~~1803.6~~ 1804.6.

1809.5 Frost protection. Except where otherwise protected from frost, foundations ~~walls~~ and other permanent supports of buildings and structures shall be protected from frost by one or more of the following methods:

1. Extending below the frost line of the locality;
2. Constructing in accordance with ASCE-32; or
3. Erecting on solid rock.

Exception: Free-standing buildings meeting all of the following conditions shall not be required to be protected:

1. Assigned to Occupancy Category I, in accordance with Section 1604.5;
2. Area of 600 square feet (56 m^2) or less for light-frame construction or 400 square feet (37 m^2) or less for other than light-frame construction; and
3. Eave height of 10 feet (3048 mm) or less.

Shallow foundations shall not bear on frozen soil unless such frozen condition is of a permanent character.

TABLE 1809.7
PRESCRIPTIVE FOOTINGS SUPPORTING WALLS OF LIGHT-FRAME CONSTRUCTION [a, b, c, d, e]

NUMBER OF FLOORS SUPPORTED BY THE FOOTING [f]	WIDTH OF FOOTING (inches)	THICKNESS OF FOOTING (inches)
1	12	6
2	15	6
3	18	8 [g]

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm.
a. Depth of footings shall be in accordance with Section 1809.4.
b. The ground under the floor shall be permitted to be excavated to the elevation of the top of the footing.
c. Interior-stud-bearing walls shall be permitted to be supported by isolated footings. The footing width and length shall be twice the width shown in this table, and footings shall be spaced not more than 6 feet on center.
d. See Section 1908 for additional requirements for concrete footings of structures assigned to Seismic Design Category C, D, E or F.
e. For thickness of foundation walls, see Section ~~1805.5~~ 1807.1.6.
f. Footings shall be permitted to support a roof in addition to the stipulated number of floors. Footings supporting roof only shall be as required for supporting one floor.
g. Plain concrete footings for Group R-3 occupancies shall be permitted to be 6 inches thick.

1809.8 Plain concrete footings. The edge thickness of plain concrete footings supporting walls of other than light-frame construction shall not be less than 8 inches (203 mm) where placed on soil or rock.

Exception: For plain concrete footings supporting Group R-3 occupancies, the edge thickness is permitted to be 6 inches (152 mm), provided that the footing does not extend beyond a distance greater than the thickness of the footing on either side of the supported wall.

(Portions of proposal not shown remain unchanged)

Committee Reason: This reformatting of the foundation provisions makes the section easier to understand and apply. The modification makes editorial clarifications as well as section number correlations based on prior approved code changes. In addition, the minimum strength required for precast nonprecast driven piles (see

Table 1808.8.1) is increased to avoid damage during installation. Also footnote a is removed from Table 1808.8.2 and the spacing requirement is added to Section 1808.8.2. Item 7 is added to Table 1808.8.2 to address piers in a rock socket cored in competent rock.

Assembly Action: None

S151-07/08

Committee Action: Approved as Submitted

Committee Reason: This proposal rewords the notes of Figure 1805.3.1 so that they will be clearer.

Assembly Action: None

S152-07/08

Committee Action: Approved as Modified

Modify proposal as follows:

1805.7.3 Backfill. The backfill in the annular space around columns not embedded in poured footings shall be by one of the following methods:

1. Backfill shall be of concrete with a specified compressive strength of <u>not less than</u> 2,000 psi (13.8 MPa). The hole shall not be less than 4 inches (102 mm) larger than the diameter of the column at its bottom or 4 inches (102 mm) larger than the diagonal dimension of a square or rectangular column.
2. Backfill shall be of clean sand. The sand shall be thoroughly compacted by tamping in layers not more than 8 inches (203 mm) in depth.
3. Backfill shall be of controlled low-strength material (CLSM).

(Portions of proposal not shown remain unchanged)

Committee Reason: This code change makes the IBC references to concrete compressive strength consistent with ACI 318 terminology. The modification makes an editorial change that helps to clarify the intent of Section 1805.7.3.

Assembly Action: None

S153-07/08

Committee Action: Approved as Submitted

Committee Reason: This code change makes editorial improvements and fixes an overly restrictive requirement for foundation ties.

Assembly Action: None

S154-07/08

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standard AF & PA PWF-07 indicated that, in the opinion of ICC Staff, the standard did comply with ICC standards criteria.

PART I – IBC STRUCTURAL

Committee Action: Approved as Submitted

Committee Reason: This code change improves the code by adopting the latest version of the wood foundation standard by reference.

Assembly Action: None

PART II – IRC B/E
Committee Action: Approved as Submitted

Committee Reason: This change updates the referenced technical Report to a consensus standard and is needed in the code.

Assembly Action: None

S155-07/08

PART I – IBC STRUCTURAL
Committee Action: Approved as Submitted

Committee Reason: This proposal creates consistency in terminology referring fo light-frame construction throughout the code. It also harmonizes the IBC with the IRC.

Assembly Action: None

PART II – IRC B/E
Committee Action: Approved as Submitted

Committee Reason: This change updates the terminology of light-gage to cold-formed to agree with current standards. Also, harmonizes the IRC with the IBC with respect to light-frame construction.

Assembly Action: None

S156-07/08

Committee Action: Approved as Modified

Modify proposal as follows:

1806.1 General. Retaining walls shall be designed to ensure stability against overturning, sliding, excessive foundation pressure and water uplift. Retaining walls shall be designed for a safety factor of 1.5 against lateral sliding and overturning ~~excluding load combinations that include seismic forces. The dead load factor used in load combinations under Section 1605.3 shall be taken as 1.0 when used with the safety factors defined in this section.~~ The safety factor against lateral sliding shall be taken as the available soil resistance at the base of the ~~structure's~~ retaining wall foundation divided by the net lateral force applied to the ~~structure~~ retaining wall.

> **Exception:** Where earthquake loads are included, the minimum safety factor for retaining wall sliding and overturning shall be 1.1.

Committee Reason: This code change clarifies the required factor of safety against overturning and sliding under earthquake loading. The modification makes the factor of safety under earthquake loading 1.1 and moves the requirement to an exception.

Assembly Action: None

S157-07/08

Committee Action: Approved as Submitted

Committee Reason: This proposal provides information on keyways used in retaining walls that should help designers in their analysis.

Assembly Action: None

S158-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:

1805.1.1 Story above grade plane. Where a basement is considered a story above grade plane and the finished ground level adjacent to the basement wall is below the basement floor elevation for 25 percent or more of the perimeter, the floor and walls shall be dampproofed in accordance with Section 1805.2 and a foundation drain shall be installed in accordance with Section 1805.4.2. The foundation drain shall be installed around the portion of the perimeter where the basement floor is below ground level. The provisions of Sections ~~1802.2.3~~ <u>1803.5.4</u>, 1805.3 and 1805.4.1 shall not apply in this case.

1805.1.2 Under-floor space. The finished ground level of an under-floor space such as a crawl space shall not be located below the bottom of the footings. Where there is evidence that the ground-water table rises to within 6 inches (152 mm) of the ground level at the outside building perimeter, or that the surface water does not readily drain from the building site, the ground level of the under-floor space shall be as high as the outside finished ground level, unless an approved drainage system is provided. The provisions of Sections ~~1802.2.3,~~ <u>1803.5.4,</u> 1805.2, 1805.3 and 1805.4 shall not apply in this case.

1805.2 Dampproofing. Where hydrostatic pressure will not occur as determined by Section ~~1802.2.3~~ <u>1803.5.4</u>, floors and walls for other than wood foundation systems shall be dampproofed in accordance with this section. Wood foundation systems shall be constructed in accordance with AF&PA Technical Report No. 7.

1805.3 Waterproofing. Where the ground-water investigation required by Section ~~1802.2.3~~ <u>1803.5.4</u> indicates that a hydrostatic pressure condition exists, and the design does not include a ground-water control system as described in Section 1805.1.3, walls and floors shall be waterproofed in accordance with this section.

(Portions of proposal not shown remain unchanged)

Committee Reason: This proposal makes some editorial corrections in the section on dampproofing and waterproofing and is coordinated with the other proposals to reformat Chapter 18. The modification provides further section number correlations based on approval of code change S146-07/08.

Assembly Action: **None**

S159-07/08

PART I – IBC STRUCTURAL
Committee Action: **Approved as Submitted**

Committee Reason: This change aligns an IBC requirement in flood hazard areas with the NFIP.

Assembly Action: **None**

PART II – IRC B/E
Committee Action: **Approved as Submitted**

Committee Reason: This change provides a much needed clarification and removes an overly restrictive requirement.

Assembly Action: **None**

S160-07/08

Errata: Replace the entire proposal with the following:

Revise as follows:

SECTION ~~1808~~ <u>1802</u>
~~PIER AND PILE~~ <u>DEFINITIONS</u>

~~1808.1~~ <u>1802.1</u> Definitions. The following words and terms shall, for the purposes of this ~~section~~ <u>chapter</u>, have the meanings shown herein.

<u>**DEEP FOUNDATION.** A deep foundation is a foundation element that does not satisfy the definition of a shallow foundation.</u>

DRILLED SHAFT. A drilled shaft is a cast-in-place deep foundation element constructed by drilling a hole (with or without permanent casing) into soil or rock and filling it with fluid concrete.

Socketed drilled shaft A socketed drilled shaft is a drilled shaft with a permanent pipe or tube casing that extends down to bedrock and an uncased socket drilled into the bedrock

MICROPILE~~S~~. ~~Micropiles are 12 inch (305 mm) diameter or less,~~ A micropile is a bored, grouted-in-place ~~piles incorporating steel pipe (casing) and/or steel reinforcement~~ deep foundation element that develops its load-carrying capacity by means of a bond zone in soil, bedrock, or a combination of soil and bedrock.

SHALLOW FOUNDATION. A shallow foundation is an individual or strip footing, a mat foundation, a slab on grade foundation, or a similar foundation element.

~~**FLEXURAL LENGTH.** Flexural length is the length of the pile from the first point of zero lateral deflection to the underside of the pile cap or grade beam.~~

~~**PIER FOUNDATIONS.** Pier foundations consist of isolated masonry or cast-in-place concrete structural elements extending into firm materials. Piers are relatively short in comparison to their width, with lengths less than or equal to 12 times the least horizontal dimension of the pier. Piers derive their load-carrying capacity through skin friction, through end bearing, or a combination of both.~~

~~**Belled piers.** Belled piers are cast-in-place concrete piers constructed with a base that is larger than the diameter of the remainder of the pier. The belled base is designed to increase the load-bearing area of the pier in end bearing.~~

~~**PILE FOUNDATIONS.** Pile foundations consist of concrete, wood or steel structural elements either driven into the ground or cast in place. Piles are relatively slender in comparison to their length, with lengths exceeding 12 times the least horizontal dimension. Piles derive their load-carrying capacity through skin friction, through end bearing, or a combination of both.~~

~~**Augered uncased piles.** Augered uncased piles are constructed by depositing concrete into an uncased augered hole, either during or after the withdrawal of the auger.~~

~~**Caisson piles.** Caisson piles are cast-in-place concrete piles extending into bedrock. The upper portion of a caisson pile consists of a cased pile that extends to the bedrock. The lower portion of the caisson pile consists of an uncased socket drilled into the bedrock.~~

~~**Concrete-filled steel pipe and tube piles.** Concrete-filled steel pipe and tube piles are constructed by driving a steel pipe or tube section into the soil and filling the pipe or tube section with concrete. The steel pipe or tube section is left in place during and after the deposition of the concrete.~~

~~**Driven uncased piles.** Driven uncased piles are constructed by driving a steel shell into the soil to shore an unexcavated hole that is later filled with concrete. The steel casing is lifted out of the hole during the deposition of the concrete.~~

~~**Enlarged base piles.** Enlarged base piles are cast-in-place concrete piles constructed with a base that is larger than the diameter of the remainder of the pile. The enlarged base is designed to increase the load-bearing area of the pile in end bearing.~~

~~**Steel-cased piles.** Steel-cased piles are constructed by driving a steel shell into the soil to shore an unexcavated hole. The steel casing is left permanently in place and filled with concrete.~~

~~**Timber piles.** Timber piles are round, tapered timbers with small (tip) end embedded into the soil.~~

SECTION 1810
DEEP FOUNDATIONS

~~**1808.2 Piers and piles—general requirements.**~~

~~**1808.2.1 Design.** Piles are permitted to be designed in accordance with provisions for piers in Section 1808 and Sections 1812.3 through 1812.10 where either of the following conditions exists, subject to the approval of the building official:~~

1. ~~Group R-3 and U occupancies not exceeding two stories of light-frame construction, or~~
2. ~~Where the surrounding foundation materials furnish adequate lateral support for the pile.~~

1810.1 General. Deep foundations shall be analyzed, designed, detailed, and installed in accordance with Sections 1810.1 through 1810.4.

~~**1808.2.2**~~ **1810.1.1** ~~**General**~~ **Geotechnical investigation.** ~~Pier and pile~~ Deep foundations shall be designed and installed on the basis of a foundation investigation as defined in Section 1802, unless sufficient data upon which to base the design and installation is available. The investigation and report provisions of Section 1802 shall be expanded to include, but not be limited to, the following:

1. Recommended pier or pile types and installed capacities.
2. Recommended center-to-center spacing of piers or piles.

3. Driving criteria.
4. Installation procedures.
5. Field inspection and reporting procedures (to include procedures for verification of the installed bearing capacity where required).
6. Pier or pile load test requirements.
7. Durability of pier or pile materials.
8. Designation of bearing stratum or strata.
9. Reductions for group action, where necessary.

~~1808.2.18~~ 1810.1.2 Use of existing ~~piers or piles~~ deep foundation elements. ~~Piers or piles~~ Deep foundation elements left in place where a structure has been demolished shall not be used for the support of new construction unless satisfactory evidence is submitted to the building official, which indicates that the ~~piers or piles~~ elements are sound and meet the requirements of this code. Such ~~piers or piles~~ elements shall be load tested or redriven to verify their capacities. The design load applied to such ~~piers or piles~~ elements shall be the lowest allowable load as determined by tests or redriving data.

1810.1.3 Deep foundation elements classified as columns. Deep foundation elements standing unbraced in air, water, or fluid soils shall be classified as columns and designed as such in accordance with the provisions of this code from their top down to the point where adequate lateral support is provided in accordance with Section 1810.2.1.

Exception: Where the unsupported height to least horizontal dimension of a cast-in-place deep foundation element does not exceed three, it shall be permitted to design and construct such an element as a pedestal in accordance with ACI 318.

~~1808.2.3~~ 1810.1.4 Special types of ~~piles~~ deep foundations. The use of types of ~~piles~~ deep foundation elements not specifically mentioned herein is permitted, subject to the approval of the building official, upon the submission of acceptable test data, calculations and other information relating to the structural properties and load capacity of such ~~piles~~ elements. The allowable stresses for materials shall not in any case exceed the limitations specified herein.

~~1808.2.9 Lateral support.~~

1810.2 Analysis. The analysis of deep foundations for design shall be in accordance with Sections 1810.2.1 through 1810.2.5.

~~1808.2.9.1~~ 1810.2.1 ~~General~~ Lateral support. Any soil other than fluid soil shall be deemed to afford sufficient lateral support ~~to the pier or pile~~ to prevent buckling of deep foundation elements and to permit the design of the ~~pier or pile~~ elements in accordance with accepted engineering practice and the applicable provisions of this code.

~~1808.2.9.2 Unbraced piles.~~ ~~Piles standing~~ Where deep foundation elements stand unbraced in air, water, or ~~in~~ fluid soils, it shall be ~~designed as columns in accordance with the provisions of this code. Such piles driven into firm ground can be considered~~ permitted to consider them fixed and laterally supported at a point 5 feet (1524 mm) ~~below the ground surface and in soft material at~~ into stiff soil or 10 feet (3048 mm) ~~below the ground surface~~ into soft soil unless otherwise prescribed by the building official after a foundation investigation by an approved agency.

~~1808.2.5~~ 1810.2.2 Stability. ~~Piers or piles~~ Deep foundation elements shall be braced to provide lateral stability in all directions. Three or more ~~piles~~ elements connected by a rigid cap shall be considered braced, provided that the ~~piles~~ elements are located in radial directions from the centroid of the group not less than 60 degrees (1 rad) apart. A ~~two- pile~~ two-element group in a rigid cap shall be considered to be braced along the axis connecting the two ~~piles~~ elements. Methods used to brace ~~piers or piles~~ deep foundation elements shall be subject to the approval of the building official.

~~Piles~~ Deep foundation elements supporting walls shall be ~~driven~~ placed alternately in lines spaced at least 1 foot (305 mm) apart and located symmetrically under the center of gravity of the wall load carried, unless effective measures are taken to provide for eccentricity and lateral forces, or the ~~wall piles~~ foundation elements are adequately braced to provide for lateral stability. ~~A single row of piles without lateral bracing is permitted for one- and two-family dwellings and lightweight construction not exceeding two stories above grade plane or 35 feet (10 668 mm) in building height, provided the centers of the piles are located within the width of the foundation wall.~~

Exceptions:

1. Isolated cast-in-place deep foundation elements without lateral bracing shall be permitted where the least horizontal dimension is no less than 2 feet (610 mm), adequate lateral support in accordance with Section 1810.2.1 is provided for the entire height and the height does not exceed 12 times the least horizontal dimension.
2. A single row of deep foundation elements without lateral bracing is permitted for one- and two-family dwellings and lightweight construction not exceeding two stories above grade plane or 35 feet (10 668 mm) in building height, provided the centers of the elements are located within the width of the supported wall.

~~1808.2.12~~ 1810.2.3 Settlement ~~analysis~~. The settlement of ~~piers, individual piles or groups of piles~~ a single deep foundation element or group thereof shall be estimated based on approved methods of analysis. The predicted settlement shall cause neither harmful distortion of, nor instability in, the structure, nor cause any stresses to exceed allowable values.

~~1808.2.23.1.2~~ 1810.2.4 ~~Design details~~ Lateral loads. ~~Pier or pile~~ The moments, shears and lateral deflections used for design of deep foundation elements shall be established considering the nonlinear interaction of the shaft and soil, as ~~recommended~~ determined by a registered design professional. Where the ratio of the depth of embedment of the ~~pile-to-pile diameter or width~~ element to its least horizontal dimension is less than or equal to six, ~~the pile may be assumed to be~~ it shall be permitted to assume the element is rigid.

~~Pile group effects from soil on lateral pile nominal strength shall be included where pile center-to-center spacing in the direction of lateral force is less than eight pile diameters. Pile group effects on vertical nominal strength shall be included where pile center-to-center spacing is less than three pile diameters. The pile uplift soil nominal strength shall be taken as the pile uplift strength as limited by the frictional force developed between the soil and the pile.~~

~~Where a minimum length for reinforcement or the extent of closely spaced confinement reinforcement is specified at the top of the pier or pile, provisions shall be made so that those specified lengths or extents are maintained after pier or pile cutoff.~~

~~1808.2.23.2.1~~ 1810.2.4.1 ~~Design details for piers, piles and grade beams~~ Seismic Design Categories D through F. ~~Piers or piles~~ For structures assigned to Seismic Design Category D, E, or F, deep foundation elements on Site Class E or F sites, as determined in Section 1613.5.2, shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains modified for soil-~~pile~~ foundation-structure interaction coupled with ~~pier or pile~~ foundation element deformations ~~induced by lateral pier or pile resistance to structure seismic forces~~ associated with earthquake loads imparted to the foundation by the structure.

Exception: ~~Piers or piles~~ Deep foundation elements that satisfy the following additional detailing requirements shall be deemed to comply with the curvature capacity requirements of this section.

1. Precast prestressed concrete piles detailed in accordance with Section ~~1809.2.3.2.2~~ 1810.3.8.3.3.
2. Cast-in-place ~~concrete piles~~ deep foundation elements with a minimum longitudinal reinforcement ratio of 0.005 extending the full length of the ~~pile~~ element and detailed in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 as required by ~~this~~ Section 1810.3.9.4.2.2.

~~Where constructed of nonprestressed concrete such piers or piles shall be designed and detailed in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within seven pile diameters of the pile cap and within seven pile diameters of the interfaces of strata that are hard or stiff and strata that are liquefiable or are composed of soft to medium stiff clay.~~

~~Grade beams shall comply with the provisions in Section 21.10.3 of ACI 318 for grade beams, except where they have the capacity to resist the forces from the load combinations in Section 1605.4.~~

1810.2.5 Group effects. The analysis shall include group effects on lateral behavior where the center-to-center spacing of deep foundation elements in the direction of lateral force is less than eight times the least horizontal dimension of an element. The analysis shall include group effects on axial behavior where the center-to-center spacing of deep foundation elements is less than three times the least horizontal dimension of an element.

1810.3 Design and Detailing. Deep foundations shall be designed and detailed in accordance with Sections 1810.3.1 through 1810.3.12.

1810.3.1 Design conditions. Design of deep foundations shall include the design conditions specified in Sections 1810.3.1.1 through 1810.3.1.5, as applicable.

1810.3.1.1 Design methods for concrete elements. Where concrete deep foundations are laterally supported in accordance with Section 1810.2.1 for the entire height and applied forces cause bending moments no greater than those resulting from accidental eccentricities, structural design of the element using the load combinations of Section 1605.3 and the allowable stresses specified in this chapter shall be permitted. Otherwise, the structural design of concrete deep foundation elements shall use the load combinations of Section 1605.2 and approved strength design methods.

1810.3.1.2 Composite elements. Where a single deep foundation element comprises two or more sections of different materials or different types spliced together, each section of the composite assembly shall satisfy the applicable requirements of this code, and the maximum allowable load shall be limited by the capacity of the weakest section.

~~1808.2.8.8~~ 1810.3.1.3 ~~Overloads on piers or piles~~ Mislocation. ~~The maximum compressive load on any pier or pile due to mislocation shall not exceed~~ To resist the effects of mislocation, compressive overload of deep foundation elements to 110 percent of the allowable design load shall be permitted.

~~1809.2.1.1~~ 1810.3.1.4 ~~Design and manufacture~~ Driven piles. Driven piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by handling, driving and service loads.

~~1810.5.1~~ 1810.3.1.5 ~~Materials~~ Casings. ~~Pile shells or~~ Temporary and permanent casings shall be of steel and shall be sufficiently strong to resist collapse and sufficiently water tight to exclude any foreign materials during the placing of concrete. ~~Steel shells shall have a sealed tip with a diameter of not less than 8 inches (203 mm).~~ Where a permanent casing is considered reinforcing steel, the steel shall be protected under the conditions specified in Section 1810.3.2.5. Horizontal joints in the casing shall be spliced in accordance with Section 1810.3.6.

1810.3.2 Materials. The materials used in deep foundations elements shall satisfy the requirements of Sections 1810.3.2.1 through 1810.3.2.8, as applicable.

~~**1810.2.1**~~ **1810.3.2.1** ~~**Materials**~~ **Concrete.** Where concrete is cast in a steel pipe or where an enlarged base is formed by compacting concrete, the maximum size for coarse aggregate ~~for concrete~~ shall be 3/4 inch (19.1 mm). Concrete to be compacted shall have a zero slump.

1810.3.2.1.1 Seismic hooks. For structures assigned to Seismic Design Category C, D, E, or F in accordance with Section 1613, the ends of hoops, spirals and ties used in concrete deep foundation elements shall be terminated with seismic hooks, as defined in Section 21.1 of ACI 318, and shall be turned into the confined concrete core.

1810.3.2.2 Prestressing steel. Prestressing steel shall conform to ASTM A 416.

~~**1809.3.1**~~ **1810.3.2.3** ~~**Materials**~~ **Structural steel.** Structural steel piles, steel pipe and fully welded steel piles fabricated from plates shall conform to ASTM A 36, ASTM A 252, ASTM A 283, ASTM A 572, ASTM A 588, ASTM A 690, ASTM A 913 or ASTM A 992.

~~**1809.1**~~ **1810.3.2.4 Timber** ~~**piles**~~**.** Timber ~~piles~~ deep foundation elements shall be designed as piles or poles in accordance with the AF&PA NDS.

~~**1809.1.1 Materials.**~~ Round timber ~~piles~~ elements shall conform to ASTM D 25. Sawn timber ~~piles~~ elements shall conform to DOC PS-20.

~~**1809.1.2**~~ **1810.3.2.4.1 Preservative treatment.** Timber ~~piles~~ deep foundation elements used to support permanent structures shall be treated in accordance with this section unless it is established that the tops of the untreated timber ~~piles~~ elements will be below the lowest ground-water level assumed to exist during the life of the structure. Preservative and minimum final retention shall be in accordance with AWPA U1 (Commodity Specification E, Use Category 4C) for round timber ~~piles~~ elements and AWPA U1 (Commodity Specification A, Use Category 4B) for sawn timber ~~piles~~ elements. Preservative-treated timber ~~piles~~ elements shall be subject to a quality control program administered by an approved agency. ~~Pile~~ Element cutoffs shall be treated in accordance with AWPA M4.

~~**1808.2.17**~~ **1810.3.2.5 Protection of** ~~**pile**~~ **materials.** Where boring records or site conditions indicate possible deleterious action on ~~pier or pile~~ the materials used in deep foundation elements because of soil constituents, changing water levels or other factors, the ~~pier or pile materials~~ elements shall be adequately protected by materials, methods or processes approved by the building official. Protective materials shall be applied to the ~~piles~~ elements so as not to be rendered ineffective by ~~driving~~ installation. The effectiveness of such protective measures for the particular purpose shall have been thoroughly established by satisfactory service records or other evidence.

1810.3.2.6 Allowable stresses. The allowable stresses for materials used in deep foundation elements shall not exceed those specified in Table 1810.3.2.6.

TABLE 1810.3.2.6
ALLOWABLE STRESSES FOR MATERIALS USED IN DEEP FOUNDATION ELEMENTS

MATERIAL TYPE AND CONDITION	MAXIMUM ALLOWABLE STRESS [a]
1. Concrete or grout in compression [b] Cast-in-place with a permanent casing in accordance with Section 1810.3.2.7 Cast-in-place in a pipe, tube, or other permanent casing Cast-in-place without a permanent casing Precast nonprestressed Precast prestressed	 $0.4\ f'_c$ $0.33\ f'_c$ $0.3\ f'_c$ $0.33\ f'_c$ $0.33\ f'_c - 0.27\ f_{pc}$
2. Nonprestressed reinforcement in compression	$0.4\ f_y \leq 30{,}000$ psi
3. Structural steel in compression Cores within concrete-filled pipes or tubes Pipes, tubes, or H-piles, where justified in accordance with Section 1810.3.2.8 Pipes or tubes for micropiles Other pipes, tubes, or H-piles	 $0.5\ F_y \leq 32{,}000$ psi $0.5\ F_y \leq 32{,}000$ psi $0.4\ F_y \leq 32{,}000$ psi $0.35\ F_y \leq 16{,}000$ psi
5. Nonprestressed reinforcement in tension Within micropiles Other conditions	 $0.6\ f_y$ $0.5\ f_y \leq 24{,}000$ psi
6. Structural steel in tension Pipes, tubes, or H-piles, where justified in accordance with Section 1810.3.2.8 Other pipes, tubes, or H-piles	 $0.5\ F_y \leq 32{,}000$ psi $0.35\ F_y \leq 16{,}000$ psi
7. Timber	In accordance with the AF&PA NDS

a. f'_c is the specified compressive strength of the concrete or grout; f_{pc} is the compressive stress on the gross concrete section due to effective prestress forces only; f_y is the specified yield strength of reinforcement; F_y is the specified minimum yield stress of structural steel.
b. The stresses specified apply to the gross cross-sectional area within the concrete surface. Where a temporary or permanent casing is used, the inside face of the casing shall be considered the concrete surface.

~~1810.5.2~~ 1810.3.2.7 ~~Allowable stresses~~ Increased allowable compressive stress for cased cast-in-place elements. The allowable ~~design~~ compressive stress in the concrete ~~shall not exceed 33 percent of the 28-day specified compressive strength (f'_c). The allowable concrete compressive stress shall be 0.40 (f'_c) for that portion of the pile meeting the conditions specified in Sections 1810.5.2.1 through 1810.5.2.4.~~ shall be permitted to be increased as specified in Table 1810.3.2.6 for those portions of permanently cased cast-in-place elements that satisfy the following conditions:

1. The design shall not use the casing to resist any portion of the axial load imposed.
2. The casing shall have a sealed tip and be mandrel driven.
3. **~~1810.5.2.1 Shell thickness.~~** The thickness of the ~~steel shell~~ casing shall not be less than manufacturer's standard gage No. 14 ~~gage~~ (0.068 inch) (1.75 mm) ~~minimum~~.
4. **~~1810.5.2.2 Shell type.~~** The ~~shell~~ casing shall be seamless or provided with seams of strength equal to the basic material and be of a configuration that will provide confinement to the cast-in-place concrete.
5. **~~1810.5.2.3 Strength.~~** The ratio of steel yield strength (F_y) to ~~28-day~~ specified compressive strength (f'_c) shall not be less than six.
6. **~~1810.5.2.4 Diameter.~~** The nominal ~~pile~~ diameter of the element shall not be greater than 16 inches (406 mm).

~~1808.2.10~~ 1810.3.2.8 ~~Use~~ Justification of higher allowable ~~pier or pile~~ stresses. Use of allowable stresses greater than those specified ~~for piers or for each pile type in Sections 1809 and 1810 are~~ in Section 1810.3.2.6 shall be permitted where supporting data justifying such higher stresses is filed with the building official. Such substantiating data shall include:

1. A soils investigation in accordance with Section 1802~~.~~; and
2. ~~Pier or pile~~ Load tests in accordance with Section ~~1808.2.8.3~~ 1810.3.3.1.2, regardless of the load supported by the ~~pier or pile~~ element.

The design and installation of the ~~pier or pile~~ deep foundation elements shall be under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and ~~pier or pile~~ deep foundations who shall certify to the building official that the ~~piers or piles~~ elements as installed satisfy the design criteria.

~~1808.2.8.1~~ 1810.3.3 Determination of allowable loads. The allowable axial and lateral loads on ~~piers or piles~~ deep foundation elements shall be determined by an approved formula, load tests or method of analysis.

1810.3.3.1 Allowable axial load. The allowable axial load on a deep foundation element shall be determined in accordance with Section 1810.3.3.1.

~~1808.2.8.2~~ 1810.3.3.1.1 Driving criteria. The allowable compressive load on any ~~pile~~ driven deep foundation element where determined by the application of an approved driving formula shall not exceed 40 tons (356 kN). For allowable loads above 40 tons (356 kN), the wave equation method of analysis shall be used to estimate ~~pile~~ driveability ~~of~~ for both driving stresses and net displacement per blow at the ultimate load. Allowable loads shall be verified by load tests in accordance with Section ~~1808.2.8.3~~ 1810.3.3.1.2. The formula or wave equation load shall be determined for gravity-drop or power-actuated hammers and the hammer energy used shall be the maximum consistent with the size, strength and weight of the driven ~~piles~~ elements. The use of a follower is permitted only with the approval of the building official. The introduction of fresh hammer cushion or pile cushion material just prior to final penetration is not permitted.

~~1808.2.8.3~~ 1810.3.3.1.2 Load tests. Where design compressive loads ~~per pier or pile~~ are greater than those ~~permitted by Section 1808.2.10 or~~ determined using the allowable stresses specified in Section 1810.3.2.6 where the design load for any ~~pier or pile~~ deep foundation element is in doubt, or where cast-in-place deep foundation elements have an enlarged base formed either by compacting concrete or by driving a precast base, control test ~~piers or piles~~ elements shall be tested in accordance with ASTM D 1143 or ASTM D 4945. At least one ~~pier or pile~~ element shall be ~~test loaded~~ load tested in each area of uniform subsoil conditions. Where required by the building official, additional ~~piers or piles~~ elements shall be load tested where necessary to establish the safe design capacity. The resulting allowable loads shall not be more than one-half of the ultimate axial load capacity of the test ~~pier or pile~~ element as assessed by one of the published methods listed in Section ~~1808.2.8.3.1~~ 1810.3.3.1.3 with consideration for the test type, duration and subsoil. The ultimate axial load capacity shall be determined by a registered design professional with consideration given to tolerable total and differential settlements at design load in accordance with Section ~~1808.2.12~~ 1810.2.3. In subsequent installation of the balance of deep foundation ~~piles~~ elements, all ~~piles~~ elements shall be deemed to have a supporting capacity equal to that of the control ~~pile~~ element where such ~~piles~~ elements are of the same type, size and relative length as the test ~~pile~~ element; are installed using the same or comparable methods and equipment as the test ~~pile~~ element; are installed in similar subsoil conditions as the test ~~pile~~ element; and, for driven ~~piles~~ elements, where the rate of penetration (e.g., net displacement per blow) of such ~~piles~~ elements is equal to or less than that of the test ~~pile~~ element driven with the same hammer through a comparable driving distance.

~~1808.2.8.3.1~~ 1810.3.3.1.3 Load test evaluation methods. It shall be permitted to evaluate ~~pile~~ load tests ~~with~~ of deep foundation elements using any of the following methods:

1. Davisson Offset Limit.
2. Brinch-Hansen 90% Criterion.
3. Butler-Hoy Criterion.
4. Other methods approved by the building official.

~~1808.2.8.4~~ 1810.3.3.1.4 Allowable frictional resistance. The assumed frictional resistance developed by any ~~pier or~~ uncased cast-in-place ~~pile~~ deep foundation element shall not exceed one-sixth of the bearing value of the soil material at minimum depth as set forth in Table 1804.2, up to a maximum of 500 psf (24 kPa), unless a greater value is allowed by the building official ~~after~~ on the basis of a soil investigation as specified in Section 1802 is submitted or a greater value is substantiated by a load test in accordance with Section ~~1808.2.8.3~~ 1810.3.3.1.2. Frictional resistance and bearing resistance shall not be assumed to act simultaneously unless recommended by a soil investigation as specified in Section 1802.

~~1808.2.8.5~~ 1810.3.3.1.5 Uplift capacity of a single deep foundation element. Where required by the design, the uplift capacity of a single ~~pier or pile~~ deep foundation element shall be determined by an approved method of analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D 3689. The maximum allowable uplift load shall not exceed the ultimate load capacity as determined in Section ~~1808.2.8.3~~ 1810.3.3.1.2 divided by a factor of safety of two.

1810.3.3.1.6 Uplift capacity of grouped deep foundation elements. For ~~pile groups~~ grouped deep foundation elements subjected to uplift, the allowable working uplift load for the group shall be the lesser of:

1. The proposed individual ~~pile~~ uplift working load times the number of ~~piles~~ elements in the group.
2. Two-thirds of the effective weight of the ~~pile~~ group and the soil contained within a block defined by the perimeter of the group and the length of the ~~pile~~ element.

~~1808.2.8.6~~ 1810.3.3.1.7 Load-bearing capacity. ~~Piers, individual piles and groups of piles~~ Deep foundation elements shall develop ultimate load capacities of at least twice the design working loads in the designated load-bearing layers. Analysis shall show that no soil layer underlying the designated load-bearing layers causes the load-bearing capacity safety factor to be less than two.

~~1808.2.8.7~~ 1810.3.3.1.8 Bent ~~piers or piles~~ deep foundation elements. The load-bearing capacity of ~~piers or piles~~ deep foundation elements discovered to have a sharp or sweeping bend shall be determined by an approved method of analysis or by load testing a representative ~~pier or pile~~ element.

~~1808.2.9.3~~ 1810.3.3.2 Allowable lateral load. Where required by the design, the lateral load capacity of a ~~pier, a~~ single ~~pile~~ deep foundation element or a ~~pile~~ group thereof shall be determined by an approved method of analysis or by lateral load tests to at least twice the proposed design working load. The resulting allowable load shall not be more than one-half of that test load that produces a gross lateral movement of 1 inch (25 mm) at the ground surface.

~~1808.2.11~~ 1810.3.4 ~~Piles in~~ Subsiding ~~areas~~ soils. Where ~~piles~~ deep foundation elements are installed through subsiding fills or other subsiding strata and derive support from underlying firmer materials, consideration shall be given to the downward frictional forces that may be imposed on the ~~piles~~ elements by the subsiding upper strata.

Where the influence of subsiding fills is considered as imposing loads on the ~~pile~~ element, the allowable stresses specified in this chapter ~~are~~ shall be permitted to be increased where satisfactory substantiating data are submitted.

1810.3.5 Dimensions of deep foundation elements. The dimensions of deep foundation elements shall be in accordance with Sections 1810.3.5.1 through 1810.3.5.3, as applicable.

~~1809.2.1.2~~ 1810.3.5.1 ~~Minimum dimension~~ Precast. The minimum lateral dimension of precast concrete deep foundation elements shall be 8 inches (203 mm). Corners of square ~~piles~~ elements shall be chamfered.

1810.3.5.2 Cast-in-place or grouted-in-place. Cast-in-place and grouted-in-place deep foundation elements shall satisfy the requirements of this section.

1810.3.5.2.1 Cased. Cast-in-place deep foundation elements with a permanent casing shall have a nominal outside diameter of not less than 8 inches (203 mm).

~~1810.3.2~~ 1810.3.5.2.2 ~~Dimensions~~ Uncased. Cast-in-place deep foundation elements without a permanent casing shall have a diameter of not less than 12 inches (305 mm). The ~~pile~~ element length shall not exceed 30 times the average diameter. ~~The minimum diameter shall be 12 inches (305 mm).~~

> **Exception:** The length of the ~~pile~~ element is permitted to exceed 30 times the diameter, provided ~~that~~ the design and installation of the ~~pile foundation~~ deep foundations are under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and ~~pile~~ deep foundations. The registered design professional shall certify to the building official that the ~~piles~~ elements were installed in compliance with the approved construction documents.

1810.3.5.2.3 Micropiles. Micropiles shall have an outside diameter of 12 inches (305 mm) or less. There is no minimum diameter for micropiles.

1810.3.5.3 Steel. Steel deep foundation elements shall satisfy the requirements of this section.

~~1809.3.3~~ 1810.3.5.3.1 ~~Dimensions of~~ H-piles. Sections of H-piles shall comply with the following:

1. The flange projections shall not exceed 14 times the minimum thickness of metal in either the flange or the web and the flange widths shall not be less than 80 percent of the depth of the section.
2. The nominal depth in the direction of the web shall not be less than 8 inches (203 mm).
3. Flanges and web shall have a minimum nominal thickness of 3/8 inch (9.5 mm).

~~1809.3.4~~ 1810.3.5.3.2 ~~Dimensions of~~ Steel pipes ~~piles~~ and tubes. Steel ~~pipe piles driven open ended~~ pipes and tubes used as deep foundation elements shall have a nominal outside diameter of not less than 8 inches (203 mm). ~~The pipe~~ Where steel pipes or tubes are driven open-ended, they shall have a minimum of 0.34 square inch (219 mm^2) of steel in cross section to resist each 1,000 foot-pounds (1356 Nm) of pile hammer energy, or shall have the equivalent strength for steels having a yield strength greater than 35,000 psi (241 MPa) or the wave equation analysis shall be permitted to be used to assess compression stresses induced by driving to evaluate if the pile section is appropriate for the selected hammer. Where a pipe or tube with wall thickness less than 0.179 inch (4.6 mm) is driven open ended, a suitable cutting shoe shall be provided. Concrete filled steel pipes or tubes in structures assigned to Seismic Design Category C, D, E, or F shall have a wall thickness of not less than 3/16 inch (5 mm). The pipe or tube casing for socketed drilled shafts shall have a nominal outside diameter of not less than 18 inches (457 mm), a wall thickness of not less than 3/8 inch (9.5 mm), and a suitable steel driving shoe welded to the bottom; the diameter of the rock socket shall be approximately equal to the inside diameter of the casing.

Exceptions:

1. There is no minimum diameter for steel pipes or tubes used in micropiles.
2. ~~**1810.6.3 Minimum dimensions.** Piles shall have a nominal outside diameter of not less than 8 inches (203 mm) and a minimum wall thickness in accordance with Section 1809.3.4.~~ For mandrel-driven pipes or tubes ~~piles~~, the minimum wall thickness shall be 1/10 inch (2.5 mm).

~~1808.2.7~~ 1810.3.6 Splices. Splices shall be constructed so as to provide and maintain true alignment and position of the component parts of the ~~pier or pile~~ deep foundation element during installation and subsequent thereto and shall be of adequate strength to transmit the vertical and lateral loads and moments occurring at the location of the splice during driving and under service loading. Where deep foundation elements of the same type are being spliced, splices shall develop not less than 50 percent of the ~~least capacity of the pier or pile in~~ 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.

~~In addition,~~ Splices occurring in the upper 10 feet (3048 mm) of the embedded portion of ~~the pier or pile~~ an element shall be capable of resisting at allowable working stresses the moment and shear that would result from an assumed eccentricity of the ~~pier or pile~~ axial load of 3 inches (76 mm), or the ~~pier or pile~~ element shall be braced in accordance with Section ~~1808.2.5~~ 1810.2.2 to other ~~piers or piles~~ deep foundation elements that do not have splices in the upper 10 feet (3048 mm) of embedment.

1810.3.6.1 Seismic Design Categories C through F. For structures assigned to Seismic Design Category C, D, E, or F, splices of deep foundation elements shall develop the lesser of the following:

1. The full strength of the deep foundation element; and
2. The axial and shear forces and moments from the load combinations with overstrength factor in Section 12.4.3.2 of ASCE 7.

1810.3.7 Top of pile detailing at cutoffs. Where a minimum length for reinforcement or the extent of closely spaced confinement reinforcement is specified at the top of a deep foundation element, provisions shall be made so that those specified lengths or extents are maintained after cutoff.

~~1809.2 Precast concrete piles.~~

~~1809.2.1~~ 1810.3.8 ~~General~~ Precast concrete piles. ~~The materials, reinforcement and installation of~~ Precast concrete piles shall ~~conform to~~ be designed and detailed in accordance with Sections ~~1809.2.1.1 through 1809.2.1.4~~ 1810.3.8.1 through 1810.3.8.3.

~~1809.2.1.3~~ 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:

1. At not more than 1 inch (25 mm) for the first five ties or spirals at each end; then
2. At not more than 4 inches (102 mm) ~~apart, center to center,~~ for ~~a distance of~~ the remainder of the first 2 feet (610 mm) from ~~the ends of the pile~~ each end; and then
3. At not more than 6 inches (152 mm) elsewhere ~~except that at the ends of each pile, the first five ties or spirals shall be spaced 1 inch (25 mm) center to center~~.

The size ~~gage~~ of ties and spirals shall be as follows:

1. For piles having a ~~diameter~~ least horizontal dimension of 16 inches (406 mm) or less, wire shall not be smaller than 0.22 inch (5.6 mm) (No. 5 gage).
2. For piles having a ~~diameter~~ least horizontal dimension of more than 16 inches (406 mm) and less than 20 inches (508 mm), wire shall not be smaller than 0.238 inch (6 mm) (No. 4 gage).
3. For piles having a ~~diameter~~ least horizontal dimension of 20 inches (508 mm) and larger, wire shall not be smaller than 0.25 inch (6.4 mm) round or 0.259 inch (6.6 mm) (No. 3 gage).

~~1809.2.2~~ 1810.3.8.2 Precast nonprestressed piles. Precast nonprestressed concrete piles shall ~~conform to~~ comply with the requirements of Sections ~~1809.2.2.1 through 1809.2.2.5~~ 1810.3.8.2.1 through 1810.3.8.2.3.

~~1809.2.2.2~~ 1810.3.8.2.1 Minimum reinforcement. ~~The minimum amount of~~ Longitudinal reinforcement shall ~~be 0.8 percent of the concrete section and shall consist of at least four bars~~ consist of at least four bars with a minimum longitudinal reinforcement ratio of 0.008.

~~1809.2.2.2.1~~ 1810.3.8.2.2 Seismic reinforcement in Seismic Design ~~Category~~ Categories C through F. ~~Where a~~ For structures ~~is~~ assigned to Seismic Design Category C, D, E, or F in accordance with Section 1613, ~~the following shall apply~~ precast nonprestressed piles shall be reinforced as specified in this section. The minimum longitudinal reinforcement ~~with a minimum steel~~ ratio ~~of~~ shall be 0.01 ~~shall be provided~~ throughout the length ~~of precast concrete piles~~. ~~Within three pile diameters of the bottom of the pile cap, the longitudinal reinforcement shall be confined with~~ Transverse reinforcement shall consist of closed ties or spirals ~~of~~ with a minimum 3/8 inch (9.5 mm) diameter. ~~Ties or spirals shall be provided at a maximum spacing of~~ Spacing of transverse reinforcement shall not exceed the smaller of eight times the diameter of the smallest longitudinal bar ~~not to exceed~~ or, 6 inches (152 mm) within a distance of three times the least pile dimension from the bottom of the pile cap. Spacing of transverse reinforcement shall not exceed 6 inches (152 mm) throughout the remainder of the pile~~, the closed ties or spirals shall have a maximum spacing of 16 times the smallest longitudinal bar diameter, not to exceed 8 inches (203 mm)~~.

~~1809.2.2.2.2~~ 1810.3.8.2.3 Additional seismic reinforcement in Seismic Design ~~Category~~ Categories D through~~, E or~~ F. ~~Where a~~ For structures ~~is~~ assigned to Seismic Design Category D, E or F in accordance with Section 1613, ~~the requirements for Seismic Design Category C in Section 1809.2.2.2.1 shall apply except as modified by this section. Transverse confinement reinforcement consisting of closed ties or equivalent spirals shall be provided in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within three pile diameters of the bottom of the pile cap. For other than Site Class E or F, or liquefiable sites and where spirals are used as the transverse reinforcement, it shall be permitted to use a volumetric ratio of spiral reinforcement of not less than one half that required by Section 21.4.4.1(a) of ACI 318~~ transverse reinforcement shall be in accordance with Section 1810.3.9.4.2.

~~1809.2.3~~ 1810.3.8.3 Precast prestressed piles. Precast prestressed concrete piles shall ~~conform to~~ comply with the requirements of Sections ~~1809.2.3.1 through 1809.2.3.5~~ 1810.3.8.3.1 through 1810.3.8.3.3.

~~1809.2.3.2~~ 1810.3.8.3.1 ~~Design~~ Effective prestress. ~~Precast prestressed piles shall be designed to resist stresses induced by handling and driving as well as by loads.~~ The effective prestress in the pile shall not be less than 400 psi (2.76MPa) for piles up to 30 feet (9144 mm) in length, 550 psi (3.79 MPa) for piles up to 50 feet (15 240 mm) in length and 700 psi (4.83 MPa) for piles greater than 50 feet (15 240 mm) in length.
Effective prestress shall be based on an assumed loss of 30,000 psi (207 MPa) in the prestressing steel. The tensile stress in the prestressing steel shall not exceed the values specified in ACI 318.

~~1809.2.3.2.1~~ 1810.3.8.3.2 ~~Design~~ Seismic reinforcement in Seismic Design Category C. ~~Where a~~ For structures ~~is~~ assigned to Seismic Design Category C in accordance with Section 1613, ~~the following shall apply~~ precast prestressed piles shall have transverse reinforcement in accordance with this section. The ~~minimum~~ volumetric ratio of spiral reinforcement shall not be less than ~~0.007 or~~ the amount required by the following formula for the upper 20 feet (6096 mm) of the pile.

$\rho_s = 0.12 f'_c / f_{yh}$ **(Equation 18-4)**

where:

f'_c = Specified compressive strength of concrete, psi (MPa)

f_{yh} = Yield strength of spiral reinforcement ≤ 85,000 psi (586 MPa).

ρ_s = Spiral reinforcement index (vol. spiral/vol. core).

At least one-half the volumetric ratio required by Equation 18-4 shall be provided below the upper 20 feet (6096 mm) of the pile.

~~The pile cap connection by means of dowels as indicated in Section 1808.2.23.1 is permitted. Pile cap connection by means of developing pile reinforcing strand is permitted provided that the pile reinforcing strand results in a ductile connection.~~

~~1809.2.3.2.2~~ 1810.3.8.3.3 ~~Design~~ Seismic reinforcement in Seismic Design ~~Category~~ Categories D through~~, E or~~ F. ~~Where a~~ For structures ~~is~~ assigned to Seismic Design Category D, E or F in accordance with Section 1613, ~~the requirements for Seismic Design Category C in Section 1809.2.3.2.1 shall be met, in addition to~~ precast prestressed piles shall have transverse reinforcement in accordance with the following:

1. Requirements in ACI 318, Chapter 21, need not apply, unless specifically referenced.
2. Where the total pile length in the soil is 35 feet (10 668 mm) or less, the lateral transverse reinforcement in the ductile region shall occur through the length of the pile. Where the pile length exceeds 35 feet (10 668 mm), the ductile pile region shall be taken as the greater of 35 feet (10 668 mm) or the distance from the underside of the pile cap to the point of zero curvature plus three times the least pile dimension.
3. In the ductile region, the center-to-center spacing of the spirals or hoop reinforcement shall not exceed one-fifth of the least pile dimension, six times the diameter of the longitudinal strand, or 8 inches (203 mm), whichever is ~~smaller~~ smallest.
4. Circular spiral reinforcement shall be spliced by lapping one full turn and bending the end of ~~the~~ each spiral to a 90-degree hook or by use of a mechanical or welded splice complying with Sec. 12.14.3 of ACI 318.
5. Where the transverse reinforcement consists of circular spirals, the volumetric ratio of spiral transverse reinforcement in the ductile region shall comply with the following:

   $\rho_s = 0.25(f'_c / f_{yh})(A_g / A_{ch} - 1.0)[0.5 + 1.4P/(f'_c A_g)]$ **(Equation 18-5)**

   but not less than:

   $\rho_s = 0.12(f'_c / f_{yh})\ [0.5 + 1.4P/(f'_c A_g)] \geq 0.12 f'_c / f_{yh}$ **(Equation 18-6)** and need not exceed:

$\rho_s = 0.021$ **(Equation 18-7)**

where:

A_g = Pile cross-sectional area, square inches (mm^2).
A_{ch} = Core area defined by spiral outside diameter, square inches (mm^2).
f'_c = Specified compressive strength of concrete, psi (MPa)
f_{yh} = Yield strength of spiral reinforcement ≤ 85,000 psi (586 MPa).
P = Axial load on pile, pounds (kN), as determined from Equations 16-5 and ~~16-6~~ 16-7.
ρ_s = Volumetric ratio (vol. spiral/ vol. core).

This required amount of spiral reinforcement is permitted to be obtained by providing an inner and outer spiral.

6. ~~When~~ Where transverse reinforcement consists of rectangular hoops and cross ties, the total cross-sectional area of lateral transverse reinforcement in the ductile region with spacing, s, and perpendicular ~~to~~ dimension, h_c, shall conform to:

$A_{sh} = 0.3s\, h_c\, (f'_c / f_{yh})(A_g / A_{ch} - 1.0)[0.5 + 1.4P/(f'_c\, A_g)]$ **(Equation 18-8)**

but not less than:

$A_{sh} = 0.12s\, h_c\, (f'_c / f_{yh})\, [0.5 + 1.4P/(f'_c\, A_g)]$ **(Equation 18-9)**

where:
f_{yh} = ≤ 70,000 psi (483 MPa).
h_c = Cross-sectional dimension of pile core measured center to center of hoop reinforcement, inch (mm).
s = Spacing of transverse reinforcement measured along length of pile, inch (mm).
A_{sh} = Cross-sectional area of tranverse reinforcement, square inches (mm^2)
f'_c = Specified compressive strength of concrete, psi (MPa)

The hoops and cross ties shall be equivalent to deformed bars not less than No. 3 in size. Rectangular hoop ends shall terminate at a corner with seismic hooks.

Outside of the length of the pile requiring transverse confinement reinforcing, the spiral or hoop reinforcing with a volumetric ratio not less than one-half of that required for transverse confinement reinforcing shall be provided.

1810.3.9 Cast-in-place deep foundations. Cast-in-place deep foundation elements shall be designed and detailed in accordance with Sections 1810.3.9.1 through 1810.3.9.6.

1810.3.9.1 Design cracking moment. The design cracking moment (ϕM_n) for a cast-in-place deep foundation element not enclosed by a structural steel pipe or tube shall be determined using the following equation:

$$\phi M_n = 3\sqrt{f'_c}\, S_m$$ **(Equation 18-10)**

where:
f'_c = Specified compressive strength of concrete or grout, psi (MPa)
S_m = Elastic section modulus, neglecting reinforcement and casing, in^3 (mm^3)

1810.3.9.2 Required reinforcement. Where subject to uplift or where the required moment strength determined using the load combinations of Section 1605.2 exceeds the design cracking moment determined in accordance with Section 1810.3.9.1, cast-in-place deep foundations not enclosed by a structural steel pipe or tube shall be reinforced.

~~1810.1.2~~ 1810.3.9.3 Placement of reinforcement. ~~Except for steel dowels embedded 5 feet (1524 mm) or less in the pile and as provided in Section 1810.3.4,~~ Reinforcement where required shall be assembled and tied together and shall be placed in the ~~pile~~ deep foundation element as a unit before the reinforced portion of the ~~pile~~ element is filled with concrete ~~except in augered uncased cast-in-place piles. Tied reinforcement in augered uncased cast-in-place piles shall be placed after piles are concreted, while the concrete is still in a semifluid state~~.

Exceptions:

1. Steel dowels embedded 5 feet (1524 mm) or less shall be permitted to be placed after concreting, while the concrete is still in a semifluid state.
2. **~~1810.3.4 Reinforcement.~~** For ~~piles~~ deep foundation elements installed with a hollow-stem auger ~~where full-length~~, tied reinforcement shall be placed after elements are concreted, while the concrete is still in a semifluid state. Longitudinal ~~steel~~ reinforcement ~~is placed~~ without lateral ties~~, the reinforcement~~ shall be placed either through the hollow stem of the auger prior to ~~filling the pile with concrete~~ concreting or after concreting, while the concrete is still in a semifluid state. ~~All pile reinforcement shall have a concrete cover of not less than 2.5 inches (64 mm).~~

Exception: Where physical constraints do not allow the placement of the longitudinal reinforcement prior to filling the pile with concrete or where partial-length longitudinal reinforcement is placed without lateral ties, the reinforcement is allowed to be placed after the piles are completely concreted but while concrete is still in a semifluid state.

3. For Group R-3 and U occupancies not exceeding two stories of light-frame construction, reinforcement is permitted to be placed after concreting, while the concrete is still in a semifluid state, and the concrete cover requirement is permitted to be reduced to 2 inches (51 mm), provided the construction method can be demonstrated to the satisfaction of the building official.

~~**1812.4**~~ **1810.3.9.4** ~~**Reinforcement**~~ **Seismic reinforcement.** ~~Except for steel dowels embedded 5 feet (1524 mm) or less in the pier, reinforcement where required shall be assembled and tied together and shall be placed in the pier hole as a unit before the reinforced portion of the pier is filled with concrete~~ Where a structure is assigned to Seismic Design Category C reinforcement shall be provided in accordance with Section 1810.3.9.4.1. Where a structure is assigned to Seismic Design Category D, E, or F reinforcement shall be provided in accordance with Section 1810.3.9.4.2.

~~**Exception:** Reinforcement is permitted to be wet set and the 2-1/2 inch (64 mm) concrete cover requirement be reduced to 2 inches (51 mm) for Group R-3 and U occupancies not exceeding two stories of light-frame construction, provided the construction method can be demonstrated to the satisfaction of the building official.~~

~~Reinforcement shall conform to the requirements of Sections 1810.1.2.1 and 1810.1.2.2.~~

Exceptions:

1. Isolated ~~piers~~ deep foundation elements supporting posts of Group R-3 and U occupancies not exceeding two stories of light-frame construction ~~are~~ shall be permitted to be reinforced as required by rational analysis but with not less than ~~a minimum of~~ one No. 4 bar, without ties or spirals, ~~when~~ where detailed so the ~~pier~~ element is not subject to lateral loads and the soil ~~is determined to be of adequate stiffness~~ provides adequate lateral support in accordance with Section 1810.2.1.
2. Isolated ~~piers~~ deep foundation elements supporting posts and bracing from decks and patios appurtenant to Group R-3 and U occupancies not exceeding two stories of light-frame construction ~~are~~ shall be permitted to be reinforced as required by rational analysis but with not less than one No. 4 bar, without ties or spirals, ~~when~~ where the lateral load, *E*, to the top of the ~~pier~~ element does not exceed 200 pounds (890 N) and the soil ~~is determined to be of adequate stiffness~~ provides adequate lateral support in accordance with Section 1810.2.1.
3. ~~Piers~~ Deep foundation elements supporting the concrete foundation wall of Group R-3 and U occupancies not exceeding two stories of light-frame construction ~~are~~ shall be permitted to be reinforced as required by rational analysis but with not less than two No. 4 bars, without ties or spirals, ~~when it can be shown the concrete pier will not rupture when designed for the maximum seismic load, E_m,~~ where the design cracking moment determined in accordance with Section 1810.3.9.1 exceeds the required moment strength determined using the load combinations with overstrength factor in Section 12.4.3.2 of ASCE 7 and the soil ~~is determined to be of adequate stiffness~~ provides adequate lateral support in accordance with Section 1810.2.1.
4. Closed ties or spirals where required by Section ~~1810.1.2.2~~ 1810.3.9.4.2 ~~are~~ shall be permitted to be limited to the top 3 feet (914 mm) of ~~the piers~~ deep foundation elements 10 feet (3048 mm) or less in depth supporting Group R-3 and U occupancies of Seismic Design Category D, not exceeding two stories of light-frame construction.

~~**1810.1.2.1**~~ **1810.3.9.4.1 Seismic reinforcement in Seismic Design Category C.** For ~~Where a~~ structures ~~is~~ assigned to Seismic Design Category C in accordance with Section 1613, ~~the following shall apply~~ cast-in-place deep foundation elements shall be reinforced as specified in this section. Reinforcement shall be provided where required by analysis.

A minimum of four longitudinal bars, with a minimum longitudinal reinforcement ratio of 0.0025, shall be provided ~~for uncased cast-in-place concrete drilled or augered piles, piers or caissons in the top~~ throughout the minimum reinforced length of the element as defined below starting at the top of the element. The minimum reinforced length of the element shall be taken as the greatest of the following:

1. One-third of the ~~pile~~ element length;
2. A ~~minimum length~~ distance of 10 feet (3048 mm);
3. Three times the least element dimension; and
4. The distance from the top of the element to the point where the design cracking moment determined in accordance with Section 1810.3.9.1 exceeds the required moment strength determined using the load combinations of Section 1605.2. ~~below the ground or that required by analysis, whichever length is greatest. The minimum reinforcement ratio, but no less than that ratio required by rational analysis, shall be continued throughout the flexural length of the pile. There shall be a minimum of four longitudinal bars with~~

Transverse reinforcement shall consist of closed ties ~~(or equivalent~~ spirals~~) of~~ with a minimum 3/8 inch (9.5 mm) diameter ~~provided at 16-longitudinal-bar diameter maximum spacing. Transverse confinement reinforcement with a maximum~~. Spacing of transverse reinforcement shall not exceed the smaller of 6 inches (152 mm) or 8-longitudinal-bar diameters~~, whichever is less, shall be provided~~ within a distance ~~equal to~~ of three times the least ~~pile~~ element dimension ~~of~~ from the bottom of the pile cap. Spacing of transverse reinforcement shall not exceed 16 longitudinal bar diameters throughout the remainder of the reinforced length.

Exceptions:

1. The requirements of this section shall not apply to concrete cast in structural steel pipes or tubes.
2. A spiral-welded metal casing of a thickness not less than manufacturer's standard gage No. 14 gage (0.068 inch) is permitted to provide concrete confinement in lieu of the closed ties or spirals. Where used as such, the metal casing shall be protected against possible deleterious action due to soil constituents, changing water levels or other factors indicated by boring records of site conditions.

~~1810.1.2.2~~ 1810.3.9.4.2 Seismic reinforcement in Seismic Design ~~Category~~ Categories D~~, E or~~ through F. For ~~Where a~~ structures ~~is~~ assigned to Seismic Design Category D, E or F in accordance with Section 1613, ~~the requirements for Seismic Design Category C given above shall be met, in addition to the following~~ cast-in-place deep foundation elements shall be reinforced as specified in this section. Reinforcement shall be provided where required by analysis.

A minimum of four longitudinal bars, with a minimum longitudinal reinforcement ratio of 0.005 shall be provided ~~for uncased cast-in-place drilled or augered concrete piles, piers or caissons in the top~~ throughout the minimum reinforced length of the element as defined below starting at the top of the element. The minimum reinforced length of the element shall be taken as the greatest of the following:

1. One-half of the ~~pile~~ element length;
2. A ~~minimum length~~ distance of 10 feet (3048 mm);
3. Three times the least element dimension; and
4. The distance from the top of the element to the point where the design cracking moment determined in accordance with Section 1810.3.9.1 exceeds the required moment strength determined using the load combinations of Section 1605.2.

~~below ground or throughout the flexural length of the pile, whichever length is greatest. The flexural length shall be taken as the length of the pile to a point where the concrete section cracking moment strength multiplied by 0.4 exceeds the required moment strength at that point. There shall be a minimum of four longitudinal bars with transverse confinement reinforcement provided in the pile in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within three times the least pile dimension of the bottom of the pile cap. A transverse spiral reinforcement ratio of not less than one-half of that required in Section 21.4.4.1(a) of ACI 318 for other than Class E, F or liquefiable sites is permitted. Tie spacing throughout the remainder of the concrete section shall not exceed 12 longitudinal bar diameters, one-half the least dimension of the section, nor 12 inches (305 mm). Ties shall be a minimum of No. 3 bars for piles with a least dimension up to 20 inches (508 mm), and No. 4 bars for larger piles.~~

Transverse reinforcement shall consist of closed ties or spirals no smaller than No. 3 bars for elements with a least dimension up to 20 inches (508 mm), and No. 4 bars for larger elements. Throughout the remainder of the reinforced length outside the regions with transverse confinement reinforcement, as specified in Section 1810.3.9.4.2.1 or 1810.3.9.4.2.2, the spacing of transverse reinforcement shall not exceed the least of the following:

1. 12 longitudinal bar diameters;
2. One-half the least dimension of the element; and
3. 12 inches (305 mm).

Exceptions:

1. The requirements of this section shall not apply to concrete cast in structural steel pipes or tubes.
2. ~~**1810.5.4.1 Seismic reinforcement.** Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 1613, the reinforcement requirements for drilled or augered uncased piles in Section 1810.3.5 shall be met.~~

   ~~**Exception:**~~ A spiral-welded metal casing of a thickness not less than manufacturer's standard gage No. 14 gage (0.068 inch) is permitted to provide concrete confinement in lieu of the closed ties or ~~equivalent~~ spirals ~~required in an uncased concrete pile~~. Where used as such, the metal casing shall be protected against possible deleterious action due to soil constituents, changing water levels or other factors indicated by boring records of site conditions.

1810.3.9.4.2.1 Site Classes A through D. For Site Class A, B, C, or D sites, transverse confinement reinforcement shall be provided in the element in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within three times the least element dimension of the bottom of the pile cap. A transverse spiral reinforcement ratio of not less than one-half of that required in Section 21.4.4.1(a) of ACI 318 shall be permitted.

1810.3.9.4.2.2 Site Classes E and F. For Site Class E or F sites, transverse confinement reinforcement shall be provided in the element in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within seven times the least element dimension of the pile cap and within seven times the least element dimension of the interfaces of strata that are hard or stiff and strata that are liquefiable or are composed of soft to medium stiff clay.

~~1812.6~~ 1810.3.9.5 Belled ~~bottoms~~ drilled shafts. Where ~~pier foundations~~ drilled shafts are belled at the bottom, the edge thickness of the bell shall not be less than that required for the edge of footings. Where the sides of the bell slope at an angle less than 60 degrees (1 rad) from the horizontal, the effects of vertical shear shall be considered.

~~**1810.7.1**~~ **1810.3.9.6** ~~**Construction**~~ **Socketed drilled shafts.** ~~Caisson piles shall consist of a shaft section of concrete-filled pipe extending to bedrock with an uncased socket drilled into the bedrock and filled with concrete.~~ Socketed drilled shafts shall have a permanent pipe or tube casing that extends down to bedrock and an uncased socket drilled into the bedrock, both filled with concrete. ~~The caisson pile~~ Socketed drilled shafts shall have a full-length structural steel core or a stub core installed in the rock socket and extending into the pipe portion a distance equal to the socket depth.

~~**1810.7.3 Design.**~~ The depth of the rock socket shall be sufficient to develop the full load-bearing capacity of the ~~caisson pile~~ element with a minimum safety factor of two, but the depth shall not be less than the outside diameter of the pipe or tube casing. The design of the rock socket is permitted to be predicated on the sum of the allowable load-bearing pressure on the bottom of the socket plus bond along the sides of the socket. ~~The minimum outside diameter of the caisson pile shall be 18 inches (457 mm), and the diameter of the rock socket shall be approximately equal to the inside diameter of the pile.~~

~~**1810.7.4 Structural core.**~~ The gross cross-sectional area of the structural steel core shall not exceed 25 percent of the gross area of the ~~caisson~~ drilled shaft. ~~The minimum clearance between the structural core and the pipe shall be 2 inches (51 mm). Where cores are to be spliced, the ends shall be milled or ground to provide full contact and shall be full-depth welded.~~

~~**1810.8**~~ **1810.3.10** **[Supp] Micropiles.** Micropiles shall ~~comply with the requirements of~~ be designed and detailed in accordance with Sections ~~1810.8.1 through 1810.8.5~~ 1810.3.10.1 through 1810.3.10.4.

~~**1810.8.1**~~ **1810.3.10.1** **[Supp] Construction.** ~~Micropiles shall consist of a grouted section reinforced with steel pipe or steel reinforcement.~~ Micropiles shall develop their load-carrying capacity ~~through~~ by means of a bond zone in soil, bedrock or a combination of soil and bedrock. ~~The steel pipe or steel reinforcement shall extend the full length of the micropile.~~ Micropiles shall be grouted and have either a steel pipe or tube or steel reinforcement at every section along the length. It shall be permitted to transition from deformed reinforcing bars to steel pipe or tube reinforcement by extending the bars into the pipe or tube section by at least their tension development length.

~~**1810.8.2**~~ **1810.3.10.2** **[Supp] Materials.** ~~Grout shall have a specified compressive strength (f'_c) of not less than 4,000 psi (27.58 Mpa). The grout mix shall be designed and proportioned so as to produce a pumpable mixture.~~ Reinforcement shall consist of deformed reinforcing bars in accordance with ASTM A 615 Grade 60 or 75 or ASTM A 722 Grade 150.

The steel pipe or tube shall have a minimum wall thickness of 3/16 inch (4.8 mm). Splices shall comply with Section ~~1808.2.7~~ 1810.3.6. The steel pipe or tube shall have a minimum yield strength ~~exceeding~~ of 45,000 psi (310 MPa) and a minimum elongation of 15 percent as shown by mill certifications or two coupon test samples per 40,000 pounds (18 160 kg) of pipe or tube.

~~**1810.8.4**~~ **1810.3.10.3** **[Supp] Reinforcement.** For micropiles or portions thereof ~~piles~~ grouted inside a temporary or permanent casing or inside a hole drilled into bedrock or a hole drilled with grout, the steel pipe or tube or steel reinforcement shall be designed to carry at least 40 percent of the design compression load. Micropiles or portions thereof ~~piles~~ grouted in an open hole in soil without temporary or permanent casing and without suitable means of verifying the hole diameter during grouting shall be designed to carry the entire compression load in the reinforcing steel. Where a steel pipe or tube is used for reinforcement, the portion of the grout enclosed within the pipe is permitted to be included in the determination of the allowable stress in the grout.

~~**1810.8.4.1**~~ **1810.3.10.4** **[Supp] Seismic reinforcement.** ~~Where a~~ For structures ~~is~~ assigned to Seismic Design Category C, a permanent steel casing shall be provided from the top of the micropile down ~~a minimum of 120 percent of the flexural length~~ to the point of zero curvature. ~~Where a~~ For structures ~~is~~ assigned to Seismic Design D, E or F, the micropile shall be considered as an alternative system in accordance with Section 104.11. The alternative ~~pile~~ system design, supporting documentation and test data shall be submitted to the building official for review and approval.

~~**1808.2.4**~~ **1810.3.11** **Pile caps.** Pile caps shall be of reinforced concrete, and shall include all elements to which ~~piles~~ vertical deep foundation elements are connected, including grade beams and mats. The soil immediately below the pile cap shall not be considered as carrying any vertical load. The tops of ~~piles~~ vertical deep foundation elements shall be embedded not less than 3 inches (76 mm) into pile caps and the caps shall extend at least 4 inches (102 mm) beyond the edges of ~~piles~~ the elements. The tops of ~~piles~~ elements shall be cut or chipped back to sound material before capping.

~~**1808.2.23.1.1**~~ **1810.3.11.1** ~~**Connection to pile cap**~~ **Seismic Design Categories C through F.** ~~Concrete piles and concrete-filled steel pipe piles~~ For structures assigned to Seismic Design Category C, D, E, or F in accordance with Section 1613, concrete deep foundation elements shall be connected to the pile cap by embedding the ~~pile~~ element reinforcement or field-placed dowels anchored in the ~~concrete pile~~ element into the pile cap for a distance equal to ~~the~~ their development length in accordance with ACI 318. It shall be permitted to connect precast prestressed piles to the pile cap by developing the element prestressing strands into the pile cap provided the connection is ductile. For deformed bars, the development length is the full development length for compression, or tension~~,~~ in the case of uplift, without reduction ~~in length~~ for excess ~~area~~ reinforcement in accordance with Section 12.2.5 of ACI 318. Alternative measures for laterally confining concrete and maintaining toughness and ductile-like behavior at the top of the ~~pile will~~ element shall be permitted provided the design is such that any hinging occurs in the confined region.

~~Ends of hoops, spirals and ties shall be terminated with seismic hooks, as defined in Section 21.1 of ACI 318 turned into the confined concrete core.~~ The minimum transverse steel ratio for confinement shall not be less than one-half of that required for columns.

For resistance to uplift forces, anchorage of steel ~~pipe (round HSS sections), concrete-filled steel pipe or~~ pipes, tubes, or H-piles to the pile cap shall be made by means other than concrete bond to the bare steel section. Concrete-filled steel pipes or tubes shall have reinforcement of not less than 0.01 times the cross-sectional area of the concrete fill developed into the cap and extending into the fill a length equal to two times the required cap embedment, but not less than the tension development length of the reinforcement.

> ~~**Exception:** Anchorage of concrete-filled steel pipe piles is permitted to be accomplished using deformed bars developed into the concrete portion of the pile.~~

~~Splices of pile segments shall develop the full strength of the pile, but the splice need not develop the nominal strength of the pile in tension, shear and bending when it has been designed to resist axial and shear forces and moments from the load combinations of Section 1605.4.~~

~~1808.2.23.2.2~~ 1810.3.11.2 ~~Connection to pile cap~~ Seismic Design Categories D through F. For ~~piles required to resist~~ structures assigned to Seismic Design Category D, E, or F in accordance with Section 1613, deep foundation element resistance to uplift forces or ~~provide~~ rotational restraint~~, design of anchorage of piles into the pile cap shall be provided~~ 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 a minimum of 25 percent of the strength of the ~~pile~~ element in tension. Anchorage into the pile cap shall be capable of developing the following:

1. In the case of uplift, the ~~lesser~~ least of the following: nominal tensile strength of the longitudinal reinforcement in a concrete ~~pile~~ element~~,~~; ~~or~~ the nominal tensile strength of a steel ~~pile~~ element~~,~~; ~~or~~ the ~~pile uplift soil nominal strength factored~~ frictional force developed between the element and the soil multiplied by 1.3; ~~or~~ and the axial tension force resulting from the load combinations of Section 1605.4.
2. In the case of rotational restraint, the lesser of the following: the axial ~~and~~ force, shear forces, and bending moments resulting from the load combinations of Section 1605.4; ~~or~~ and development of the full axial, bending and shear nominal strength of the ~~pile~~ element.

~~1808.2.23.2.3 Flexural strength.~~ Where the vertical lateral-force-resisting elements are columns, the ~~grade beam or~~ pile cap flexural strengths shall exceed the column flexural strength. The connection between batter piles and ~~grade beams or~~ 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 capable of resisting forces and moments from the load combinations of Section 1605.4.

1810.3.12 Grade beams. For structures assigned to Seismic Design Category D, E, or F in accordance with Section 1613, grade beams shall comply with the provisions in Section 21.10.3 of ACI 318 for grade beams, except where they have the capacity to resist the forces from the load combinations with overstrength factor in Section 12.4.3.2 of ASCE 7.

~~1808.2.23.1~~ 1810.3.13 [Supp] ~~Seismic Design Category C~~ Seismic ties. ~~Where a~~ For structures ~~is~~ assigned to Seismic Design Category C, D, E, or F in accordance with Section 1613, ~~the following shall apply.~~ Individual ~~pile caps, piers or piles~~ deep foundations shall be interconnected by ties. Ties shall be capable of carrying, in tension and compression, a force equal to the product of the larger pile cap or column load times the seismic coefficient, S_{DS}, divided by 10 unless it can be demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils or very dense granular soils.

> **Exception:** In Group R-3 and U occupancies of light-frame construction, ~~pier foundations~~ deep foundation elements supporting foundation walls, isolated interior posts detailed so the ~~pier~~ element is not subject to lateral loads, or exterior decks and patios are not subject to interconnection ~~if it can be shown~~ where the soils are of adequate stiffness, subject to the approval of the building official.

1810.4 Installation. Deep foundations shall be installed in accordance with Section 1810.4. Where a single deep foundation element comprises two or more sections of different materials or different types spliced together, each section shall satisfy the applicable conditions of installation.

~~1808.2.6~~ 1810.4.1 Structural integrity. ~~Piers or piles~~ Deep foundation elements shall be installed in such a manner and sequence as to prevent distortion or damage that may adversely affect the structural integrity of ~~piles~~ adjacent structures or of foundation elements being installed or already in place and as to avoid compacting the surrounding soil to the extent that other foundation elements cannot be installed properly.

~~1809.2.2.4~~ 1810.4.1.1 ~~Installation~~ Compressive strength of precast concrete piles. A precast concrete pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the ~~28-day~~ specified compressive strength (f'_c), but not less than the strength sufficient to withstand handling and driving forces.

1810.4.1.2 Casing. Where cast-in-place deep foundation elements are formed through unstable soils and concrete is placed in an open-drilled hole, a casing shall be inserted in the hole prior to placing the concrete. Where the casing is withdrawn during concreting, the level of concrete shall be maintained above the bottom of the casing at a sufficient height to offset any hydrostatic or lateral soil pressure. Driven casings shall be mandrel driven their full length in contact with the surrounding soil.

~~1810.4.3~~ 1810.4.1.3 ~~Installation~~ Driving near uncased concrete. ~~Piles~~ Deep foundation elements shall not be driven within six ~~pile~~ element diameters center to center in granular soils or within one-half the ~~pile~~ element length in cohesive soils of ~~a pile~~ an uncased element filled with concrete less than 48 hours old unless approved by the building official. If the concrete surface in any completed ~~pile~~ element rises or drops, the ~~pile~~ element shall be replaced. ~~Piles~~ Driven uncased deep foundation elements shall not be installed in soils that could cause ~~pile~~ heave.

~~1810.5.3~~ 1810.4.1.4 ~~Installation~~ Driving near cased concrete. ~~Steel shells shall be mandrel driven their full length in contact with the surrounding soil.~~

~~The steel shells shall be driven in such order and with such spacing as to ensure against distortion of or injury to piles already in place. A pile~~ Deep foundation elements shall not be driven within four and one-half average ~~pile~~ diameters of a ~~pile~~ cased element filled with concrete less than 24 hours old unless approved by the building official. Concrete shall not be placed in ~~steel shells~~ casings within heave range of driving.

~~1809.1.3~~ 1810.4.1.5 Defective timber piles. Any substantial sudden increase in rate of penetration of a timber pile shall be investigated for possible damage. If the sudden increase in rate of penetration cannot be correlated to soil strata, the pile shall be removed for inspection or rejected.

~~1808.2.20~~ 1810.4.2 Identification. ~~Pier or pile~~ Deep foundation materials shall be identified for conformity to the specified grade with this identity maintained continuously from the point of manufacture to the point of installation or shall be tested by an approved agency to determine conformity to the specified grade. The approved agency shall furnish an affidavit of compliance to the building official.

~~1808.2.21~~ 1810.4.3 ~~Pier or pile~~ Location plan. A plan showing the location and designation of ~~piers or piles~~ deep foundation elements by an identification system shall be filed with the building official prior to installation of such ~~piers or piles~~ elements. Detailed records for ~~piers or individual piles~~ elements shall bear an identification corresponding to that shown on the plan.

~~1808.2.13~~ 1810.4.4 Preexcavation. The use of jetting, augering or other methods of preexcavation shall be subject to the approval of the building official. Where permitted, preexcavation shall be carried out in the same manner as used for ~~piers or piles~~ deep foundation elements subject to load tests and in such a manner that will not impair the carrying capacity of the ~~piers or piles~~ elements already in place or damage adjacent structures. ~~Pile~~ Element tips shall be driven below the preexcavated depth until the required resistance or penetration is obtained.

~~1808.2.15~~ 1810.4.5 ~~Use of~~ Vibratory ~~drivers~~ driving. Vibratory drivers shall only be used to install ~~piles~~ deep foundation elements where the ~~pile~~ element load capacity is verified by load tests in accordance with Section ~~1808.2.8.3~~ 1810.3.3.1.2. The installation of production ~~piles~~ elements shall be controlled according to power consumption, rate of penetration or other approved means that ensure ~~pile~~ element capacities equal or exceed those of the test ~~piles~~ elements.

~~1808.2.19~~ 1810.4.6 Heaved ~~piles~~ elements. ~~Piles~~ Deep foundation elements that have heaved during the driving of adjacent ~~piles~~ elements shall be redriven as necessary to develop the required capacity and penetration, or the capacity of the ~~pile~~ element shall be verified by load tests in accordance with Section ~~1808.2.8.3~~ 1810.3.3.1.2.

1810.4.7 Enlarged base cast-in-place elements ~~1810.2.3 Installation.~~ Enlarged bases for cast-in-place deep foundation elements formed ~~either~~ by compacting concrete or by driving a precast base shall be formed in or driven into granular soils. ~~Piles~~ Such elements shall be constructed in the same manner as successful prototype test ~~piles~~ elements driven for the project. ~~Pile~~ Shafts extending through peat or other organic soil shall be encased in a permanent steel casing. Where a cased shaft is used, the shaft shall be adequately reinforced to resist column action or the annular space around the ~~pile~~ shaft shall be filled sufficiently to reestablish lateral support by the soil. Where ~~pile~~ heave occurs, the ~~pile~~ element shall be replaced unless it is demonstrated that the ~~pile~~ element is undamaged and capable of carrying twice its design load.

~~1810.3.3~~ 1810.4.8 ~~Installation~~ Hollow-stem augered, cast-in-place elements. ~~Where pile shafts are formed through unstable soils and concrete is placed in an open-drilled hole, a steel liner shall be inserted in the hole prior to placing the concrete. Where the steel liner is withdrawn during concreting, the level of concrete shall be maintained above the bottom of the liner at a sufficient height to offset any hydrostatic or lateral soil pressure.~~

Where concrete is placed by pumping through a hollow-stem auger, the auger shall be permitted to rotate in a clockwise direction during withdrawal. The auger shall be withdrawn in continuous increments. Concreting pumping pressures shall be measured and maintained high enough at all times to offset hydrostatic and lateral earth pressures. Concrete volumes shall be measured to ensure that the volume of concrete placed in each ~~pile~~ element is equal to or greater than the theoretical volume of the hole created by the auger. Where the installation process of any ~~pile~~ element is interrupted or a loss of concreting pressure occurs, the ~~pile~~ element shall be redrilled to 5 feet (1524 mm) below the elevation of the tip of the auger when the installation was interrupted or concrete pressure was lost and reformed. Augered cast-in-place ~~piles~~ elements shall not be installed within six ~~pile~~ diameters center to center of ~~a pile~~ an element filled with concrete less than 12 hours old, unless approved by the building official. If the concrete level in any completed ~~pile~~ element drops due to installation of an adjacent ~~pile~~ element, the ~~pile~~ element shall be replaced.

~~1810.7.6~~ 1810.4.9 ~~Installation~~ Socketed drilled shafts. The rock socket and ~~pile~~ pipe or tube casing of socketed drilled shafts shall be thoroughly cleaned of foreign materials before filling with concrete. Steel cores shall be bedded in cement grout at the base of the rock socket. ~~Concrete shall not be placed through water except where a tremie or other approved method is used.~~

~~1810.8.5~~ 1810.4.10 ~~Installation~~ Micropiles. ~~The pile~~ Micropile deep foundation elements shall be permitted to be formed in ~~a~~ holes advanced by rotary or percussive drilling methods, with or without casing. The ~~pile~~ elements shall be grouted with a fluid cement grout. The grout shall be pumped through a tremie pipe extending to the bottom of the ~~pile~~ element until grout of suitable quality returns at the top of the ~~pile~~ element. The following requirements apply to specific installation methods:

1. For micropiles grouted inside a temporary casing, the reinforcing bars shall be inserted prior to withdrawal of the casing. The casing shall be withdrawn in a controlled manner with the grout level maintained at the top of the ~~pile~~ element to ensure that the grout completely fills the drill hole. During withdrawal of the casing, the grout level inside the casing shall be monitored to ~~check~~ verify that the flow of grout inside the casing is not obstructed.
2. For a micropile or portion thereof ~~a pile~~ grouted in an open drill hole in soil without temporary casing, the minimum design diameter of the drill hole shall be verified by a suitable device during grouting.
3. For micropiles designed for end bearing, a suitable means shall be employed to verify that the bearing surface is properly cleaned prior to grouting.
4. Subsequent micropiles shall not be drilled near ~~piles~~ elements that have been grouted until the grout has had sufficient time to harden.
5. Micropiles shall be grouted as soon as possible after drilling is completed.
6. For micropiles designed with a full length casing, the casing shall be pulled back to the top of the bond zone and reinserted or some other suitable means employed to assure grout coverage outside the casing.

~~1808.2.22~~ 1810.4.11 Special inspection. Special inspections in accordance with Sections 1704.8 and 1704.9 shall be provided for ~~piles and piers~~ driven and cast-in-place deep foundation elements, respectively.

1808.2.23.2 Seismic Design Category D, E or F. ~~Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613, the requirements for Seismic Design Category C given in Section 1808.2.23.1 shall be met, in addition to the following.~~ Provisions of ACI 318, Section 21.10.4, shall apply when not in conflict with the provisions of Sections 1808 through 1812. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.

Exceptions:

1. Group R or U occupancies of light-framed construction and two stories or less above grade plane are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2 MPa) at 28 days.
2. Detached one- and two-family dwellings of light-frame construction and two stories or less in height are not required to comply with the provisions of ACI 318, Section 21.10.4.
3. Section 21.10.4.4(a) of ACI 318 need not apply to concrete piles.

1809.2.3.1 Materials. ~~Prestressing steel shall conform to ASTM A 416.~~ Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 5,000 psi (34.48 MPa).

1810.5.4 Reinforcement. Reinforcement shall not be placed within 1 inch (25 mm) of the steel shell. ~~Reinforcing shall be required for unsupported pile lengths or where the pile is designed to resist uplift or unbalanced lateral loads.~~

1810.6.4 Reinforcement. ~~Reinforcement steel shall conform to Section 1810.1.2.~~ Reinforcement shall not be placed within 1 inch (25 mm) of the steel casing.

1810.7.2 Materials. ~~Pipe and steel cores shall conform to the material requirements in Section 1809.3. Pipes shall have a minimum wall thickness of 3/8 inch (9.5 mm) and shall be fitted with a suitable steel-driving shoe welded to the bottom of the pipe.~~ Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 4,000 psi (27.58 MPa). The concrete mix shall be designed and proportioned so as to produce a cohesive workable mix with a slump of 4 inches to 6 inches (102 mm to 152 mm).

Delete without substitution:

~~**1808.2.8 Allowable pier or pile loads.**~~

~~**1808.2.14 Installation sequence.** Piles shall be installed in such sequence as to avoid compacting the surrounding soil to the extent that other piles cannot be installed properly, and to prevent ground movements that are capable of damaging adjacent structures.~~

~~**1808.2.16 Pile driveability.** Pile cross sections shall be of sufficient size and strength to withstand driving stresses without damage to the pile, and to provide sufficient stiffness to transmit the required driving forces.~~

~~**1808.2.23 Seismic design of piers or piles.**~~

~~SECTION 1809~~
~~DRIVEN PILE FOUNDATIONS~~

~~**1809.1.4 Allowable stresses**. The allowable stresses shall be in accordance with the AF&PA NDS.~~

~~**1809.2 Precast concrete piles.**~~

~~**1809.2.1.4 Installation.** Piles shall be handled and driven so as not to cause injury or overstressing, which affects durability or strength.~~

~~**1809.2.2.1 Materials.** Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 3,000 psi (20.68 MPa).~~

~~**1809.2.2.3 Allowable stresses.** The allowable compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (f'_c) applied to the gross cross-sectional area of the pile. The allowable compressive stress in the reinforcing steel shall not exceed 40 percent of the yield strength of the steel (f_y) or a maximum of 30,000 psi (207 MPa). The allowable tensile stress in the reinforcing steel shall not exceed 50 percent of the yield strength of the steel (f_y) or a maximum of 24,000 psi (165 MPa).~~

~~**1809.2.2.5 Concrete cover.** Reinforcement for piles that are not manufactured under plant conditions shall have a concrete cover of not less than 2 inches (51 mm).~~

~~Reinforcement for piles manufactured under plant control conditions shall have a concrete cover of not less than 1.25 inches (32 mm) for No. 5 bars and smaller, and not less than 1.5 inches (38 mm) for No. 6 through No. 11 bars except that longitudinal bars spaced less than 1.5 inches (38 mm) clear distance apart shall be considered bundled bars for which the minimum concrete cover shall be equal to that for the equivalent diameter of the bundled bars. Reinforcement for piles exposed to seawater shall have a concrete cover of not less than 3 inches (76 mm).~~

~~**1809.2.3.3 Allowable stresses.** The allowable design compressive stress, f_c, in concrete shall be determined as follows:~~

~~$f_c = 0.33 f'_c - 0.27 f_{pc}$ **(Equation 18-10)**~~

~~where:~~

~~f'_c = The 28-day specified compressive strength of the concrete.~~
~~f_{pc} = The effective prestress stress on the gross section.~~

~~**1809.2.3.4 Installation.** A prestressed pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the 28-day specified compressive strength ($f'c$), but not less than the strength sufficient to withstand handling and driving forces.~~

~~**1809.2.3.5 Concrete cover.** Prestressing steel and pile reinforcement shall have a concrete cover of not less than 1-1/4 inches (32 mm) for square piles of 12 inches (305 mm) or smaller size and 1-1/2 inches (38 mm) for larger piles, except that for piles exposed to seawater, the minimum protective concrete cover shall not be less than 2-1/2 inches (64 mm).~~

~~**1809.3 Structural steel piles.** Structural steel piles shall conform to the requirements of Sections 1809.3.1 through 1809.3.4.~~

~~**1809.3.2 Allowable stresses.** The allowable axial stresses shall not exceed 35 percent of the minimum specified yield strength (F_y).~~

> ~~**Exception:** Where justified in accordance with Section 1808.2.10, the allowable axial stress is permitted to be increased above $0.35F_y$, but shall not exceed $0.5F_y$.~~

~~SECTION 1810~~
~~CAST-IN-PLACE CONCRETE PILE FOUNDATIONS~~

~~**1810.1 General.** The materials, reinforcement and installation of cast-in-place concrete piles shall conform to Sections 1810.1.1 through 1810.1.3.~~

~~**1810.1.3 Concrete placement.** Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. Concrete shall not be placed through water except where a tremie or other approved method is used. When depositing concrete from the top of the pile, the concrete shall not be chuted directly into the pile but shall be poured in a rapid and continuous operation through a funnel hopper centered at the top of the pile.~~

~~**1810.2 Enlarged base piles.** Enlarged base piles shall conform to the requirements of Sections 1810.2.1 through 1810.2.5.~~

~~**1810.2.2 Allowable stresses.** The maximum allowable design compressive stress for concrete not placed in a permanent steel casing shall be 25 percent of the 28-day specified compressive strength (f'_c). Where the concrete is place in a permanent steel casing, the maximum allowable concrete stress shall be 33 percent of the 28-day specified compressive strength (f'_c).~~

~~**1810.2.4 Load-bearing capacity.** Pile load-bearing capacity shall be verified by load tests in accordance with Section 1808.2.8.3.~~

~~**1810.2.5 Concrete cover.** The minimum concrete cover shall be 2-1/2 inches (64 mm) for uncased shafts and 1 inch (25 mm) for cased shafts.~~

~~**1810.3 Drilled or augered uncased piles.** Drilled or augered uncased piles shall conform to Sections 1810.3.1 through 1810.3.5.~~

~~**1810.3.1 Allowable stresses.** The allowable design stress in the concrete of drilled or augered uncased piles shall not exceed 33 percent of the 28-day specified compressive strength (f'_c). The allowable compressive stress of reinforcement shall not exceed 40 percent of the yield strength of the steel or 25,500 psi (175.8 MPa).~~

~~**1810.3.5 Reinforcement in Seismic Design Category C, D, E or F.** Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 1613, the corresponding requirements of Sections 1810.1.2.1 and 1810.1.2.2 shall be met.~~

~~**1810.4 Driven uncased piles.** Driven uncased piles shall conform to Sections 1810.4.1 through 1810.4.4.~~

~~**1810.4.1 Allowable stresses.** The allowable design stress in the concrete shall not exceed 25 percent of the 28-day specified compressive strength (f'_c) applied to a cross-sectional area not greater than the inside area of the drive casing or mandrel.~~

~~**1810.4.2 Dimensions.** The pile length shall not exceed 30 times the average diameter. The minimum diameter shall be 12 inches (305 mm).~~

> ~~**Exception:** The length of the pile is permitted to exceed 30 times the diameter, provided that the design and installation of the pile foundation is under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pile foundations. The registered design professional shall certify to the building official that the piles were installed in compliance with the approved design.~~

~~**1810.4.4 Concrete cover.** Pile reinforcement shall have a concrete cover of not less than 2.5 inches (64 mm), measured from the inside face of the drive casing or mandrel.~~

~~**1810.5 Steel-cased piles.** Steel-cased piles shall comply with the requirements of Sections 1810.5.1 through 1810.5.4.~~

~~**1810.6 Concrete-filled steel pipe and tube piles.** Concrete-filled steel pipe and tube piles shall conform to the requirements of Sections 1810.6.1 through 1810.6.5.~~

~~**1810.6.1 Materials.** Steel pipe and tube sections used for piles shall conform to ASTM A 252 or ASTM A 283. Concrete shall conform to Section 1810.1.1. The maximum coarse aggregate size shall be 3/4 inch (19.1 mm).~~

~~**1810.6.2 Allowable stresses.** The allowable design compressive stress in the concrete shall not exceed 33 percent of the 28-day specified compressive strength (f'_c). The allowable design compressive stress in the steel shall not exceed 35 percent of the minimum specified yield strength of the steel (F_y), provided F_y shall not be assumed greater than 36,000 psi (248 MPa) for computational purposes.~~

> ~~**Exception:** Where justified in accordance with Section 1808.2.10, the allowable stresses are permitted to be increased to 0.50 F_y.~~

~~**1810.6.4.1 Seismic reinforcement.** Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 1613, the following shall apply. Minimum reinforcement no less than 0.01 times the cross-sectional area of the pile concrete shall be provided in the top of the pile with a length equal to two times the required cap embedment anchorage into the pile cap, but not less than the tension development length of the reinforcement. The wall thickness of the steel pipe shall not be less than 3/16 inch (5 mm).~~

~~**1810.6.5 Placing concrete.** The placement of concrete shall conform to Section 1810.1.3, but is permitted to be chuted directly into smooth sided pipes and tubes without a centering funnel hopper.~~

~~**1810.7 Caisson piles.** Caisson piles shall conform to the requirements of Sections 1810.7.1 through 1810.7.6.~~

~~**1810.7.5 Allowable stresses.** The allowable design compressive stresses shall not exceed the following: concrete, 0.33 f'_c, steel pipe, 0.35 F_y and structural steel core, 0.50 F_y.~~

~~**1810.8.3 Allowable stresses.** The allowable compressive stress in the grout shall not exceed 0.33 f'_c. The allowable compressive stress in the steel pipe and steel reinforcement shall not exceed the lesser of 0.4 F_y, and 32,000 psi (220 Mpa). The allowable tensile stress in the steel reinforcement shall not exceed 0.60 F_y. The allowable tensile stress in the cement grout shall be zero.~~

~~SECTION 1811~~
~~COMPOSITE PILES~~

~~**1811.1 General.** Composite piles shall conform to the requirements of Sections 1811.2 through 1811.5.~~

~~**1811.2 Design.** Composite piles consisting of two or more approved pile types shall be designed to meet the conditions of installation.~~

~~**1811.3 Limitation of load.** The maximum allowable load shall be limited by the capacity of the weakest section incorporated in the pile.~~

~~**1811.4 Splices.** Splices between concrete and steel or wood sections shall be designed to prevent separation both before and after the concrete portion has set, and to ensure the alignment and transmission of the total pile load. Splices shall be designed to resist uplift caused by upheaval during driving of adjacent piles, and shall develop the full compressive strength and not less than 50 percent of the tension and bending strength of the weaker section.~~

~~**1811.5 Seismic reinforcement.** Where a structure is assigned to Seismic Design Category C, D, E or F in accordance with Section 1613, the following shall apply. Where concrete and steel are used as part of the pile assembly, the concrete reinforcement shall comply with that given in Sections 1810.1.2.1 and 1810.1.2.2 or the steel section shall comply with Section 1810.6.4.1.~~

~~SECTION 1812~~
~~PIER FOUNDATIONS~~

~~**1812.1 General.** Isolated and multiple piers used as foundations shall conform to the requirements of Sections 1812.2 through 1812.10, as well as the applicable provisions of Section 1808.2.~~

~~**1812.2 Lateral dimensions and height.** The minimum dimension of isolated piers used as foundations shall be 2 feet (610 mm), and the height shall not exceed 12 times the least horizontal dimension.~~

~~**1812.3 Materials.** Concrete shall have a 28-day specified compressive strength (f'_c) of not less than 2,500 psi (17.24 MPa). Where concrete is placed through a funnel hopper at the top of the pier, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 6 inches (152 mm). Where concrete is to be pumped, the mix design including slump shall be adjusted to produce a pumpable concrete.~~

~~**1812.5 Concrete placement.** Concrete shall be placed in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. Concrete shall not be placed through water except where a tremie or other approved method is used. When depositing concrete from the top of the pier, the concrete shall not be chuted directly into the pier but shall be poured in a rapid and continuous operation through a funnel hopper centered at the top of the pier.~~

~~**1812.8 Concrete.** Where adequate lateral support is not provided, and the unsupported height to least lateral dimension does not exceed three, piers of plain concrete shall be designed and constructed as pilasters in accordance with ACI 318. Where the unsupported height to least lateral dimension exceeds three, piers shall be constructed of reinforced concrete, and shall conform to the requirements for columns in ACI 318.~~

~~**Exception:** Where adequate lateral support is furnished by the surrounding materials as defined in Section 1808.2.9, piers are permitted to be constructed of plain or reinforced concrete. The requirements of ACI 318 for bearing on concrete shall apply.~~

~~**1812.9 Steel shell.** Where concrete piers are entirely encased with a circular steel shell, and the area of the shell steel is considered reinforcing steel, the steel shall be protected under the conditions specified in Section 1808.2.17. Horizontal joints in the shell shall be spliced to comply with Section 1808.2.7.~~

~~**1812.10 Dewatering.** Where piers are carried to depths below water level, the piers shall be constructed by a method that will provide accurate preparation and inspection of the bottom, and the depositing or construction of sound concrete or other masonry in the dry.~~

Committee Action: Approved as Modified

Modify proposal as follows:

1810.3.1.2 Composite elements. Where a single deep foundation element comprises two or more sections of different materials or different types spliced together, each section of the composite assembly shall satisfy the applicable requirements of this code, and the maximum allowable load in each section shall be limited by the structural capacity of ~~the weakest~~ that section.

TABLE 1810.3.2.6
ALLOWABLE STRESSES FOR MATERIALS USED IN DEEP FOUNDATION ELEMENTS

MATERIAL TYPE AND CONDITION	MAXIMUM ALLOWABLE STRESS [a]
1. Concrete or grout in compression [b]	
Cast-in-place with a permanent casing in accordance with Section 1810.3.2.7	$0.4\ f'_c$
Cast-in-place in a pipe, tube, ~~or~~ other permanent casing, or rock	$0.33\ f'_c$
Cast-in-place without a permanent casing	$0.3\ f'_c$
Precast nonprestressed	$0.33\ f'_c$
Precast prestressed	$0.33\ f'_c - 0.27\ f_{pc}$
2. Nonprestressed reinforcement in compression	$0.4\ f_y \leq 30{,}000$ psi
3. Structural steel in compression	
Cores within concrete-filled pipes or tubes	$0.5\ F_y \leq 32{,}000$ psi
Pipes, tubes, or H-piles, where justified in accordance with Section 1810.3.2.8	$0.5\ F_y \leq 32{,}000$ psi
Pipes or tubes for micropiles	$0.4\ F_y \leq 32{,}000$ psi
Other pipes, tubes, or H-piles	$0.35\ F_y \leq 16{,}000$ psi
~~5~~ 4. Nonprestressed reinforcement in tension	
Within micropiles	$0.6\ f_y$
Other conditions	$0.5\ f_y \leq 24{,}000$ psi
~~6~~ 5. Structural steel in tension	
Pipes, tubes, or H-piles, where justified in accordance with Section 1810.3.2.8	$0.5\ F_y \leq 32{,}000$ psi
Other pipes, tubes, or H-piles	$0.35\ F_y \leq 16{,}000$ psi
~~7~~ 6. Timber	In accordance with the AF&PA NDS

a. f'_c is the specified compressive strength of the concrete or grout; f_{pc} is the compressive stress on the gross concrete section due to effective prestress forces only; f_y is the specified yield strength of reinforcement; F_y is the specified minimum yield stress of structural steel.

b. The stresses specified apply to the gross cross-sectional area within the concrete surface. Where a temporary or permanent casing is used, the inside face of the casing shall be considered the concrete surface.

1810.3.2.7 Increased allowable compressive stress for cased cast-in-place elements. The allowable compressive stress in the concrete shall be permitted to be increased as specified in Table 1810.3.2.6 for those portions of permanently cased cast-in-place elements that satisfy all of the following conditions:

1. The design shall not use the casing to resist any portion of the axial load imposed.
2. The casing shall have a sealed tip and be mandrel driven.
3. The thickness of the casing shall not be less than manufacturer's standard gage No. 14 (0.068 inch) (1.75 mm).
4. The casing shall be seamless or provided with seams of strength equal to the basic material and be of a configuration that will provide confinement to the cast-in-place concrete.
5. The ratio of steel yield strength (F_y) to specified compressive strength (f'_c) shall not be less than six.
6. The nominal diameter of the element shall not be greater than 16 inches (406 mm).

1810.3.3.1 Allowable axial load. The allowable axial load on a deep foundation element shall be determined in accordance with Sections 1810.3.3.1.1 through 1810.3.3.1.8.

1810.3.3.1.4 Allowable frictional resistance. The assumed frictional resistance developed by any uncased cast-in-place deep foundation element shall not exceed one-sixth of the bearing value of the soil material at minimum depth as set forth in Table ~~1804.2~~ 1806.2, up to a maximum of 500 psf (24 kPa), unless a greater value is allowed by the building official on the basis of a soil investigation as specified in Section 1802 is submitted or a greater value is substantiated by a load test in accordance with Section 1810.3.3.1.2. Frictional resistance and bearing resistance shall not be assumed to act simultaneously unless recommended by a soil investigation as specified in Section 1802.

1810.3.5.2.3 Micropiles. Micropiles shall have an outside diameter of 12 inches (305 mm) or less. ~~There is no minimum diameter for micropiles.~~ The minimum diameter set forth elsewhere in Section 1810.3.5 shall not apply to micropiles.

1810.3.6 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 ~~of adequate strength~~ designed to ~~transmit the vertical and lateral loads~~ resist the axial and shear forces and moments occurring at the location of the splice during driving and for design load combinations ~~under service loading~~. 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.

Splices occurring in the upper 10 feet (3048 mm) of the embedded portion of an element shall be ~~capable of~~ designed to resist~~ing~~ at allowable ~~working~~ 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.

1810.3.10.1 [Supp] Construction. Micropiles shall develop their load-carrying capacity by means of a bond zone in soil, bedrock or a combination of soil and bedrock. Micropiles shall be grouted and have either a steel pipe or tube or steel reinforcement at every section along the length. It shall be permitted to transition from deformed reinforcing bars to steel pipe or tube reinforcement by extending the bars into the pipe or tube section by at least their tension development length in accordance with ACI 318.

1810.4.8 Hollow-stem augered, cast-in-place elements. Where concrete or grout is placed by pumping through a hollow-stem auger, the auger shall be permitted to rotate in a clockwise direction during withdrawal. As the auger is ~~shall be~~ withdrawn at a steady rate or in increments not to exceed 1 foot (305 mm), ~~in continuous increments.~~ concreting or grouting pumping pressures shall be measured and maintained high enough at all times to offset hydrostatic and lateral earth pressures. Concrete or grout volumes shall be measured to ensure that the volume of concrete or grout placed in each element is equal to or greater than the theoretical volume of the hole created by the auger. Where the installation process of any element is interrupted or a loss of concreting or grouting pressure occurs, the element shall be redrilled to 5 feet (1524 mm) below the elevation of the tip of the auger when the installation was interrupted or concrete or grout pressure was lost and reformed. Augered cast-in-place elements shall not be installed within six diameters center to center of an element filled with concrete or grout less than 12 hours old, unless approved by the building official. If the concrete or grout level in any completed element drops due to installation of an adjacent element, the element shall be replaced.

(Portions of proposal not shown remain unchanged)

Committee Reason: This proposal is part of the coordinated series of Chapter 18 reformatting code changes. It provides a necessary reorganization regarding deep foundations and fills in some holes in the current requirements. The modification provides correlation with section numbers resulting from the action taken on S147-07/08 and makes the terminology consistent throughout these sections. The modification of Section 1810.3.1.2 clarifies the assessment of composite deep foundation element. The modification to Section 1810.4.8 allows grout as well as concrete in hollow-stem augered elements.

Assembly Action: None

S161-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This code change adds an exception for the uplift capacity for wind and seismic effects that is consistent with current practice.

Assembly Action: **None**

S162-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:

1808.2.8.5 Uplift capacity. Where required by the design, the uplift capacity of a single pier or pile shall be determined by an approved method of analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D 3689. The maximum allowable uplift load shall not exceed the ultimate load capacity as determined in Section 1808.2.8.3, using the results of load tests conducted in accordance with ASTM D 3689, divided by a factor of safety of two. For pile groups subjected to uplift, the allowable working uplift load for the group shall be calculated by an approved method of analysis. Where the deep foundation elements in the group are placed at a center-to-center spacing of at least 2.5 times the least horizontal dimension of the largest single element, the allowable working uplift load for the group is permitted to be calculated as the lesser of:

1. The proposed individual pile uplift working load times the number of piles in the group.
2. Two-thirds of the effective weight of the pile group and the soil contained within a block defined by the perimeter of the group and the length of the pile.

Committee Reason: This proposal provides a limitation on the use of the provision to determine uplift capacity of a pile. The modification makes a clarification regarding the minimum pile spacing.

Assembly Action: **None**

S163-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This change gives the designer a necessary notification to consider the effects of mislocation of piles or piers. This will make designs conform to normal construction tolerances.

Assembly Action: **None**

S164-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This aligns the code provision to be more consistent with current practice for unbraced piles.

Assembly Action: **None**

S165-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This item clarifies the intent of the provision for unbraced piles. The consideration of pile fixity would be considered normal practice in evaluating lateral restraint.

Assembly Action: **None**

S166-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This proposal clarifies determination of pile lateral capacity by providing guidance with respect to the lateral movement criterion.

Assembly Action: **None**

S167-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This proposal appropriately replaces the term certify with a requirement to submit reports, since certify often means sealing by a register design professional under state law.

Assembly Action: **None**

S168-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This makes a necessary correction to the code requirements for deep foundations by relaxing an overly restrictive code requirement, because pile capacity is controlled by soil confinement rather than concrete spalling.

Assembly Action: **None**

S169-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This proposal corrects an oversight by expanding the range of permitted concrete slumps in drilled piers to match current practice.

Assembly Action: **None**

S170-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This code change updates requirements for drilled piers to match industry practice. The use of reinforcement rather than a structural core is widely acknowledged.

Assembly Action: **None**

S171-07/08

Committee Action: **Disapproved**

Committee Reason: The proposal to add helical pile foundations to Chapter 18 is a good concept and the committee has no objection in principle. The language used, such as references to ICC-ES evaluation reports, is an indicator that this change needs work. The committee recommends submitting a public comment with a modified proposal.

Assembly Action: **None**

S172-07/08

Errata: In item 3 correct new section numbers as follows:

~~**1904.2**~~ 1904.4 **Freezing and thawing exposures.** Concrete that will be exposed to freezing and thawing, in the presence of moisture, with or without deicing chemicals being present, ~~or other exposure conditions as defined below~~ shall comply with Sections ~~1904.2.1 through 1904.2.3~~ 1904.4.1 and 1904.4.2.

~~**1904.2.1**~~ 1904.4.1 **Air entrainment.** Concrete exposed to freezing and thawing ~~or deicing chemicals~~ while moist shall be air entrained in accordance with ACI 318, Section ~~4.2.1~~ 4.4.1:

In item 9 correct to read as follows:

~~**1908.1.4**~~ **1908.1.2 ACI 318,** Section ~~21.2.1~~ 21.1.1. Modify ACI 318 Sections ~~21.2.1.2, 21.2.1.3 and 21.2.1.4~~ 21.1.1.3 through 21.1.1.5, to read as follows:

Revise item 10 to read as follows:

21.1.1.3 – Structures assigned to SDC B shall comply with Chapters 1 through 19 and 22. For a structure assigned to SDC B using ordinary moment frames as part of the seismic-force resisting system, the provisions of 21.1.2 and 21.2 shall apply. For a structure assigned to SDC B and using intermediate or special systems, the applicable provisions of 21.1.3 through 21.1.7, and 21.3 through 21.10 shall also apply.

21.1.1.4 – Structures assigned to SDC C shall comply with Chapters 1 through 19, and the seismic-force-resisting system shall be intermediate or special moment frames, intermediate precast structural walls, or ordinary reinforced concrete or special structural walls. For a structure assigned to SDC C and using intermediate moment frames as part of the seismic-force-resisting system the provisions of 21.1.2 and 21.3 shall apply. For a structure assigned to SDC C and using special moment frames, or intermediate precast or special structural walls, the applicable provisions of 21.1.3 through 21.1.7, and 21.4 through 21.10 shall also apply. Any structure assigned to SDC C shall satisfy 21.1.8. *Except for footings, pedestals and basement walls in accordance with 22.10 or as permitted by the International Building Code, structural elements of plain concrete are prohibited.*

21.1.1.5 – Structures assigned to SDC D, E or F shall comply with Chapters 1 through 19, and the seismic-force-resisting system shall be special moment frames, intermediate precast structural walls, or special structural walls. For a structure assigned to SDC D, E, or F, the provisions of 21.1.2 through 21.1.8 and 21.4 through 21.13 shall apply. *Except for footings, pedestals and basement walls in accordance with 22.10 or as permitted by the International Building Code, structural elements of plain concrete are prohibited.*

In item 17 revise to read as follows:

1912.1 Scope. The provisions of this section shall govern the strength design of anchors installed in concrete for purposes of transmitting structural loads from one connected element to the other. Headed bolts, headed studs and hooked (J- or L-) bolts cast in concrete and expansion anchors and undercut anchors installed in hardened concrete shall be designed in accordance with Appendix D of ACI 318 as modified by Section ~~1908.1.16~~ 1908.1.9, provided they are within the scope of Appendix D.

> ~~**Exception**: Where the basic concrete breakout strength in tension of a single anchor, Nb, is determined in accordance with Equation (D-7), the concrete breakout strength requirements of Section D.4.2.2 shall be considered satisfied by the design procedures of Sections D.5.2 and D.6.2 for anchors exceeding 2 inches (51 mm) in diameter or 25 inches (635 mm) tensile embedment depth.~~

The strength design of anchors that are not within the scope of Appendix D of ACI 318, and as amended ~~above~~ in Section 1908.1.9, shall be in accordance with an approved procedure.

Add update to referenced standard as follows:

ACI
318-~~05~~08 Building Code Requirements for Structural Concrete

Committee Action: Approved as Modified

Modify proposal as follows:

1904.2 Exposure categories and classes. Concrete shall be assigned to exposure classes in accordance with ACI 318, Section 4.2 based on:

1. Exposure to freezing and thawing in a moist condition or deicer chemicals;
2. Exposure to sulfates in water or soil;
3. Exposure to water where the concrete is intended to have low permeability; and
4. Exposure to chlorides from deicing chemicals, salt, salt water, brackish water, seawater or spray from these sources, where the concrete has steel reinforcement.

SPECIAL STRUCTURAL WALL. A cast-in-place or precast wall complying with the requirements of ~~21.2.3 through 21.2.7, 21.10, and 21.11~~ 21.1.3 through 21.1.7, 21.9 and 21.10, as applicable, in addition to the requirements for ordinary reinforced concrete structural walls *or ordinary precast structural walls, as applicable. Where ASCE 7 refers to a "special reinforced concrete structural wall," it shall be deemed to mean a "special structural wall."*

1908.1.2 ACI 318, Section 21.1.1. Modify ACI 318 Sections 21.1.1.3 ~~through 21.1.1.5~~ and 21.1.1.7 to read as follows:

~~*21.1.1.3 – Structures assigned to SDC B shall comply with Chapters 1 through 19 and 22.* For a structure assigned to SDC B using ordinary moment frames as part of the seismic-force resisting system, the provisions of 21.1.2 and 21.2 shall apply. For a structure assigned to SDC B and using intermediate or special systems, the applicable provisions of 21.1.3 through 21.1.7, and 21.3 through 21.10 shall also apply.~~

21.1.1.3 – Structures assigned to Seismic Design Category A shall satisfy requirements of Chapters 1 to 19 and 22*; Chapter 21 does not apply*. Structures assigned to *Seismic Design Category* B, C, D, E, or F also shall satisfy 21.1.1.4 through 21.1.1.8, as applicable. *Except for structural elements of plain concrete complying with Section 1908.1.8 of the International Building Code, structural elements of plain concrete are prohibited in structures assigned to Seismic Design Category C, D, E or F.*

~~*21.1.1.4 – Structures assigned to SDC C shall comply with Chapters 1 through 19, and the seismic-force-resisting system shall be intermediate or special moment frames, intermediate precast structural walls, or ordinary reinforced concrete or special structural walls.* For a structure assigned to SDC C and using intermediate moment frames as part of the seismic-force resisting system the provisions of 21.1.2 and 21.3 shall apply. For a structure assigned to SDC C and using special moment frames, or intermediate precast or special structural walls, the applicable provisions of 21.1.3 through 21.1.7, and 21.4 through 21.10 shall also apply. Any structure assigned to SDC C shall satisfy 21.1.8. *Except for footings, pedestals and basement walls in accordance with 22.10 or as permitted by the International Building Code, structural elements of plain concrete are prohibited.*~~

~~*21.1.1.5 – Structures assigned to SDC D, E or F shall comply with Chapters 1 through 19, and the seismic-force-resisting system shall be special moment frames, intermediate precast structural walls, or special structural walls.* For a structure assigned to SDC D, E, or F, the provisions of 21.1.2 through 21.1.8 and 21.4 through 21.13 shall apply. *Except for footings, pedestals and basement walls in accordance with 22.10 or as permitted by the International Building Code, structural elements of plain concrete are prohibited.*~~

21.1.1.7 – Structural systems designated as part of the seismic-force-resisting system shall be restricted to those *permitted by ASCE 7*. Except for *Seismic Design Category* A, for which Chapter 21 does not apply, the following provision shall be satisfied for each structural system designated as part of the seismic-force-resisting system, regardless of the *Seismic Design Category*:

(a) Ordinary moment frames shall satisfy 21.2.
(b) Ordinary *reinforced concrete* structural walls *and ordinary precast structural walls* need not satisfy any provisions in Chapter 21.
(c) Intermediate moment frames shall satisfy 21.3.
(d) Intermediate precast *structural* walls shall satisfy 21.4.
(e) Special moment frames shall satisfy 21.5 through 21.8.
(f) Special structural wall shall satisfy 21.9.
Special structural walls *constructed using precast concrete* shall satisfy 21.10.
All special moment frames and special structural walls shall also satisfy 21.1.3 through 21.1.7.

1908.1.9 ACI 318, Section D.3.3. Modify ACI 318, Sections D.3.3.4 and D.3.3.5 to read as follows:

D.3.3.4 Anchors shall be designed to be governed by the steel strength of a ductile steel element as determined in accordance with D.5.1 and D.6.1, unless either D.3.3.5 or D3.3.6 is satisfied.

Exception: *Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.4.*

D.3.3.5 Instead of D.3.3.4, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a force level corresponding to anchor forces not greater that the design strength of anchor specified in D.3.3.3.

Exception: *Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.5.*

~~1908.1.9~~ 1908.1.10 ACI 318, Section D.4.2.2. Modify ACI 318, Section D.4.2.2 to read as follows:

D.4.2.2 – The concrete breakout strength requirements *for anchors* shall be considered satisfied by the design procedure of D.5.2 and D.6.2.

1912.1 Scope. The provisions of this section shall govern the strength design of anchors installed in concrete for purposes of transmitting structural loads from one connected element to the other. Headed bolts, headed studs and hooked (J- or L-) bolts cast in concrete and expansion anchors and undercut anchors installed in hardened concrete shall be designed in accordance with Appendix D of ACI 318 as modified by Section 1908.1.9 and 1908.10, provided they are within the scope of Appendix D.

The strength design of anchors that are not within the scope of Appendix D of ACI 318, and as amended in Section 1908.1.9 and 1908.1.10, shall be in accordance with an approved procedure.

(Portions of proposal not shown remain unchanged)

Committee Reason: This change updates the concrete provisions of the code to coordinate with the latest edition of the ACI 318 concrete standard. The modification updates section references to reflect the final published version of the 2008 edition of ACI 318. Similarly the modifications to Section 1908.1.2 reflect changes made after submittal of this proposal in the related portions of ACI 318 Chapter 21 based on public comments that ACI received. The modification also retains current section 1908.1.16, renumbered as 1908.1.9, in order to retain needed modifications that were added to the IBC in the 2006/2007 code cycle.

Assembly Action: None

S173-07/08

Committee Action: Approved as Modified

Modify proposal as follows:

1908.1.16 (Supp) ACI 318, Section D.3.3. Modify ACI 318, Sections D.3.3.2 through D.3.3.5 to read as follows:

D.3.3.2 - *In structures assigned to Seismic Design Category C, D, E or F,* post-installed anchors for use under D.2.3 shall have passed the Simulated Seismic Tests of ACI 355.2.

D.3.3.3 - *In structures assigned to Seismic Design Category C, D, E or F,* the design strength of anchors shall be taken as $0.75\Phi N_n$ and $0.75\Phi V_n$, where Φ is given in D.4.4 or D.4.5, and N_n and V_n are determined in accordance with D.4.1.

D.3.3.4 - *In structures assigned to Seismic Design Category C, D, E or F,* anchors shall be designed to be governed by tensile or shear strength of a ductile steel element, unless D.3.3.5 is satisfied.

Exceptions:

1. *Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.4.*
2. *~~Concrete wall anchorage designed to the forces of~~ <u>Wall anchors with design strengths equal to or greater than the force determined in accordance with</u> ASCE 7 Equation 12.11-1 <u>or 12.14-10</u> need not satisfy Section D.3.3.4.*

D.3.3.5 - Instead of D.3.3.4, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a load level corresponding to anchor forces not greater that the design strength of anchor specified in D.3.3.3, *or the minimum design strength of the anchor shall be at least 2.5 times the factored forces transmitted by the attachment*.

Exceptions:

1. *Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.5.*
2. *~~Concrete wall anchorage designed to the forces of~~ <u>Wall anchors with design strengths equal to or greater than the force determined in accordance with</u> ASCE 7 Equation 12.11-1 <u>or 12.14-10</u> need not satisfy Section D.3.3.5.*

Committee Reason: This proposal is necessary to improve concrete wall anchorage constructability and provide coordination with the overstrength load combinations. The modification adds a reference to the comparable requirement under the simplified procedure of ASCE 7. The modification also resolves a potential problem by making the exception applicable when the design force is equal to or greater than the value obtained by these equations.

Assembly Action: None

S174-07/08

Errata: Add updates to referenced standards as follows:

ACI
ACI 530-~~05~~<u>08</u> Building Code Requirements for Masonry Structures
ACI 530.1-~~05~~<u>08</u> Specification~~s~~ for Masonry Structures

ASCE/SEI
ASCE/SEI 5-~~05~~<u>08</u> Building Code Requirements for Masonry Structures
ASCE/SEI 6-~~05~~<u>08</u> Specification~~s~~ for Masonry Structures

TMS
TMS 402-~~05~~<u>08</u> Building Code Requirements for Masonry Structures
TMS 602-~~05~~<u>08</u> Specification for Masonry Structures

Committee Action: **Approved as Modified**

Modify proposal as follows:

2106.1 Seismic design requirements for masonry. Masonry structures and components shall comply with the requirements in Section ~~1.17.4.3.2~~ 1.17.2 and 1.17.3 and 1.17.4.1, 1.17.4.2, 1.17.4.3, 1.17.4.4, or 1.17.4.5 of TMS 402/ACI 530/ASCE 5 depending on the structure's seismic design category as determined in Section 1613. All masonry walls, unless isolated on three edges from in-plane motion of the basic structural systems, shall be considered to be part of the seismic-force-resisting system. In addition, the following requirements shall be met.

2106.1.1 Basic seismic-force-resisting system. Buildings relying on masonry shear walls as part of the basic seismic-force-resisting system shall comply with Section ~~1.17.4.3.2~~ 1.17.3.2 of TMS 402/ACI 530/ASCE 5 or with Section 2106.1.1.1, 2106.1.1.2 or 2106.1.1.3.

(Portions of proposal not shown remain unchanged)

Committee Reason: These revisions to the masonry code requirements are necessary to maintain consistency with the latest edition of the MSJC code and specification. The modification correlates section references with the published version of the standard.

Assembly Action: **None**

S175-07/08

Errata: Add updates to referenced standards as follows:

ACI
ACI 530-~~05~~08 Building Code Requirements for Masonry Structures
ACI 530.1-~~05~~08 Specification~~s~~ for Masonry Structures

ASCE/SEI
ASCE/SEI 5-~~05~~08 Building Code Requirements for Masonry Structures
ASCE/SEI 6-~~05~~08 Specification~~s~~ for Masonry Structures

TMS
TMS 402-~~05~~08 Building Code Requirements for Masonry Structures
TMS 602-~~05~~08 Specification for Masonry Structures

Committee Action: **Approved as Modified**

Modify proposal as follows:

2103.12 Grout. Grout shall comply with Article ~~2.1 C.1~~ 2.2 of TMS 602/ACI 530.1/ASCE 6.

(Portions of proposal not shown remain unchanged)

Committee Reason: These revisions to the masonry code requirements are necessary to maintain consistency with the latest edition of the MSJC code and specification and this approval is consistent with other actions concerning updates to the masonry provisions. This proposal appropriately maintains masonry QC requirements in the IBC. The modification correlates section references with the published version of the standard. The committee has a concern with item 8 in Section 2102.3, requiring the size and location of conduits, pipes and sleeves to be shown on the construction documents. Some believe this could have unintended consequences and hope the MSJC can address this item. The committee also indicated that should a conflict arise as a result of another masonry related code change, it is the committee's preference that this proposal should govern.

Assembly Action: **None**

S176-07/08

Errata: Add updates to referenced standards as follows:

ACI
ACI 530-~~05~~08 Building Code Requirements for Masonry Structures

ASCE/SEI
ASCE/SEI 5-~~05~~08 Building Code Requirements for Masonry Structures

TMS
TMS 402-~~05~~08 Building Code Requirements for Masonry Structures

Committee Action: **Approved as Submitted**

Committee Reason: These revisions to Section 2101 are necessary to maintain consistency with the latest edition of the MSJC code.

Assembly Action: **None**

S177-07/08

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standard ASTM C 1611/C 1611M-05 indicated that, in the opinion of ICC Staff, the standard did comply with ICC standards criteria.

Committee Action: **Approved as Submitted**

Committee Reason: In addition to coordinating portions of Section 2103 with the MSJC code, this proposal adds requirements for self-consolidating grout that include reference to the appropriate ASTM standard.

Assembly Action: **None**

S178-07/08

Errata: Add updates to referenced standards:

ACI
ACI 530.1-~~05~~08 Specification~~s~~ for Masonry Structures

ASCE/SEI
ASCE/SEI 6-~~05~~08 Specification~~s~~ for Masonry Structures

TMS
TMS 602-~~05~~08 Specification for Masonry Structures

Committee Action: **Approved as Modified**

Modify proposal as follows:

2103.8 Mortar. ~~Except for mortars listed in Sections 2103.9, 2103.10, and 2103.11,~~ Mortar for use in masonry construction shall conform to ASTM C 270 and Articles 2.1 and 2.6 A of TMS 602/ACI 530.1/ASCE 6, except for mortars listed in Sections 2103.9, 2103.10, and 2103.11.

Committee Reason: This proposal resolves conflicts between the IBC table and referenced standards by replacing the IBC requirement with a reference to the MSJC Specification. The modification rearranges the wording of Section 2103.8 that lists mortars not addressed by the MSJC Specification.

Assembly Action: **None**

S179-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This proposal corrects a footnote in Table 2103.8(2) to agree with the ASTM standard on which it is based.

Assembly Action: **None**

S180-07/08

Errata: Add updates to referenced standards as follows:

ACI
ACI 530.1-~~05~~08 Specification~~s~~ for Masonry Structures

ASCE/SEI
ASCE/SEI 6-~~05~~08 Specification~~s~~ for Masonry Structures

TMS
TMS 602-~~05~~08 Specification for Masonry Structures

Committee Action: **Approved as Submitted**

Committee Reason: This proposal adds the appropriate QA requirements for self-consolidating grout and is consistent with the action taken on S177-07/08.

Assembly Action: **None**

S181-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This change to "registered design professional" is an appropriate correction to the code terminology.

Assembly Action: **None**

S182-07/08

Errata: Add updates to referenced standards:

ACI
ACI 530.1-~~05~~08 Specification~~s~~ for Masonry Structures

ASCE/SEI
ASCE/SEI 6-~~05~~08 Specification~~s~~ for Masonry Structures

TMS
TMS 602-~~05~~08 Specification for Masonry Structures

Committee Action: **Approved as Modified**

Modify proposal as follows:

2104.1.2 Placing mortar and units. Placement of mortar and units ~~and mortar~~ shall comply with TMS 602/ACI 530.1/ASCE 6 and Section 2104.1.2.1.

(Portions of proposal not shown remain unchanged)

Committee Reason: This proposal will eliminate some confusion by replacing portions of Section 2104 with a direct reference to the MSJC specification. The requirements for open end units are retained in the IBC since they modify the corresponding MSJC provision. The modification is an editorial rewording of Section 2104.1.2.

Assembly Action: **None**

S183-07/08

Committee Action: **Approved as Submitted**

Committee Reason: These revised grouted masonry requirements provide consistency with the latest edition of the MSJC code and this approval is consistent with other actions concerning updates to the masonry provisions.

Assembly Action: **None**

S184-07/08

Errata: Add updates to referenced standards:

ACI
ACI 530.1-~~05~~08 Specification~~s~~ for Masonry Structures

ASCE/SEI
ASCE/SEI 6-~~05~~08 Specification~~s~~ for Masonry Structures

TMS
TMS 602-~~05~~08 Specification for Masonry Structures

Committee Action: **Approved as Submitted**

Committee Reason: This proposal will eliminate some confusion by replacing portions of Section 2104 on cold weather construction and cold weather construction with a direct reference to the MSJC specification.

Assembly Action: **None**

S185-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This code change provides clearer, more concise wording of the masonry QA requirements as well as consistency with the latest the MSJC specification.

Assembly Action: **None**

S186-07/08

Errata: Add updates to referenced standards as follows:

ACI
ACI 530-~~05~~08 Building Code Requirements for Masonry Structures

ASCE/SEI
ASCE/SEI 5-~~05~~08 Building Code Requirements for Masonry Structures

TMS
TMS 402-~~05~~08 Building Code Requirements for Masonry Structures

Committee Action: **Approved as Modified**

Modify proposal as follows:

2106.1 Seismic design requirements for masonry. Masonry structures and components shall comply with the requirements in Section 1.17 of TMS 402/ACI 530/ASCE 5 depending on the structure's seismic design category as determined in Section 1613. All masonry walls, unless isolated on three edges from in-plane motion of the basic structural systems, shall be considered to be part of the seismic-force-resisting system. ~~In addition, the following requirements shall be met.~~

(Portions of proposal not shown remain unchanged)

Committee Reason: This proposal will eliminate some confusion by replacing requirements in Section 2106 for seismic-force-resisting systems with a direct reference to the MSJC code. The modification is an editorial clarification.

Assembly Action: **None**

S187-07/08

Errata: Add updates to referenced standards as follows:

ACI
ACI 530-~~05~~08 Building Code Requirements for Masonry Structures

ASCE/SEI
ASCE/SEI 5-~~05~~08 Building Code Requirements for Masonry Structures

TMS
TMS 402-~~05~~08 Building Code Requirements for Masonry Structures

Committee Action: **Approved as Submitted**

Committee Reason: These revisions to the masonry code requirements are necessary to maintain consistency with the latest edition of the MSJC code.

Assembly Action: **None**

S188-07/08

Errata: Add updates to referenced standards as follows:

ACI
ACI 530-~~05~~08 Building Code Requirements for Masonry Structures

ASCE/SEI
ASCE/SEI 5-~~05~~08 Building Code Requirements for Masonry Structures

TMS
TMS 402-~~05~~08 Building Code Requirements for Masonry Structures

Committee Action: Approved as Submitted

Committee Reason: These revisions to the allowable stress design requirements align the building code with the latest edition of the MSJC code. IBC modifications to the standard that have been incorporated into the latest edition are now being removed from the IBC.

Assembly Action: None

S189-07/08

Errata: Add updates to referenced standards as follows:

ACI
ACI 530-~~05~~08 Building Code Requirements for Masonry Structures

ASCE/SEI
ASCE/SEI 5-~~05~~08 Building Code Requirements for Masonry Structures

TMS
TMS 402-~~05~~08 Building Code Requirements for Masonry Structures

Committee Action: Approved as Submitted

Committee Reason: These revisions to the strength design requirements align the building code with the latest edition of the MSJC code. An IBC modification to the standard that has been incorporated into the latest edition is now being removed from the IBC.

Assembly Action: None

S190-07/08

Committee Action: Approved as Submitted

Committee Reason: These revisions to empirical design of masonry maintain consistency with the latest edition of the MSJC code and this approval is consistent with other actions taken on updates to the masonry provisions. The requirements that are not contained in the MSJC are retained in the IBC. Redundant IBC provisions are replaced with references to the MSJC Code.

Assembly Action: None

S191-07/08

Committee Action: Approved as Submitted

Committee Reason: These revisions to the masonry code requirements are necessary to maintain consistency with the latest edition of the MSJC code.

Assembly Action: None

S192-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:

2110.3.3 Interior panels. For interior panels where the wind pressure does not exceed 10 psf (480 Pa), the maximum area of each individual standard-unit panel shall be 250 square feet (23.2 m^2) and the maximum area of each thin-unit panel shall be 150 square feet (13.9 m^2). The maximum dimension between structural supports shall be 25 feet (7620 mm) in width or 20 feet (6096 mm) in height. Where the wind pressure exceeds 10 psf (480 Pa), standard-unit panels shall be designed in accordance with Section 2110.3.1 and thin-unit panels shall be designed in accordance with Section 2110.3.2.

FIGURE 2110.3.1
GLASS UNIT MASONRY DESIGN WIND LOAD RESISTANCE

Committee Reason: These revisions to the glass unit masonry requirements provide correlation with the latest edition of the MSJC code. The modification clarifies that the provision applies to interior panels.

Assembly Action: **None**

S193-07/08

Committee Action: **Approved as Submitted**

Committee Reason: Masonry chimneys are the most vulnerable part of a residence in an earthquake. Extending the current requirements to include Seismic Design Category C is a prudent measure.

Assembly Action: **None**

S194-07/08

PART I – IBC STRUCTURAL
Committee Action: **Approved as Submitted**

Committee Reason: This proposal promotes better integrity for masonry fireplaces, by specifying minimum requirements for the fireplace smoke chamber.

Assembly Action: **None**

PART II – IRC B/E
Committee Action: **Approved as Submitted**

Committee Reason: This change adds parging to the smoke chamber. Also provides cover for cores in corbelled units.

Assembly Action: **None**

S195-07/08

PART I – IBC STRUCTURAL
Committee Action: **Disapproved**

Committee Reason: The proposed change in not necessary. The current wording of Section 2113.1 is preferred since it is consistent with the corresponding provision for masonry fireplaces.

Assembly Action: **None**

PART II – IRC B/E
Committee Action: **Approved as Submitted**

Committee Reason: This change provides language to be consistent with fireplaces and smoke chambers.

Assembly Action: **None**

S196-07/08

PART I – IBC STRUCTURAL
Committee Action: **Disapproved**

Committee Reason: This proposal was disapproved at the request of the proponent. There was no information provided to the committee to indicate that the current clearance to combustible trim is excessive.

Assembly Action: **None**

PART II – IRC B/E
Committee Action: **Disapproved**

Committee Reason: The proponent provided no test data for justification for reducing the chimney clearance from 12 inches to 8 inches.

Assembly Action: **None**

S197-07/08

PART I – IBC STRUCTURAL
Committee Action: **Disapproved**

Committee Reason: This proposal would introduce language that is not appropriate for the code. The term “rain cap” is vague and it is unclear what the phrase “net free area” refers to. The language needs to be more specific. Disapproval is consistent with the action of the IRC B/E committee.

Assembly Action: **None**

PART II – IRC B/E
Committee Action: **Disapproved**

Committee Reason: The proposal does not address all easily foreseeable circumstances. The net free area is not clearly defined.

Assembly Action: **None**

S198-07/08

PART I – IBC STRUCTURAL
Committee Action: **Approved as Submitted**

Committee Reason: This code change removes unnecessary wording. Equivalency is already allowed by Section 104.11 of the code.

Assembly Action: **None**

PART II – IRC B/E
Committee Action: **Approved as Submitted**

Committee Reason: This change deletes the redundant term "or equivalent". Section R104.11 of the code already permits alternate materials.

Assembly Action: **None**

S199-07/08

PART I – IBC STRUCTURAL
Committee Action: **Disapproved**

Committee Reason: This proposal would introduce an undefined term. The committee is unclear what “non-water soluble” would mean.

Assembly Action: **None**

PART II – IRC B/E
Committee Action: **Approved as Modified**

Modify proposal as follows:

R1003.12 Clay flue lining (installation). Clay flue liners shall be installed in accordance with ASTM C 1283 and extend from a point not less than 8 inches (203 mm) below the lowest inlet or, in the case of fireplaces, from the top of the smoke chamber to <u>a point above</u> ~~the top of~~ the enclosing walls. The lining shall be carried up vertically, with a maximum slope no greater than 30 degrees (0.52 rad) from the vertical.

Clay flue liners shall be laid in medium-duty non-water soluble refractory mortar conforming to ASTM C 199 with tight mortar joints left smooth on the inside and installed to maintain an air space or insulation not to exceed the thickness of the flue liner separating the flue liners from the interior face of the chimney masonry walls. Flue liners shall be supported on all sides. Only enough mortar shall be placed to make the joint and hold the liners in position.

Committee Reason: This change brings in new requirements from ASTM C 1283 and adds the qualifier that refractory mortar be non-water soluble. The modification was made to retain the point of termination of the clay flue liner.

Assembly Action: **None**

S200-07/08

Committee Action: **Disapproved**

Committee Reason: This item was disapproved at the request of the proponent.

Assembly Action: **None**

S201-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This change adopts the material-specific approach of the recognized material standard for identifying structural steel members.

Assembly Action: **None**

S202-07/08

Errata: Correct Section 2203.2 to read as follows:

2203.2 Protection. Painting of structural steel shall comply with the requirements contained in AISC 360. Individual structural members and assembled panels of cold-formed steel construction~~, except where fabricated of approved corrosion-resistant steel or of steel having a corrosion-resistant or other approved coating,~~ shall be protected against corrosion ~~with an approved coat of paint, enamel or other approved protection~~ <u>in accordance with the requirements contained in AISI S100. Protection of cold-formed steel light frame construction shall also comply with the requirements contained in AISI S200</u>.

Committee Action: **Approved as Submitted**

Committee Reason: This code change clarifies the identification and protection of cold-formed steel members by adding references to the applicable material standard.

Assembly Action: **None**

S203-07/08

Withdrawn by Proponent

S204-07/08

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standard SJI CJ-1.0-06 indicated that, in the opinion of ICC Staff, the standard did comply with ICC standards criteria.

Committee Action: Approved as Submitted

Committee Reason: It is appropriate to reference the new standard for composite steel joists. This proposal also clarifies the protection requirements for open web steel joist and joist girders by referring directly to the applicable material standards.

Assembly Action: None

S205-07/08

Committee Action: Approved as Modified

Modify proposal as follows:

2208.1 Storage racks. The design, testing and utilization of industrial steel storage racks, made of cold-formed or hot-rolled steel structural members, shall be in accordance with the RMI/ANSI MH 16.1. ~~Racks in the scope of this specification include industrial pallet racks, movable shelf racks and stacker racks made of cold-formed or hot-rolled steel structural members. Such rack types also include push-back rack, pallet-flow rack, case-flow rack pick modules, rack supported platforms, and the storage rack portion of any rack structure that acts as support for the exterior walls and roof, except as noted; and not other types of racks, such as drive-in and drive-through racks, cantilever racks, or portable racks.~~ Where required by ASCE 7, the seismic design of storage racks shall be in accordance with the provisions of Section 15.5.3 of ASCE 7, except that items (1), (2), and (3) of Section 15.5.3 of ASCE 7 do not apply when the rack design satisfies RMI/ANSI MH 16.1.

Rack Manufacturing Institute

RMI/ANSI MH 16.1-~~07~~ 08	Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks

Committee Reason: The proposal to refer to the latest version of the standard provides clarifications that address many problems in the design of racks. The modification removes redundant language regarding the scope of the standard. In addition it corrects the date of the standard to 2008 and clarifies the applicability of ASCE 7 Section 15.5.3.

Assembly Action: None

S206-07/08

Committee Action: Approved as Submitted

Committee Reason: This change is consistent with the standard update approved in S238-07/08. The change in referring to AISI S100 is editorial correction to reflect the new designation of the AISI standard.

Assembly Action: None

S207-07/08

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standards SDI C1.0-06, NC1.0-06 and RD1.0-06 indicated that, in the opinion of ICC Staff, the standards did comply with ICC standards criteria.

Committee Action: Disapproved

Committee Reason: There are some technical issues with the steel deck standards, regarding the use of fibers to substitute for steel reinforcement required by ACI 318 that should be resolved. This was evidenced by testimony on proposed modifications to both the composite steel floor deck and non-composite floor deck standards.

Assembly Action: None

S208-07/08

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standard AISI S210-07 indicated that, in the opinion of ICC Staff, the standard did comply with ICC standards criteria.

Committee Action: Approved as Modified

Modify proposal as follows:

2210.6 Lateral design. Light-framed shear walls, diagonal strap bracing that is part of a structural wall and diaphragms <u>used</u> to resist wind, seismic and other in-plane lateral loads shall be designed in accordance with AISI S213.

2210.7 Prescriptive framing. Detached one- and two-family dwellings and townhouses, less than or equal to three stories ~~in height~~ <u>above grade plane</u>, shall be permitted to be constructed in accordance with AISI S230 subject to the limitations therein.

(Portions of proposal not shown remain unchanged)

Committee Reason: This code change correlates the IBC section on cold-formed steel with the latest editions of AISI standards and adds the new standard for floor and roof framing. The modification makes editorial changes, providing terminology that is consistent with other code provision.

Assembly Action: None

S209-07/08

Committee Action: Approved as Submitted

Committee Reason: The addition of requirements for truss design drawings and submittals is an important clarification for cold-formed steel that is similar to the current requirements for wood trusses.

Assembly Action: None

S210-07/08

Committee Action: Disapproved

Committee Reason: Disapproval was requested by the proponent.

Assembly Action: None

S211-07/08

Committee Action: Disapproved

Committee Reason: This proposal would unnecessarily duplicate requirements that are currently part of Chapter 22. Also the definition of “metal building system” is confusing.

Assembly Action: None

S212-07/08

Committee Action: Disapproved

Committee Reason: Disapproval was requested by the proponent.

Assembly Action: None

S213-07/08

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standard ASTM D 7032-07 indicated that, in the opinion of ICC Staff, the standard did comply with ICC standards criteria.

Committee Action: **Disapproved**

Committee Reason: The proposed definition of "wood plastic composite" is overly broad which could lead to misapplication. Another concern is that there is no design method provided for these materials.

Assembly Action: **None**

S214-07/08

PART I – IBC STRUCTURAL
Committee Action: **Disapproved**

Committee Reason: The intent to clarify the methods of fire retardant treatment may be good, but the proposed wording is not appropriate code language. Also enforcers would need some way to identify complying products in the field.

Assembly Action: **None**

PART II – IRC B/E
Committee Action: **Disapproved**

Committee Reason: The term "considerable" is ambiguous and unenforceable. There is no definition of the term "other means".

Assembly Action: **None**

S215-07/08

PART I – IBC STRUCTURAL
Committee Action: **Disapproved**

Committee Reason: Disapproval was requested by the proponent.

Assembly Action: **None**

PART II – IRC B/E
Committee Action: **Disapproved**

Committee Reason: This proposal needs more work, especially in the area of who is responsible for the entire building roof system. The proponent should rework and include definitions.

Assembly Action: **None**

S216-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:

2303.4.1.1 (Supp)Truss design drawings. The written, graphic and pictorial depiction of each individual truss shall be provided to the building official for approval prior to installation. Truss design drawings shall also be provided with the shipment of trusses delivered to the job site. Truss design drawings shall include, at a minimum, the information specified below:

1. Slope or depth, span and spacing;
2. Location of all joints and support locations;
3. Number of plies if greater than one;
4. Required bearing widths;

5. Design loads as applicable, including;
    5.1. Top chord live load ~~(for roof trusses, this shall be the controlling case of live load or snow load~~
    5.2. Top chord dead load;
    5.3. Bottom chord live load;
    5.4. Bottom chord dead load;
    5.5. Additional loads and locations
    5.6. Environmental design criteria and loads (wind, rain, snow, seismic, etc.)
6. Other lateral loads, including drag strut loads.
7. Adjustments to wood member and metal connector plate design value for conditions of use;
8. Maximum reaction force and direction, including maximum uplift reaction forces where applicable;
9. Metal connector plate type, size, and thickness or gage, and the dimensioned location of each metal connector plate except where symmetrically located relative to the joint interface;
10. Size, species and grade for each wood member;
11. Truss-to-truss connections and truss field assembly requirements
12. Calculated span to deflection ratio and maximum vertical and horizontal deflection for live and total load as applicable;
13. Maximum axial tension and compression forces in the truss members; and
14. Required permanent individual truss member restraint location and the method and details of restraint/bracing to be used per Section 2303.4.1.2.

(Portions of proposal not shown remain unchanged)

Committee Reason: This code change reformats and rewords the current wood truss design requirements and is largely editorial. This will benefit enforcement. The modifications clarify the required information regarding environmental loads.

Assembly Action: None

S217-07/08

Committee Action: Approved as Submitted

Committee Reason: This proposal removes an unnecessary code provision and it is consistent with the prior action associated with wood truss design requirements.

Assembly Action: None

S218-07/08

Committee Action: Approved as Modified

Modify proposal as follows:

2304.9.5.1 (Supp) Fasteners for preservative-treated wood. Fasteners for preservative-treated wood shall be of hot-dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper. Fasteners other than nails, and timber rivets, wood screws and lag screws shall be permitted to be of mechanically deposited zinc-coated steel with coating weights in accordance with ASTM B 695, Class 55 minimum.

> **Exception:** Plain carbon steel fasteners in SBX/DOT and zinc borate preservative-treated wood in an interior, dry environment shall be permitted ~~need not be hot dipped galvanized~~.

Committee Reason: This proposal allows an additional option for fasteners in limited locations. The modification rewords the exception to better state the proponent's intent.

Assembly Action: None

S219-07/08

PART I – IBC STRUCTURAL
Committee Action: Disapproved

Committee Reason: The committee does not feel that this proposal is ready for the code. There was a particular concern over the appropriate relationship between a manufacturer's recommendations and the referenced ASTM standard.

Assembly Action: None

PART II – IRC B/E
Committee Action: Approved as Modified

Modify the proposal as follows:

R319.3 (Supp) Fasteners and connectors in contact with preservative-treated and fire-retardant-treated wood. Fasteners and connectors in contact with preservative-treated wood and fire-retardant-treated wood shall be in accordance with this section. The coating weights for zinc-coated fasteners shall be in accordance with ASTM A 153.

R319.3.1 (Supp) Fasteners for preservative treated wood. Fasteners for preservative-treated wood shall be of hot dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper. <u>Coating types and weights for</u> connectors in contact with preservative-treated wood ~~that are used in exterior applications~~ shall be in accordance with the connector manufacturer's recommendations. In the absence of manufacturer's recommendations a minimum of ASTM A 653 type G185 zinc-coated galvanized steel, or equivalent, shall be used.

Exceptions:

1. One-half-inch (12.7 mm) diameter or greater steel bolts.
2. Fasteners other than nails, timber rivets, wood screws and lag screws shall be permitted to be of mechanically deposited zinc coated steel with coating weights in accordance with ASTM B 695, Class 55 minimum.

R319.3.2 (Supp) Fastenings for wood foundations. Fastenings for wood foundations shall be as required in AF&PA Technical Report No. 7.

R319.3.3 (Supp) Fasteners for fire-retardant-treated wood used in exterior applications or wet or damp locations. Fasteners for fire-retardant-treated wood used in exterior applications or wet or damp locations shall be of hot dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper.

R319.3.4 (Supp) Fasteners for fire-retardant-treated wood used in interior applications. Fasteners for fire-retardant treated wood used in interior locations shall be in accordance with the manufacturer's recommendations. In the absence of the manufacturer's recommendations, Section R319.3.3 shall apply.

~~**R319.3.5 Fasteners and connectors subject to exposure and located within 300 feet of the shoreline**. Fasteners and connectors used in exterior application that are located within 300 feet (91440mm) of the shoreline shall be stainless steel grade 304 or 316 or an approved alternative.~~

Committee Reason: The proposal as modified serves to clarify the minimum coating requirements for fasteners and connectors in contact with preservative treated wood. By requiring the proper coating for fasteners and connectors the usable expectant life will be extended. The committee felt the modification was necessary to eliminate the restrictive language that specifically addressed fasteners and connectors used in exterior locations and subject to exposure within 300 feet of a shoreline as it was too restrictive.

Assembly Action: None

S220-07/08

Committee Action: Disapproved

Committee Reason: A similar proposal in the last code development cycle was disapproved and thus far no additional technical data has been provided to justify allowing wood screws and lag screws with mechanical galvanizing.

Assembly Action: None

S221-07/08

Committee Action: Approved as Submitted

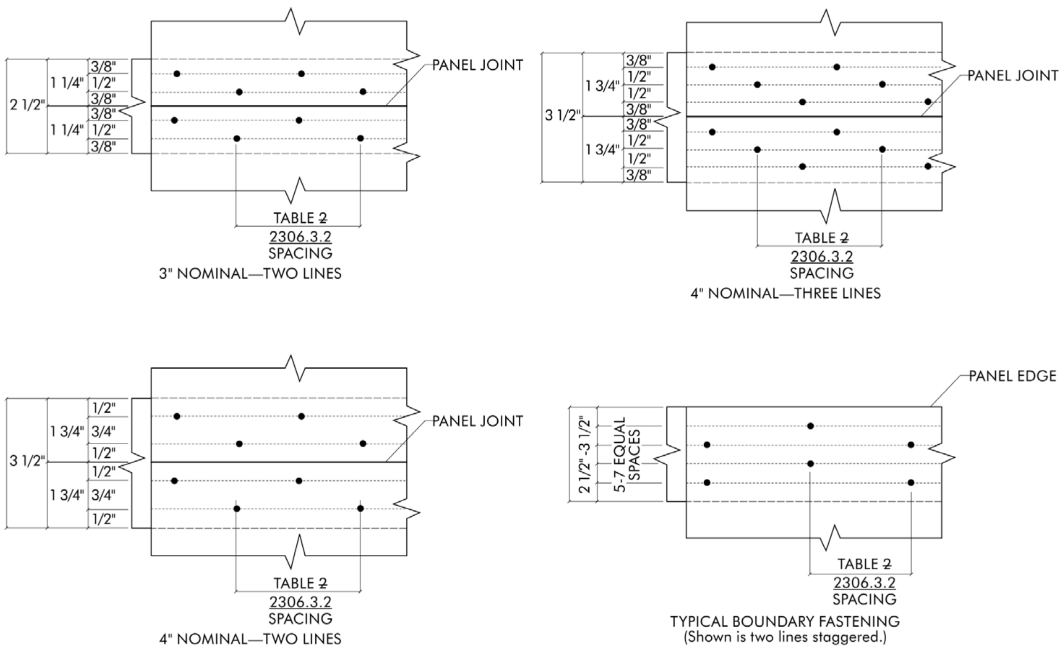
Committee Reason: The reference to floor or roof construction is a more succinct statement that is preferable to using the term "assembly".

Assembly Action: None

S222-07/08

Committee Action: **Approved as Modified**

Modify proposal as follows:



NOTE: SPACE PANEL END AND EDGE JOINT 1/8-INCH. REDUCE SPACING BETWEEN LINES OF NAILS AS NECESSARY TO MAINTAIN MINIMUM 3/8-INCH FASTENER EDGE MARGINS, MINIMUM SPACING BETWEEN LINES IS 3/8-INCH

Committee Reason: This proposal clarifies the code by adding an illustration of nail spacing and staggering to the diaphragm load table. The modification corrects typos in the figure submitted with the code change.

Assembly Action: **None**

S223-07/08

Committee Action: **Disapproved**

Committee Reason: The prescriptive construction provisions should not apply in hurricane prone regions, therefore the current wording of the exception is preferred. It is not appropriate to compare the IBC with the IRC in this respect.

Assembly Action: **None**

S224-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This is a needed clarification of the connection requirements along braced wall lines.

Assembly Action: **None**

S225-07/08

Committee Action: Disapproved

Committee Reason: The intent to clarify requirements for connecting braced wall panels to diaphragms, is a good concept, but the committee believes this proposal is not ready to be added to the code, due to technical problems with the diagrams and proposed wording.

Assembly Action: None

S226-07/08

Committee Action: Disapproved

Committee Reason: The proposal lacks foundation and it may lead to inappropriate application of the code requirement for the lateral support of wood floor and roof framing.

Assembly Action: None

S227-07/08

PART I – IBC STRUCTURAL
Committee Action: Approved as Modified

Modify proposal as follows:

2308.9.1 Size, height and spacing. The size, height and spacing of studs shall be in accordance with Table 2308.9.1 except that utility-grade studs shall not be spaced more than 16 inches (406 mm) o.c., or support more than a roof and ceiling, or exceed 8 feet (2438 mm) in height for exterior walls and load-bearing walls or 10 feet (3048 mm) for interior nonload-bearing walls. Studs shall be continuous from a support at the bottom to a support at the top to resist loads perpendicular to wall. The support shall be a foundation or floor, ceiling or roof diaphragm or shall be designed in accordance with accepted engineering practice.

> **Exception:** Jack studs, trimmer studs and cripple studs at openings in walls that comply with Table 2308.9.5.

Committee Reason: This code change clarifies the intent of the code by requiring studs to be continuous which eliminates possible hinges in walls. The modification adds an appropriate exception which addresses conditions at wall openings.

Assembly Action: None

PART II – IRC B/E
Committee Action: Disapproved

Committee Reason: This proposal would permit an unlimited height stud at maximum stud spacing from a floor to a three story ceiling. This would also permit stacking of panelized construction.

Assembly Action: None

S228-07/08

Committee Action: Disapproved

Committee Reason: The intent of clarifying the stud spacing and orientation in nonbearing walls is good, but there was some difficulty following the proponent's reason. Also the justification for the 16 inch spacing in the exception was not clear.

Assembly Action: None

S229-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This proposal corrects an obvious flaw in the wording of the exception for stone and masonry veneer.

Assembly Action: **None**

S230-07/08

Committee Action: **Approved as Submitted**

Committee Reason: This code change furthers the elimination of wind-borne debris in hurricane regions, by adding requirements for glass in-fill panels used on the exterior in wind-borne debris regions.

Assembly Action: **None**

S231-07/08

Committee Action: **Approved as Submitted**

Committee Reason: Agreement with the proponent's reason which indicates that some confusion over glass requirements can be eliminated by including a more complete set of requirements for elevator cars and hoistways in Section 2409.

Assembly Action: **None**

S232-07/08

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standard ASTM C 1658/C 1658M-06 indicated that, in the opinion of ICC Staff, the standard did comply with ICC standards criteria.

PART I – IBC STRUCTURAL
Committee Action: **Approved as Submitted**

Committee Reason: This code change adds material requirements in the form of a referenced standard that is appropriate with the wider use of glass mat gypsum panel products.

Assembly Action: **None**

PART II – IRC B/E
Committee Action: **Approved as Submitted**

Committee Reason: This change adds the appropriate ASTM Standard for glass mat gypsum panel for interior use.

Assembly Action: **None**

S233-07/08

Note: The following analysis was not in the Code Change Proposal book but was posted on the ICC website.

Analysis: Review of proposed new standards ASTM D 3270-00(2005) and D 3274-95(2002) indicated that, in the opinion of ICC Staff, the standards did not comply with ICC standards criteria, Section 3.6.2.1.

Committee Action: **Disapproved**

Committee Reason: The committee agrees that mold-resistance is a legitimate concern, but this proposal has too many flaws as written. The proposed standards are not compliant with requirements for referenced standards. Section 2509 may not be the appropriate location for introducing mold-resistance requirements ionto the code.

Assembly Action: None

S234-07/08

Committee Action: Disapproved

Committee Reason: There was insufficient justification provided in support of this proposal. It does not seem reasonable to apply this termite protection provision to all building types. The proposed language lacks enforceability. It requires use of any one of six methods, or combinations, which implies the methods are all roughly equivalent which does not appear to be correct.

Assembly Action: None

S235-07/08

Committee Action: Approved as Submitted

Committee Reason: This proposal adds a supplement to the currently referenced edition of ASCE 7, which restores prior minimum base shear requirements for earthquake design.

Assembly Action: None

S236-07/08

Committee Action: Disapproved

Committee Reason: Disapproved at the proponent’s request, because the updated standard is not complete. The standard is needed and the committee is hopeful that a public comment will be submitted.

Assembly Action: None

S237-07/08

PART I – IBC STRUCTURAL
Committee Action: Disapproved

Committee Reason: Disapproved at the proponent’s request, because the updated standard is not complete. The standard is needed and the committee is hopeful that a public comment will be submitted.

Assembly Action: None

PART II – IRC B/E
Committee Action: Disapproved

Committee Reason: Based upon the proponent's request. The standard is not ready. The proponent will bring back as a public comment if the standard is ready.

Assembly Action: None

S238-07/08

Delete reference to TMS standards as follows:

~~**TMS**~~	~~The Masonry Society~~ ~~3970 Broadway, Unit 201-D~~ ~~Boulder, CO 80304-1135~~
~~Standard reference number~~	~~Title~~
~~402—~~ 08 ~~05~~	~~Building Code Requirements for Masonry Structures~~
~~602—~~ 08 ~~05~~	~~Specification for Masonry Structures~~

Errata: Revise TIA standard update to read as follows:

222-G—2005 Structural Standard for Antenna Supporting Structures and Antennas, including Addendum 1, 222-G-1 dated 2007.

Committee Action: Approved as Submitted

Committee Reason: Approval of this code change assures that the IBC will reflect the latest updates from these standards developing organizations.

Assembly Action: None