AAMA/WDMA/CSA 101/I.S.2/A440 does require testing in accordance with ASTM E330 and measurement of deflection. However, AAMA/WDMA/CSA 101/I.S.2/A440 only places a limit on the frame and sash deflection for heavy commercial (HC) and architectural products (AW), and has no requirement on deflection for residential (R), light commercial (LC), and commercial (C) products. Excessive deflection of the frame or sash can have an adverse effect on stress in the glass and could result in glass breakage at or below design loads creating a safety concern. The single ASTM E330 load test required in AAMA/WDMA/CSA 101/I.S.2/A440 is not statistically significant in ensuring that the stress does not increase the probability of breakage beyond the industry standard of eight lites per thousand when the deflection limitation of 1/175 is exceeded. While this deflection exemption remains in the IRC for residential buildings, it is inappropriate to have an exemption for these products when used in more diverse and larger buildings built to the IBC. This proposal would ensure that an appropriate limit on frame deflection is placed on fenestration products from all performance classes. Because the deflection is already being measured for all these products (but not limited for R, LC, and C classes), there is no cost impact except for products which do not comply with this more conservative and appropriate requirement.

Cost Impact: The code change proposal will not increase the cost of construction.

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

Individual Consideration Agenda

This item is on the agenda for individual consideration because public comments were submitted.

Public Comment 1:

Thomas D. Culp, Birch Point Consulting LLC, representing Aluminum Extruders Council, requests Approval as Modified by this Public Comment.

Modify proposal as follows:

1714.5.1 Exterior windows and doors. Exterior windows and sliding doors shall be tested and labeled as conforming to AAMA/WDMA/CSA 101/I.S.2/A440. The label shall state the name of the manufacturer, the approved labeling agency, and the product designation as specified in AAMA/WDMA/CSA101/I.S.2/A440. Exterior side hinged doors shall be tested and labeled as conforming to AAMA/WDMA/CSA 101/I.S.2/A440 or comply with Section 1714.5.2. <u>Products in buildings of Group R not more than three stories above grade plane tested and labeled as conforming to AAMA/WDMA/CSA 101/I.S.2/A440 shall not be subject to the requirements of Sections 2403.2 and 2403.3.</u>

Commenter's Reason: Chapter 24 and ASTM E1300 require that glazing be firmly supported to prevent breakage under the design load by establishing maximum framing deflection limits. However, certain products are currently and inappropriately exempted from this requirement if they are labeled to the AAMA/WDMA/CSA 101/I.S.2/A440 standard. This proposal would remove that exemption to restore an appropriate safety margin of less than 8 in 1000 probability of glass breakage, consistent with ASTM E1300.

This proposal only applies to the IBC, and does not affect lighter products used when building to the IRC. However, the committee correctly pointed out that the IBC is also used for lowrise residential buildings, including both detached homes and apartments. This modification addresses the committee's concern by reinstating the exemption for lowrise residential buildings, but maintaining the structural deflection limit requirements for products in all other applications, as the top priority should be restoring a safety margin consistent with what is already in Chapter 24 and ASTM E1300.

Finally, the committee reason for disapproving the initial proposal stated that there is too much uncertainty to remove the exemption at this time. As the exemption increases the probability of glass breakage, we believe safety concerns would dictate that any exemption SHOULD be removed until any uncertainty is resolved.

Public Comment 2:

William E. Koffel, Koffel Associates, Inc., representing Glazing Industry Code Committee, requests Approval as Modified by this Public Comment.

Modify proposal as follows:

1714.5.1 Exterior windows and doors. Exterior windows and sliding doors shall be tested and labeled as conforming to AAMA/WDMA/CSA 101/I.S.2/A440. The label shall state the name of the manufacturer, the approved labeling agency, and the product designation as specified in AAMA/WDMA/CSA101/I.S.2/A440. Exterior side hinged doors shall be tested and labeled as conforming to AAMA/WDMA/CSA 101/I.S.2/A440 or comply with Section 1714.5.2. <u>Products in buildings of Group R not more than three stories in grade plane tested and labeled as conforming to AAMA/WDMA/CSA 101/I.S.2/A440 shall not be subject to the requirements of Sections 2403.2 and 2403.3.</u>

Commenter's Reason: The purpose of this proposal is to remove the exemption that fenestration products labeled to AAMA/WDMA/CSA 101/I.S.2/A440 do not have to meet the requirements of sections 2403.2 and 2403.3, which ensure safe performance through proper support of glass. Specifically, section 2403.3 requires that the deflection of framing members supporting glass may not exceed 1/175 of the glass edge length (or ¾ inch, whichever is less) when subjected to the design load. Chapter 24 of the IBC relies on glass design curves that are contained in ASTM E 1300. This ASTM standard recognizes the importance of limiting edge deflection of the glass and also recommends a limitation of 1/175 of the glass edge length. Prior to the IBC, the legacy codes required deflection limitations of 1/175 of the span for glass holding members. It was not until the IBC was published that this exemption was allowed.

AAMA/WDMA/CSA 101/I.S.2/A440 does require testing in accordance with ASTM E330 and measurement of deflection. However, AAMA/WDMA/CSA 101/I.S.2/A440 only places a limit on the frame and sash deflection for heavy commercial (HC) and architectural products (AW), and has no requirement on deflection for residential (R), light commercial (LC), and commercial (C) products. Excessive deflection of the frame or sash can have an adverse effect on stress in the glass and could result in glass breakage at or below design loads creating a safety concern. The single ASTM E330 load test required in AAMA/WDMA/CSA 101/I.S.2/A440 is not statistically significant in ensuring that the stress does not increase the probability of breakage beyond the industry standard of eight lites per thousand when the deflection limitation of 1/175 is exceeded. While this deflection exemption remains in the IRC for residential buildings, it is inappropriate to have an exemption for these products when used in more diverse and larger buildings built to the IBC. This proposal would ensure that an appropriate limit on frame deflection is placed on fenestration products from all performance classes. Because the deflection is already being measured for all these products (but not limited for R, LC, and C classes), there is no cost impact except for products which do not comply with this more conservative and appropriate requirement.

The revision proposed in the Public Comment addresses the Committee's concern regarding the impact on low-rise residential buildings.

Final Action: AS AM AMPC____ D

S143-07/08
1714.5.2, Chapter 35 (New)

Proposed Change as Submitted:

Proponent: John Woestman, The Kellen Company, representing Door Safety Council

1. Revise as follows:

1714.5.2 (Supp) Exterior windows and door assemblies not provided for in Section 1714.5.1. Exterior window and door assemblies shall be tested in accordance with ASTM E 330. <u>Exterior side-hinged door assemblies shall be permitted to be tested in accordance with ANSI/SDI A250.13.</u> Structural performance of garage doors shall be determined in accordance with either ASTM E 330 or ANSI/DASMA 108, and shall meet the acceptance criteria of ANSI/DASMA 108. Exterior window and door assemblies containing glass shall comply with Section 2403. The design pressure for testing shall be calculated in accordance with Chapter 16. Each assembly shall be tested for <u>a minimum of</u> 10 seconds at a load equal to 1.5 times the design pressure.

2. Add standard to Chapter 35 as follows:

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

Reason: This proposed change allows an alternative method to demonstrate structural performance for side-hinged doors by requiring doors to be tested per ANSI/SDI A250.13-XX. A250.13-XX, which is under development to update A250.13-03, will contain language that prescribes how door components are to be selected to create door assemblies expected to perform equivalently to a door assembly tested to ASTM E 330.

This proposal helps resolve performance and code compliance issues when doors are assembled from components from multiple sources and include interchangeable elements.

Through the ASTM standards development process, members of the Steel Door Institute (SDI) and members of the Builders Hardware Manufacturers' Association (BHMA) developed a national standard for a component-based approach to testing for windstorm resistance of swinging door assemblies. The test procedures used in this standard represent the most severe requirements found in the windstorm resistance standards in use in building codes. However, the procedures are designed to isolate, as much as possible, the loads and conditions that a particular component is subjected to in the full assembly test and duplicate these specific conditions. Using a combination of worst-case assembly design and safety factors, this testing was designed to provide a component rating that related directly to the component's ability to withstand the conditions that occur in a full assembly test.

Prior to releasing the current ANSI/SDI A250.13 standard, the BHMA/SDI task group conducted validation testing where components were expected to be rated at three design-load target values. Those components were tested to establish their ratings by the proposed procedure. The results of this process confirmed that assemblies made up of rated components would perform as expected. In addition, the validation test showed that where a component was identified as the weakest element of an assembly, based on the component tests, the same component would fail in a similar manner when tested as part of an assembly to levels exceeding the component's rated capacity.

Building designers will use performance criteria of door components, per ANSI/SDI A250.13, to select appropriate components to create door assemblies by conducting an opening-by-opening design analysis, specify components, verify code compliance, and submit the results through the normal plans review process. Code Authorities will thus need only to verify that the design load and compliance analysis has been correctly carried out and that the specified components are actually installed during construction in accordance with the manufacturer's instructions and project specifications.

Cost Impact: The code change proposal will not increase the cost of construction.

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**

Individual Consideration Agenda

This item is on the agenda for individual consideration because a public comment was submitted.

Public Comment:

John Woestman, The Kellen Company, representing Door Safety Council (DSC), requests Approval as Modified by this Public Comment.

Modify proposal as follows:

1714.5.2 (Supp) Exterior windows and door assemblies not provided for in Section 1714.5.1. Exterior window and door assemblies shall be tested in accordance with ASTM E 330. Structural performance of exterior ~~Exterior~~ side-hinged door assemblies shall be ~~permitted to be tested~~ determined in accordance with either ASTM E330 or ANSI/SDI A250.13. Structural performance of garage doors shall be determined in accordance with either ASTM E 330 or ANSI/DASMA 108, and shall meet the acceptance criteria of ANSI/DASMA 108. Exterior window and door assemblies containing glass shall comply with Section 2403. The design pressure for testing shall be calculated in accordance with Chapter 16. Each assembly shall be tested for a minimum of 10 seconds at a load equal to 1.5 times the design pressure.

Steel Door Institute
ANSI/SDI A250.13-~~XX~~ 08 Testing and Rating of Severe Windstorm Resistant Components for Swinging Door Assemblies.

Commenter's Reason: The committee's reason for recommending disapproval, as requested by the proponent, was that the newest edition of ANSI/SDI A250.13 was not yet completed at the time of the committee hearings. Subsequent to the committee hearings, ANSI/SDI A250.13-2008 was approved May 21, 2008 with a publication date of May 30, 2008.

The modifications proposed by this public comment

1. Revise the 2nd sentence of 1714.5.2 to be consistent with the current language of the 3rd sentence of this section.
2. Revise the reference to the approved ANSI/SDI A250.13 standard.

The updated A250.13 standard incorporates requirements for testing swinging door components (i.e. side-hinge door components) for structural performance. This updated standard also incorporates requirements for selecting the tested components to create a swinging door assembly (side-hinged door assembly) which can be expected to perform equivalently (or better) than a door assembly tested to ASTM E330.

This code change will allow builders, designers, and architects to select a more extensive range of door components while maintaining the structural performance requirements of the code.

The Door Safety Council recommends "Approval as Modified by this Public Comment" at the Final Action Hearing.

Final Action: AS AM AMPC____ D

S156-07/08
1806.1

Proposed Change as Submitted:

Proponent: William Sherman, CH2M HILL, representing himself.

Revise 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 foundation divided by the net lateral force applied to the structure.

Reason: This proposal provides clarification and modification of existing code provisions.

An existing code clarification related to IBC 2000 states that the safety factor of 1.5 does not apply to load combinations that include seismic, but the published code remains unclear in this respect. Wording is added to clarify.

It is not practical to apply a reduction factor to dead loads, e.g. 0.6D, when an overall safety factor of 1.5 is also applied to stability requirements. Impractical and costly structures may result without the proposed revision.

The code does not define how the sliding safety factor is to be determined. Different safety factors can be obtained where passive pressures are included for retaining wall stability (depending upon whether the passive pressure force is included in the numerator or denominator). The proposed wording is based on recommended safety factor requirements in EM 1110-2-2502.

References:
EM 1110-2-2502, Retaining and Flood Walls, by the US Army Corps of Engineers (USACE).

Cost Impact: This code change proposal will reduce the cost of construction.

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

Individual Consideration Agenda

This item is on the agenda for individual consideration because a public comment was submitted.

Public Comment:

William C. Sherman, CH2M Hill, Inc, representing himself, requests Approval as Modified by this Public Comment.

Further 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. The safety factor against lateral sliding shall be taken as the sum of the available forces resisting lateral sliding divided by the lateral driving forces ~~available soil resistance at the base of the retaining wall foundation divided by the next lateral force~~ applied ~~to~~ the retaining wall.

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

Commenter's Reason: As the original proponent of the language currently approved for incorporation, I have concluded that my original proposed language may be inappropriately applied by code users, resulting in inflated values for the sliding factor of safety. The revised language is more consistent with traditional methods to calculate the sliding factor and safety.

The previously proposed language was based on what is referred to as the "USACE method" below. The proposed revised language is based on what is referred to as the "traditional method" below. The concern is that code users will not apply the USACE method strictly in accordance with the intended procedures, whereas code users commonly use the "traditional method" with less risk of error.

The "traditional method" used to evaluate the sliding factor of safety is as follows:

(resisting forces) / (driving forces) >= Reqd SF

However, the Corps of Engineers, USACE, (ref. EM 1110-2-2502 and the related computer program CT-WALL) uses a different method to evaluate the sliding factor of safety, as follows:

(friction + cohesion) / (Reqd SF) >= (net sliding force at base)

These two methods will be compared using the following definition of terms:

SF= Safety Factor against sliding
F = Friction at base
C = Cohesion at base
P = Passive pressure (maximum permissible)
R = Resisting pressure, as required for equilibrium
H = Horizontal driving force

If we define SF as the calculated factor of safety and we equate the left and right terms in each equation, the traditional method can be written as:

SF = (F+C+P) / H

It should be noted that for the traditional method, 'P' is often taken as the ultimate passive pressure, although some engineers use a reduced value to control lateral movements.

The "USACE method" can be written as:

(F+C) / SF = (H–R)

Or

SF = (F+C) / (H-R)

For the USACE method, 'R' is the resisting pressure required for equilibrium and generally should not exceed 2 times at-rest pressure; and 'H' is generally determined using at-rest earth pressure. The USACE method is reasonable when 'R' is applied as defined; however, the calculated safety factor can appear to be inflated if the designer inserts the maximum permissible passive pressure 'P' for 'R'; i.e., if '(H-P)' is used for '(H-R)' and results in a small number, the calculated safety factor becomes unrealistically inflated. Since many engineers are accustomed to using the full passive pressure in calculating the sliding factor of safety, there is a risk that excessively high factors of safety may be calculated if the original proposed wording based on the USACE approach is used to calculate the factor of safety.

Note that if we set R=P/SF as an upper design limit for commercial retaining walls, the USACE method can be modified to be written as:

SF = (F+C) / (H - P/SF)

And then solving for SF:

SF = (F+C+P) / H

The resulting equation is the same equation as shown for the "traditional method"!

Consequently, the "traditional method" uses conventional methods and is generally easier to use, so the proposed wording revision will reduce misapplication of the code provisions.

Final Action: AS AM AMPC____ D

S160-07/08

1808, 1809, 1810 (New), 1811, 1812

Proposed Change as Submitted:

Proponent: Edwin T. Huston, Smith & Huston, Inc., representing the National Council of Structural Engineering Associations

Revise as follows:

SECTION ~~1808~~ 1802
~~PIER AND PILE~~ DEFINITIONS

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

DEEP FOUNDATION. A deep foundation is a foundation element that does not satisfy the definition of a shallow foundation.

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

MICROPILES. ~~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 S~~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.

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

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:

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

but not less than:

$$\rho_s = 0.12(f'_c/f_{yh})[0.5 + 1.4P/(f'_c A_g)] \ \underline{\geq\ 0.12 f'_c / f_{yh}} \qquad \textbf{(Equation 18-6)}$$

and need not exceed:

$$\rho_s = 0.021 \qquad \textbf{(Equation 18-7)}$$

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:

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

but not less than:

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

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 (M□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:

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

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 __micro__piles or portions __there__of ~~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. __Micro__piles or portions __there__of ~~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__ structure__s__ ~~is~~ assigned to Seismic Design Category C, a permanent steel casing shall be provided from the top of the __micro__pile down ~~a minimum of 120 percent of the flexural length~~ __to the point of zero curvature__. ~~Where a~~ __For__ structure__s__ ~~is~~ assigned to Seismic

Design D, E or F, the <u>micro</u>pile 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~~ <u>1810.3.11</u> Pile caps. Pile caps shall be of reinforced concrete, and shall include all elements to which ~~piles~~ <u>vertical deep foundation elements</u> 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~~ <u>vertical deep foundation elements</u> 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~~ <u>the elements</u>. The tops of ~~piles~~ <u>elements</u> shall be cut <u>or chipped</u> back to sound material before capping.

~~1808.2.23.1.1~~ <u>1810.3.11.1</u> ~~Connection to pile cap~~ <u>Seismic Design Categories C through F.</u> ~~Concrete piles and concrete-filled steel pipe piles~~ <u>For structures assigned to Seismic Design Category C, D, E, or F in accordance with Section 1613, concrete deep foundation elements</u> shall be connected to the pile cap by embedding the ~~pile~~ <u>element</u> reinforcement or field-placed dowels anchored in the ~~concrete pile~~ <u>element</u> in<u>to</u> the pile cap for a distance equal to ~~the~~ <u>their</u> development length <u>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.</u> For deformed bars, the development length is the full development length for compression<u>,</u> or tension~~,~~ in the case of uplift, without reduction ~~in length~~ for excess ~~area~~ <u>reinforcement in accordance with Section 12.2.5 of ACI 318</u>. Alternative measures for laterally confining concrete and maintaining toughness and ductile-like behavior at the top of the ~~pile will~~ <u>element shall</u> 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~~ <u>pipes, tubes, or</u> H-piles to the pile cap shall be made by means other than concrete bond to the bare steel section. <u>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.</u>

> ~~**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~~ <u>1810.3.11.2</u> ~~Connection to pile cap~~ <u>Seismic Design Categories D through F</u>. For ~~piles required to resist~~ <u>structures assigned to Seismic Design Category D, E, or F in accordance with Section 1613, deep foundation element resistance to</u> uplift forces or ~~provide~~ rotational restraint~~, design of anchorage of piles into the pile cap shall be provided~~ <u>shall be provided by anchorage into the pile cap, designed</u> 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~~ <u>element</u> in tension. Anchorage into the pile cap shall be capable of developing the following:

1. In the case of uplift, the ~~lesser~~ <u>least</u> of the <u>following:</u> nominal tensile strength of the longitudinal reinforcement in a concrete ~~pile~~ <u>element</u>~~,~~<u>;</u> ~~or~~ the nominal tensile strength of a steel ~~pile~~ <u>element</u>~~,~~<u>;</u> ~~or~~ the ~~pile uplift soil nominal strength factored~~ <u>frictional force developed between the element and the soil multiplied</u> by 1.3<u>;</u> ~~or~~ <u>and</u> the axial tension force resulting from the load combinations of Section 1605.4.
2. In the case of rotational restraint, the lesser of <u>the following:</u> the axial ~~and~~ <u>force,</u> shear forces, and <u>bending</u> moments resulting from the load combinations of Section 1605.4<u>;</u> ~~or~~ <u>and</u> development of the full axial, bending and shear nominal strength of the ~~pile~~ <u>element</u>.

~~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.

<u>**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.</u>

~~**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:~~

$$\cancel{f_c = 0.33 f'_c - 0.27 f_{pc}} \qquad \textbf{(Equation 18-10)}$$

~~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.~~

Reason: Significant clarification, update, generalization, and simplification of the code requirements for deep foundations.

Reorganizes deep foundation requirements to eliminate repetition, fix conflicting definitions, and generalize and simplify requirements where possible. Most of the changes proposed are either purely editorial or nearly editorial. The substantive and nearly editorial changes are described herewith.

Definitions: The current definitions cause confusion and conflict. For instance, consider the definitions of "pier" and "pile". Some of the requirements for piers differ from those for piles, but the definitions only confuse matters. By the current definitions, piers must 1) be isolated, 2) be constructed of masonry or cast-in-place concrete, and 3) have a length of no more than 12 times the least horizontal dimension; piles must 1) be of concrete, wood, or steel either driven into the ground or cast in place, and 2) have a length exceeding 12 times the least horizontal dimension. As a result, foundation elements that have length less than 12 times the least horizontal dimension are **neither piers nor piles** if grouped or if constructed of wood, steel, or precast concrete. Several sections (such as 1808.2.5 and 1808.2.9.3) assume that piers are isolated, as the definition requires; Section 1812.1 addresses "isolated and multiple piers", which conflicts with the definition. The solution is, not to revise or add definitions but, to generalize and unify the requirements to the extent possible and then describe specific conditions of concern while specifying the related requirements. In order to unify, generalize, and simplify the requirements, some minor substantive changes are produced. Where the substantive change is small but the improvement in clarity and consistency of application is great, such revisions aid the registered design professional, the building official, and the public. This change proposal groups all deep foundation systems together (by defining shallow foundations) and sets forth general rules for the analysis, design, detailing, and installation of deep foundations. Specific deep foundation types are defined only where the rules for that type are so many and so peculiar that providing verbal descriptions for scoping would become unmanageable.

The exceptions that appeared in Section 1808.2.1 are embodied in the overall revisions to the requirements for deep foundations.

In a related proposal the numbered list in Section 1810.1.1 is moved to the section for geotechnical investigations.

New Section 1810.1.3 is based on current requirements. Concrete elements with height no greater than three times the least horizontal dimension are pedestals (not pilasters) per ACI 318. The exception in current Section 1812.8 is not really an exception, as it describes a different case than that addressed in the text (laterally supported versus unsupported). The intent of that exception is carried forward in the proposed text since it permits use of unreinforced sections where lateral support is provided, moment demands are less than the design cracking moment, and seismic concerns do not govern.

Since one of the conditions of concern in Section 1810.2.1 is "fluid soils", the revised text makes clear that the embedment required is the distance into either stiff soil or soft soil (not the distance below the ground surface). Although the terms "stiff soil" and "soft soil" are, strictly speaking, not (and never have been) defined in this code as related to this provision, they are in general agreement with the terms used in site classification (Section 1613.5.2, Site Classes D and E, respectively).

In new Section 1810.2.2 the first exception addresses a prime condition of what were previously termed "piers"—that is, permission to use isolated elements without additional lateral bracing.

New Sections 1810.2.4 and 1810.2.5 generalize requirements that were previously required for deep foundations of structures assigned to Seismic Design Category C, D, E, or F. This generalization is consistent with current practice and the recommendations of every published standard for deep foundations. Compliance is possible using traditional tables, formulas, or charts or using analysis methods that have been commonly employed for several decades.

New Section 1810.3.1.1 formalizes current practice as implied by the present text of Chapter 18 and as explicitly stated in other documents. The concrete design methods commonly employed no longer recognize allowable stress or working stress design. For many decades structural concrete design has employed the strength design method. However, there is a long tradition of using simple allowable stress design approaches for the proportioning of deep foundation elements (for both soil-foundation behavior and structural design). The proposed text is consistent with the design approaches specified in ACI 543 (Design, Manufacture, and Installation of Concrete Piles). Section 2.3 of that document reads (in part) as follows:

> "Whereas axial compression may often be the primary mode of loading, concrete piles are also frequently subjected to axial tension, bending, and shear loadings as well as various combinations of loading, as noted in Section 2.2. Concrete piles must have adequate structural capacity for all modes and combinations of loading that they will experience. For combined flexure and thrust loadings, the structural adequacy can be evaluated more readily through the use of moment-thrust interaction diagrams and strength-design methods.
>
> :... Because of the historical use of allowable capacities and stresses in piling design, however, recommendations are also provided for allowable axial service capacities for concentrically loaded, laterally supported piles. The allowable service capacities P_a recommended in Section 2.3.3 are intended specifically for cases in which the soil provides full lateral support to the pile and where the applied forces cause no more than minor bending moments resulting from accidental eccentricities. Piles subjected to larger bending moments or with unsupported lengths must be treated as columns in accordance with ACI 318-95 and the provisions given in Sections 2.3.2, 2.3.4, and 2.3.5 of this report."

Rather than treating composite elements in a different section, this proposal generalizes the requirement that each component of the composite element must comply with the applicable provisions of the code.

The proposed text uses the term "casing" in a manner consistent with current practice and use of the terms as defined in ACI 336.1 (Specification for the Construction of Drilled Piers). The term "casing" is appropriate where the element in question resists earth and water pressures. The term "liner" (which does not appear in this proposed text) applies where the element in question resists internal concrete pressures, but is "not designed for external earth and water pressures." Chapter 18 of the IBC does not venture so far into construction methods as to address liners. The only prior occurrences of "liner" (Section 1810.3.3) are related to "hydrostatic and lateral soil pressure," for which the term "casing" is more appropriate. Where the current text of Chapter 18 uses "shell" interchangeably with "casing", this proposal uses "casing" consistently.

The proposed treatment of timber deep foundation elements is more consistent both internally and with respect to the reference codes and standards. The present definition of "timber pile," which requires that the element be round and be placed tip first conflicts with the text that addresses sawn timber piles, which are square, and the reference standard (AF&PA NDS), which permits use of piles (tip first) or poles (butt first). New Section 1810.3.2.4 acknowledges use of both piles and poles.

Allowable stresses: The treatment of allowable stresses in this proposal is simple, clear, consistent, and even-handed as applied to deep foundation elements of different types. In generalizing and treating consistently, minor substantive changes result. The table below compares the allowable stresses specified in ACI 543, the 2006 IBC, and this proposal. For most types this proposal represents no change from the 2006 IBC. However, a few cases have changes of up to about 10 percent (which, practically speaking, is negligible). In the 2006 IBC, driven uncased piles and drilled or augered uncased piles have considerably different allowable stresses. This proposal splits the difference and is generally consistent with ACI 543. The real strength of the proposed approach is that it can be applied to other types of deep foundations without conflict, confusion, or question. Where the present text of Section 1808.2.3 is applied to "special types of piles" it is unclear which of the ten sets of allowable stresses should not be exceeded. Using the proposed text, which generalizes the treatment of allowable stresses, such questions have a ready, defensible answer.

Element type	**ACI 543**	**2006 IBC**	**This proposal**
Precast prestressed concrete piles	$0.33\,f'_c - 0.27\,f_{pc}$	$0.33\,f'_c - 0.27\,f_{pc}$	$0.33\,f'_c - 0.27\,f_{pc}$
Precast nonprestressed concrete piles	$0.33\,f'_c$ $0.39\,f_y$	$0.33\,f'_c$ $0.4\,f_y \le 30{,}000$ psi	$0.33\,f'_c$ $0.4\,f_y \le 30{,}000$ psi
Cast-in-place, uncased, plain	$0.29\,f'_c$	$0.25\,f'_c$	$0.3\,f'_c$
Cast-in-place, uncased, reinforced DRIVEN	$0.28\,f'_c$ $0.33\,f_y$	$0.25\,f'_c$	$0.3\,f'_c$ $0.4\,f_y \le 30{,}000$ psi
Cast-in-place, uncased, reinforced DRILLED or AUGERED	$0.28\,f'_c$ $0.33\,f_y$	$0.33\,f'_c$ $0.4\,f_y \le 25{,}500$ psi	$0.3\,f'_c$ $0.4\,f_y \le 30{,}000$ psi
Cast-in-place, cased	$0.32\,f'_c$	$0.33\,f'_c$	$0.33\,f'_c$
Cast-in-place, special casing	$0.26(\,f'_c + 8.2 t_s f_{ys}/D)$ $\le 0.4\,f'_c$	$0.4\,f'_c$	$0.4\,f'_c$
Cast-in-place, structural steel pipe or tube	$0.37\,f'_c$ $0.43\,f_{yp}$	$0.33\,f'_c$ $0.35\,F_y \le 12{,}600$ psi up to $0.5\,F_y$	$0.33\,f'_c$ $0.35\,F_y \le 16{,}000$ psi up to $0.5\,F_y \le 32{,}000$ psi
Caisson (socketed drilled shaft)	$0.37\,f'_c$ $0.43\,f_{yp}$	$0.33\,f'_c$ $0.35\,F_y$ (pipe or tube) $0.5\,F_y$ (core)	$0.33\,f'_c$ $0.35\,F_y \le 16{,}000$ psi up to $0.5\,F_y \le 32{,}000$ psi $0.5\,F_y$ (core)
Structural steel elements	---	$0.35\,F_y$ up to $0.5\,F_y$	$0.35\,F_y \le 16{,}000$ psi up to $0.5\,F_y \le 32{,}000$ psi
Micropiles	---	$0.33\,f'_c$ $0.4\,f_y \le 32{,}000$ psi $0.4\,F_y \le 32{,}000$ psi $0.6\,f_y$ Tension	$0.33\,f'_c$ $0.4\,f_y \le 30{,}000$ psi $0.4\,F_y \le 32{,}000$ psi $0.6\,f_y$ Tension
Timber	---	In accordance with the AF&PA NDS	In accordance with the AF&PA NDS

The first requirement of new Section 1810.3.2.7 is consistent with current practice and the requirements of ACI 543 Table 2.2. The second requirement is moved from Section 1810.5.1; it is the sealed tip that produces a displacement pile with increased capacity.

In new Section 1810.3.3.1.2 the requirement for load testing of cast-in-place deep foundation elements with an enlarged base previously appeared in Section 1810.2.4.

For improved clarity of application and consistency with ASCE 7-05, seismic requirements are rewritten to avoid "cascading," which often led to confusion concerning scope. For instance, new Section 1810.3.8.2.2 applies to Seismic Design Categories C through F and new Section 1810.3.8.2.3 provides "additional" requirements for Seismic Design Categories D through F; in both cases the scope is clearly defined. In another instance it was possible to separate the requirements; Section 1810.3.8.3.2 applies to Seismic Design Category C and Section 1810.3.8.3.3 (with revised Equation 18-6) applies to Seismic Design Categories D through F.

The change at the end of new Section 1810.3.8.2.2 is editorial although it may not appear so. First, the 8 inch maximum spacing is changed to 6 inches since new Section 1810.3.8.1 specifies a maximum spacing of 6 inches for non-seismic cases. Then, the spacing of 16 longitudinal bar diameters can be eliminated since 16 times the smallest bar diameter (3/8") is no more stringent.

The change in new Section 1810.3.8.3.2 is editorial since Equation 18-4 produces a value greater than 0.007 where the minimum value of f'_c (5 ksi) is used with the maximum value of f_{yh} (85 ksi).

The revision to Equation 18-6 eliminates cascading requirements from the section above.

New Sections 1810.3.9.1 and 1810.3.9.1 clarify the present requirements, agree with the requirements of ACI 318-08, and allow elimination of the definition for flexural length. For both uncased and cased cast-in-place deep foundation elements (but not concrete filled pipes and tubes) reinforcement must be provided where moments exceed a reasonable lower bound for the capacity of the plain concrete section. In several sections of the 2006 IBC (and other related documents) that design cracking moment is taken as 0.4 times the "concrete section cracking moment strength." Section 9.5.2.3 of ACI 318 defines the cracking moment strength as 7.5 times the square root of f'_c times the elastic section modulus of the gross section (0.4 × 7.5 = 3). Using Chapter 22 of ACI 318-08, one would take □ = 0.6 times 5.0 times the square root of f'_c times the elastic section modulus of the gross section (0.6 × 5.0 = 3). The proposed text is consistent with the current requirement and paves the way for use of a reference standard in the future. The proposed sections are also used in place of the less clear phrase "not rupture" in Exception 3 of new Section 1810.3.9.4. Editorial note: Where metric units are used, Equation 18-10 should be shown as $\phi M_n = 0.25\sqrt{f'_c}S_m$.

The proposed revisions in new Section 1810.3.10.1 clarifies the intent to permit the pipe or tube casing to terminate above the bond zone, with deformed bar reinforcement continuing below. It also specifies a splice condition for that transition.

In new Section 1810.3.10.4 "120 percent of the flexural length" is changed to "the point of zero curvature" for two reasons. First, with the revisions related to design cracking moment, this is the only section that uses the current definition of flexural length (first point of zero lateral deflection). Second, the distance to the point of zero curvature, which is also used in new Section 1810.3.8.3.3, is approximately equal to 120 percent of the distance to the point of zero deflection.

Bibliography:
Composite of Chapter 18 reorganization assuming all of proponent's proposals are approved.

Cost Impact: The code change proposal will not increase the cost of construction.

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	$0.4\ f_y \le 30{,}000$ psi
3. Structural steel in compression	
Cores within concrete-filled pipes or tubes	$0.5\ F_y \le 32{,}000$ psi
Pipes, tubes, or H-piles, where justified in accordance with Section 1810.3.2.8	$0.5\ F_y \le 32{,}000$ psi
Pipes or tubes for micropiles	$0.4\ F_y \le 32{,}000$ psi
Other pipes, tubes, or H-piles	$0.35\ F_y \le 16{,}000$ psi
~~5~~ 4. Nonprestressed reinforcement in tension	
Within micropiles	$0.6\ f_y$
Other conditions	$0.5\ f_y \le 24{,}000$ psi
~~6~~ 5. Structural steel in tension	
Pipes, tubes, or H-piles, where justified in accordance with Section 1810.3.2.8	$0.5\ F_y \le 32{,}000$ psi
Other pipes, tubes, or H-piles	$0.35\ F_y \le 16{,}000$ 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 resisting 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

Individual Consideration Agenda

This item is on the agenda for individual consideration because public comments were submitted.

Public Comment 1:

C. Michael Morgano, GRL Engineers, Inc, requests Approval as Modified by this Public Comment.

Further modify proposal as follows:

1810.3.3.1.2 Load tests. Where design compressive loads are greater than those determined using the allowable stresses specified in Section 1810.3.2.6 where design compressive loads are greater than those determined using the presumptive load-bearing values of soils in Section 1806, where the design load for any 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 element shall be load tested in each area of uniform subsoil conditions. Where required by the building official, additional 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 element as assessed by one of the published methods listed in Section 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 1810.2.3. In subsequent installation of the balance of deep foundation elements, all elements shall be deemed to have a supporting capacity equal to that of the control element where such elements are of the same type, size and relative length as the test element; are installed using the same or comparable methods and equipment as the test element; are installed in similar subsoil conditions as the test element; and, for driven elements, where the rate of penetration (e.g., net displacement per blow) of such elements is equal to or less than that of the test element driven with the same hammer through a comparable driving distance.

(Portions of proposal not shown remain unchanged)

Commenter's Reason: The change made to reference 1810.3.2.6 refers only to the structural strength limit of the element. The prior versions of the IBC code referred to the presumptive geotechnical strength (which is usually considerably smaller than the structural strength). Thus the current change actually reduces the amount of testing required substantially, relies then solely on static analysis in most cases which is notoriously inaccurate, and substantially increases the risk of failure (and essentially makes section 1806 irrelevant). The proposed addition restores the presumptive load limits which can then only be exceeded by performing the load tests, and thus reduces the risk of failures.

Public Comment 2:

C. Michael Morgano, GRL Engineers, Inc, requests Approval as Modified by this Public Comment.

Further modify proposal as follows:

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 slowly rotate in a clockwise direction during withdrawal. As the auger ~~shall be~~ is withdrawn at a steady rate or in increments not to exceed 1 foot (305 mm), ~~in continuous increments~~. ~~C~~ 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 by automated monitoring equipment to ensure that the volume of concrete or grout placed in each 2 feet) (610 mm) depth increment of the 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 or the concrete or grout volume pumped for any depth increment is less than required, 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 or the incremental volume pumped was deficient 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 more than 5 percent of the pile length, or due to installation of an adjacent element, the element shall be replaced.

(Portions of proposal not shown remain unchanged)

Commenter's Reason: These piles usually use grout and not concrete in North America. The word "concrete" should therefore be replaced with "grout" throughout, or supplemented by "or grout" as shown.

The auger rotation should be severely reduced during extraction to prevent excessive mining of soil or removal of freshly pumped grout, so the speed restriction "slowly" should be added.

The phrase "continuous increments" is unclear; while most augered cast-in-place (ACIP) contractors now withdraw the auger at a smooth continuous rate, some still rapidly withdraw a short distance followed by a stop-and-pump prior to the next distinct small increment withdrawal (repeating this process for then entire pile length).

The FHWA recent study (contracted to Dr Dan Brown of Auburn University and published as GEC#8) requires "automated monitoring equipment" that electronically measures grout volume pumped per incremental depth. The DFI ACIP committee has for years said "incremental grout volume" is the single most important installation control" in their "Augered Cast-in-Place Pile Manual" (which is almost universally recognized as the industry standard practice for ACIP installations). While the ACIP manual states a 5 ft increment, the 5 ft increment is based on visual inspection, and the automated monitoring equipment is considerably more accurate (and now in very common use, and available from several manufacturers) as reflected in the GEC#8 publication. Knowing only the total volume (as currently stated) for the entire element (entire pile) will not assure that the structural integrity of the shaft, particularly near the top, is adequate since the distribution of grout along the length is even more critical than the actual total volume.

The concrete level (actually grout level) usually declines slightly due to absorption into the surrounding soils (particularly in non-cohesive soils) so some tolerance should be stated (and the pile then restored to the correct cut-off elevation), and it is reasonable that the tolerance be related to the pile length. The DFI ACIP manual accomplishes restoration to the cutoff elevation by suggesting "adding small quantities of grout".

Final Action: AS AM AMPC____ D

S171-07/08

1818.1, 1813 (New)

Proposed Change as Submitted:

Proponent: Robert M. Hoyt, Hoyt Engineering, PC, Representing Deep Foundations Institute Committee on Helical Foundations and Tiebacks, Deep Foundations Institute

1. Add new test as follows:

1808.1 Definitions. The following words and terms shall, for the purposes of this section, have the meanings shown herein.

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, end bearing or a combination of both.

HELICAL PILE. Manufactured steel foundation pile consisting of a central shaft and one or more helical bearing plates. A helical pile is installed by rotating into the ground. Each helical bearing plate is formed into a screw thread with a uniform defined pitch.

SECTION 1813
HELICAL PILE FOUNDATIONS

1813.1 General. Helical pile foundations shall conform to the requirements of Sections 1813.2 through 1813.7.

1813.2 Dimensions. Dimensions of the central shaft and the number, size and thicknesses of helical bearing plates shall be sufficient to support the service loads, as listed in an ICC-ES evaluation report or as determined by a licensed design professional experienced in geotechnical engineering and the design of foundations utilizing helical piles.

1813.3 Design and manufacture. Helical piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by installation into the ground and service loads. When compliance of helical piles with Section 1813.2 is based on an ICC-ES evaluation, a copy of the evaluation report shall be provided to the building department.

1813.4 Allowable stresses. The allowable design stress, F_a, in the steel components of the pile shall not exceed the least value of the following:

$F_a = 0.6\ F_y$ or $0.5\ F_u$ **(Equation 18-11)**

where:
F_y = yield strength of the steel
F_u = ultimate strength of the steel

1813.5 Allowable loads

1813.5.1 Allowable axial load. The allowable axial design load, Q_a, of helical piles shall be limited to the least value associated with the interaction of the pile with the soil, the strength of the pile shaft, the strength of the shaft couplings, the strength of the helical bearing plates, or the strength of the joint between the helical bearing plates and the shaft, as listed in an ICC-ES evaluation report or as determined by a licensed design professional experienced in such evaluations.

1813.5.2 Allowable lateral load. The allowable lateral design load, Pa, of helical piles shall be limited to the least value associated with the interaction of the pile with the soil, the strength of the pile shaft, or the strength of the shaft couplings, as listed in an ICC-ES evaluation report or as determined by a licensed design professional experienced in such evaluations of helical piles.

1813.6 Special inspection. Special inspections in accordance with Section 1704.8 are required, as prescribed in Section 1808.2.22. The records submitted to the building official shall include, in addition to the records specified in 1704.8, installation equipment used; pile shaft dimensions, helix configuration and material grades; torsional resistance vs. embedment length; and such other installation data as the special inspector may deem appropriate.

1813.7 Installation. Helical piles shall be installed to specified embedment depth and torsional resistance criteria as stated in an applicable ICC-ES evaluation report or as determined by a licensed design professional experienced in geotechnical engineering and the design of foundations utilizing helical piles. The torque applied during installation shall not exceed the maximum allowable installation torque of the helical pile as listed in an applicable ICC-ES evaluation report or, in the absence of such a report, as recommended by the helical pile manufacturer.

Reason: The purpose of this proposal is to add provisions addressing the design and installation of helical pile foundations. Helical piles are not currently listed and their design and installation are not currently addressed in the Code. Helical pile foundations are becoming more and more common in civil construction. Proposed new Section 1813 contains specifications for the design and installation of helical pile foundations. The section will extend the coverage of the code to an increasingly popular but currently un-regulated type of deep foundation. The proposed definition, design and installation provisions conform to newly adopted ICC-ES AC358.

Cost Impact: The code change proposal will not increase the cost of construction.

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**

Individual Consideration Agenda

This item is on the agenda for individual consideration because public comments were submitted.

Public Comment:

Commenter's Reason: As stated in the floor modification submitted during the hearing, design engineers and building officials are in great need of a code standard and guide line for evaluating helical piles.

Commenter's Reason: Eliminates inappropriate references to ICC-ES documents, while making the requirements consistent therewith. Locates special inspection requirements in Chapter 17, and presents those requirements in a manner consistent with that for other foundations systems. Coordinates with the NCSEA proposals that totally reorganize Chapter 18.

Commenter's Reason- Buchanan, Craig, Davidson and Gray: As stated in the floor modification submitted during the hearing, design engineers and building officials are in great need of a code standard and guide line for evaluating helical piles

Commenter's Reason – Bobbitt: Change language to eliminate reference to ICC documents and to simplify language.

Commenter's Reason – Hermanson, Paap, Perko, Pinkleton, Shipman, John, and Shipman, Mary: Eliminates inappropriate references to ICC-ES documents, while making the requirements consistent therewith. Locates special inspection requirements in Chapter 17, and presents those requirements in a manner consistent with that for other foundations systems. Coordinates with the NCSEA proposals that totally reorganize Chapter 18.

Commenter's Reason – Neal: Eliminates inappropriate references to ICC-ES documents, while making the requirements consistent therewith. Locates special inspection requirements in Chapter 17, and presents those requirements in a manner consistent with that for other foundations systems. Coordinates with the NCSEA proposals that totally reorganize Chapter 18.

Helical piers are a reliable and time tested deep foundations when installed by a certified installer. The system is virtually self-testing and provides predictable and reliable results when installed correctly. For these reasons requirements for this system should be consistent with code requirements or other foundation systems.

Melissa Bass, Thomas R. Moran Construction Company, Inc.
Darla Bates, RJT Commercial, LLC.
Ryan Bates, Thomas R. Moran Construction Company, Inc.
Albert Beane, Jr., Walder Foundation Products
Donald E. Bobbitt, PE, Consulting Engineer, Centralia MO, representing himself
Jodie Bonnette, Thomas R. Moran Construction Company, Inc.
Glenda Brady, Ram Jack Systems Distribution, LLC
Denise Brown, Basement Cracks and Leaks, Metro Inc
Robert Brown, Arizona Repair Masons Inc, representing Ram Jack Systems Distribution, LLC
Brian Buchanan, RJT Construction, LLC
Doug Burwell, Thomas R. Moran Construction Company, Inc.
Dewayne Craig, Arizona Ram Jack, LLC
Frank D'Angelo, D'Angelo Brothers, LLC
Ron Davidson, Ram Jack Systems Distribution, LLC
Brad Davis, Thomas R. Moran Construction Company, Inc.
Justin Dean, Atlas Foundation Company
Mitchell Dearth, Thomas R. Moran Construction Company, Inc.
Scott Erlewing, Ram Jack of South Carolina, Inc.
Richard W. Follett, Ram Jack of Ohio., representing Ram Jack Systems Distribution, LLC
Clark Gray, Ram Jack Systems Distribution, LLC
Darren Gregory, Ram Jack Systems Distribution, LLC.
George W. Haffert, Danbro Distributors
Randall Hart, Thomas R. Moran Construction Company, Inc.
Ben Hermanson, Structural Anchor Supply, Atlas Foundation Co.
Richard Hightower, Hightower Geotechnical Services, Inc.
Russ Howell, Thomas R. Moran Construction Company, Inc.
William R. Howell, Pacific Ram Jack
Christopher Huntley, Atlas Foundation Co., Rodgers MN
Michael Janish, Atlas Foundation Company
Robb Johnson, P.E., Engineering & Construction Innovations, Inc., representing himself
Terri King, LZB Inc., dba Earth Anchors
Scott Mackay, Ram Jack of the Tri States Inc
Tressa G. McGinty, Ram Jack Manufacturing, LLC
Michael Melworm, Premium Technical Services

Thomas Moran, Thomas R. Moran Construction Company, Inc.
Mamdouh Nasr, Advanced Geosolutions, Inc., representing Deep Foundation Institute
Frederick D. Neal, AIA, P.E. representing F.d. Neal Construction Ltd.
Mike O'Connor, Thomas R. Moran Construction Company, Inc.
Tom Osborne, T-OZ Construction, Inc
Ron Paap, Pacific Helix Distributing, Inc. So
Howard Perko, Ph.D., P.E., CTL Thompson, Inc.
Charles Gary Peterson, Peterson Structural Engineers
Lea Ann Pharr, Ram Jack Systems Distribution, LLC
Michael A. Pinkleton, Intech Anchoring Systems, Inc.
Mike Pinley, Ram Jack Manufacturing LLC
Danny Plaugher, Thomas R. Moran Construction Company, Inc.
Delton Riehle, HiTech Foundations
Joe Rainer, Thomas R. Moran Construction Company, Inc.
Staci Ritchie, Thomas R. Moran Construction Company, Inc.
Randy Robertson, Cyntech Corporation
Mike Robinson, Thomas R. Moran Construction Company, Inc.
Frank Russell, Integral Construction
Luis M. Sanchez, Ram Jack of Virginia, Inc.
Maria T. Sanchez, Ram Jack MD, requests
Josh Sanders, Ram Jack Systems Distribution, LLC
Francine Schauwecker, Ram Jack Foundation Solutions of East Tennessee and NW Georgia
John Schauwecker, Ram Jack Foundation Solutions of East Tennessee and NW Georgia
Scott Schauwecker, Ram Jack Foundation Solutions of East Tennessee and NW Georgia
Bryan Schley, Custom Structures Foundation Systems, Inc.
Josh Schofield, Thomas R. Moran Construction Company, Inc.
Gary L. Seider, Hubbell Power Systems, representing Chance Civil Construction
John L. Shipman, Ram Jack Systems Distribution LLC
Mary M. Shipman, Ram Jack Systems Distribution LLC
Darin Willis, P.E. Ram Jack Manufacturing LLC
Brandon Smith, Thomas R. Moran Construction Company, Inc.
Jason Strauss, Thomas R. Moran Construction Company, Inc.
Danny Stephens, Thomas R. Moran Construction Company, Inc.
Jeff Torson, LZB Inc., dba Earth Anchors
Kevin Wallace, Thomas R. Moran Construction Company, Inc.
Eric Walt, Atlas Foundation Company
Darla Walters, Thomas R. Moran Construction Company, Inc.
Paul Weingart, Atlas Foundation Company
Lee Zumwalt, LZB, Inc., dba Earth Anchors

Replace proposal as follows:

1. Revise as follows:

1802.1 Definitions. The following words and terms shall, for the purposes of this chapter, have the meanings shown herein.

<u>**HELICAL PILE**. Manufactured steel deep foundation element consisting of a central shaft and one or more helical bearing plates. A helical pile is installed by rotating it into the ground. Each helical bearing plate is formed into a screw thread with a uniform defined pitch.</u>

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~~ <u>1810.3.1.6</u>, as applicable.

<u>**1810.3.1.5 Helical Piles**. Helical piles shall be designed and manufactured in accordance with accepted engineering practice to resist all stresses induced by installation into the ground and service loads.</u>

[Renumber subsequent sections]

TABLE 1810.3.2.6
ALLOWABLE STRESSES FOR MATERIALS USED IN DEEP FOUNDATION ELEMENTS

MATERIAL TYPE AND CONDITION	MAXIMUM ALLOWABLE STRESS [a]
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 Helical 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 $0.6\ F_y \leq 0.5\ F_u$
5. 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 Helical piles	 $0.5\ F_y \leq 32{,}000$ psi $0.35\ F_y \leq 16{,}000$ psi $0.6\ F_y \leq 0.5\ F_u$

(Portions of table not shown remain unchanged)

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; F_u is the specified minimum tensile 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.

1810.3.3.1 Allowable axial load. The allowable axial load on a deep foundation element shall be determined in accordance with Section 1810.3.3.1.1 through 1810.3.3.1.~~8~~9.

…

1810.3.3.1.9 Helical piles. The allowable axial design load, P_a, of helical piles shall be determined as follows:

$$P_a = 0.5\ P_u \qquad \text{(Equation 18-4)}$$

where P_u is the least value of:

P_u = sum of the areas of the helical bearing plates times the ultimate bearing capacity of the soil or rock comprising the bearing stratum
P_u = ultimate capacity determined from well documented correlations with installation torque
P_u = ultimate capacity determined from load tests
P_u = ultimate axial capacity of pile shaft
P_u = ultimate axial capacity of pile shaft couplings
P_u = sum of the ultimate axial capacity of helical bearing plates affixed to pile

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

1810.3.5.3.3 Helical piles. Dimensions of the central shaft and the number, size and thicknesses of helical bearing plates shall be sufficient to support the design loads.

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.

1810.4.10 Helical piles. Helical piles shall be installed to specified embedment depth and torsional resistance criteria as determined by a registered design professional. The torque applied during installation shall not exceed the maximum allowable installation torque of the helical pile.

1810.4.~~10~~11 Special inspection. Special inspections in accordance with Sections 1704.8 and 1704.9 shall be provided for driven and cast-in-place deep foundation elements, respectively. Special inspections in accordance with Section 1704.10 shall be provided for helical piles.

2. Add new text as follows:

1704.10 Helical Pile Foundations. Special inspections shall be performed continuously during installation of helical pile foundations. The information recorded shall include installation equipment used, pile dimensions, tip elevations, final depth, final installation torque, and other pertinent installation data as required by the registered design professional in responsible charge. The approved geotechnical report and the documents prepared by the registered design professional shall be used to determine compliance.

[Renumber subsequent sections]

Final Action: AS AM AMPC____ D

S173-07/08
1908.1.16

Proposed Change as Submitted:

Proponent: John W. Lawson, SE, representing the Structural Engineers Association of California Seismology Tilt-up Subcommittee; David L. McCormick, SE, representing the Structural Engineers Association of California Existing Building Tilt-up Subcommittee

Revise 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 ASCE 7 Equation 12.11-1 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 ASCE 7 Equation 12.11-1 need not satisfy Section D.3.3.5.

Reason: Purpose: To remove the requirements for anchorage ductility at concrete wall anchorage already designed for maximum expected seismic forces.

Background: Following the 1994 Northridge earthquake, surveys of damage to concrete and masonry buildings with flexible roof diaphragms revealed that very limited amounts of wall anchorage ductility was present to resist the induced forces. Brittle tensile failures in steel wall anchorage straps were especially troublesome. In addition, boundary nailing in plywood diaphragms tore out of the plywood edges due to wall anchorage elongation. New code provisions were introduced into the 1997 UBC to address the nonductile wall anchorage behavior observed in the Northridge earthquake.

1997 UBC Section 1633.2.8.1 was chiefly written to address many of the wall anchorage issues spotlighted in the Northridge earthquake. The lack of observed ductility and the need for greater anchorage strength were the reasons behind Section 1633.2.8.1 Items 1 and 4. Wall anchorage forces to flexible diaphragms in Seismic Zones 3 & 4 were increased 50% (a_p=1.5) and steel elements had an additional 1.4 force multiplier.

The intent of Section 1633.2.8.1 (items 1, 4, and 5) was for the wall anchorage system to resist brittle failure when subjected to maximum expected roof accelerations. Based on observations of Northridge earthquake damage, it was deemed best to resist brittle failure through the use of significantly higher design forces in conjunction with anticipated material overstrength instead of any reliance on ductility. As a result, material-specific load factors were introduced to provide a uniform level of protection against brittle failure (1.4 steel, 0.85 wood, 1.0 concrete/masonry). This approach is well documented in the 1999 SEAOC Recommended Lateral Force Requirements and Commentary (The Blue Book) [Reference C108.2.8.1].

As further evidence of the intent of these wall anchorage provisions, the 1999 SEAOC Blue Book Commentary states that the reduced R_p value for nonductile and shallow anchorage does not apply to wall anchorage designed using this overstrength approach of Section 1633.2.8.1 [Reference C108.2.8.1].

Current Provisions: In the development of ASCE 7-05, the intent was to maintain the same wall anchorage equation between the 1997 UBC and ASCE 7-05 for flexible diaphragms in high seismic zones. The wall anchorage provisions of ASCE 7-05 Section 12.11.2.1 are directly incorporated from the 1997 UBC Section 1633.2.8.1. Substituting $C_a = 0.4S_{DS}$ (2003 NEHRP Commentary), it can be confirmed that Eq. 12.11-1 is generally equivalent to the 1997 UBC.

Through an unrelated parallel effort, ACI 318-05 Appendix D Sections D.3.3.4 and D.3.3.5 require anchorage ductility in moderate and high seismic zones. ACI's ductility requirement conflicts with the intent behind Section 12.11.2.1 at wall anchorage situations. Furthermore, 2006 IBC Section 1908.1.16 allows an additional 2.5 load factor on top of ASCE forces in lieu of the ACI ductility requirement. This stacking of load factors on top of load factors and ductility requirements is in conflict with the original intent of the wall anchorage provisions.

To summarize, the 1997 UBC and subsequent ASCE 7-05 implement very high wall anchorage force levels to achieve uniform protection against brittle failure without reliance upon ductility. This was achieved using a rational approach considering inherent overstrength. Through the incorporation of ACI 318 Appendix D, anchorage ductility requirements were inadvertently added to these special wall anchorage situations in conflict with the original intent of the provisions. Furthermore, the 2006 IBC force multiplier of 2.5 is redundant to the original force increase behind the UBC and ASCE wall anchorage provisions.

<u>Impact to Design & Construction:</u> Achieving anchorage ductility under ACI 318 Appendix D is very difficult for tilt-up construction with flexible diaphragms. For the ductility condition to be met, steel anchor strength must be weaker than the concrete breakout strength. Because tilt-up walls are inherently thin slender wall designs, anchor embedment depth is limited, making it difficult to increase. In several parametric studies, it is apparent that the ductility provision encourages smaller diameter steel anchors or thicker concrete walls for deeper embedments. Neither of these approaches seems beneficial.

Another unintended consequence of providing ductile anchorage is the potential elongation of the steel causing boundary nailing at plywood diaphragms to tear out of the sheathing edges under maximum seismic force levels. Similar concerns exist for edge welding along steel deck diaphragms at the wall panels.

Using the 2006 IBC 1908.1.16 alternative, the forces are increased to an extreme level due to the 2.5-times load increase previously mentioned. In several parametric studies, this results in a larger number of thin anchors rods spread out over a larger connection area. Spreading these anchor rods out will likely result in non-uniform anchorage force distribution, and instead concentrate the forces over the closest few rods, potentially resulting in a progressive rod failure.

References:

1. SEAOC, <u>*Recommended Lateral Force Requirements and Commentary*</u>, Structural Engineers Association of California, 1999.
2. HARRIS, "Response of Tilt-up Buildings to Seismic Demands: Case Studies from the 1994 Northridge Earthquake," By S.K. Harris, R.O. Hamburger, S.C. Martin, D.L. McCormick, and P.G. Somerville. <u>*Proceedings of the NEHRP Conference and Workshop of Research on the Northridge, California Earthquake of January 17, 1994.*</u> California Universities for Research in Earthquake Engineering (CUREE), Richmond, California. 1998.

Cost Impact: The code change proposal will not increase the cost of construction.

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

Individual Consideration Agenda

This item is on the agenda for individual consideration because a public comment was submitted.

Public Comment:

Joseph J. Messersmith, Jr., P.E., Portland Cement Association, requests Approval as Modified by this Public Comment.

Further modify the 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. ~~Wall~~ Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 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. ~~Wall~~ Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.5.

Commenter's Reason: Upon further review of the modified version of S173 following the Palm Springs hearings, it was felt that the provisions may be incorrectly interpreted to apply to any anchor in a wall, which is not the intent. The revised wording should make it clear that the provisions are intended for anchors providing resistance to wall out-of-plane anchorage forces prescribed in ASCE 7.

Based on the forgoing, you are urged to vote in favor of the motion that will be made to further modify the proposal as indicated above.

Final Action: AS AM AMPC____ D

S175-07/08

2101.2.2 through 2101.3, 2102, 2103.8, Table 2103.8(1), Table 2103.8(2), 2103.11, 2103.11.1 through 2103.12, 2103.13, 2103.13.1 through 2103.13.8, 2104.1 through 2104.1.2, 2104.1.2.1 through 2104.1.2.7, 2104.1.5, 2104.1.7 through 2104.1.8, 2104.2, 2104.3, 2104.3.1 through 2104.3.3.5, 2104.4, 2104.4.1 through 2104.5, 2105.2.2.1.1, Table 2105.2.2.1.1, 2105.2.2.1.2, 2105.2.2.1.3, 2106.1, 2106.1.1 through 2106.6, 2107.1, 2107.2, 2107.4, 2107.5, 2107.6, 2107.7, 2107.8, 2108.1 through 2108.3, 2310.8.4, 2109(New) through 2110 (New)

Proposed Change as Submitted:

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards

1. Revise as follows:

2101.2.2 (Supp) Strength design. Masonry designed by the strength design method shall comply with the provisions of Sections 2106 and 2108, except that autoclaved aerated concrete (AAC) masonry shall comply with the provisions of Section 2106, Section 1613.6.3 and Chapter 1 and Appendix A of TMS 402/ACI 530/ASCE 5.

2101.2.3 Prestressed masonry. Prestressed masonry shall be designed in accordance with Chapters 1 and 4 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 and Section 2106. Special inspection during construction shall be provided as set forth in Section 1704.5.

2101.2.4 Empirical design. Masonry designed by the empirical design method shall comply with the provisions of Sections 2106 and 2109 or Chapter 5 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

2101.2.5 Glass unit masonry. Glass unit masonry shall comply with the provisions of Section 2110 or Chapter 7 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

2101.2.6 Masonry veneer. Masonry veneer shall comply with the provisions of Chapter 14 or Chapter 6 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

2101.3 Construction documents. The construction documents shall show all of the items required by this code including the following:

1. Specified size, grade, type and location of reinforcement, anchors and wall ties.
2. Reinforcing bars to be welded and welding procedure.
3. Size and location of structural elements.
4. Provisions for dimensional changes resulting from elastic deformation, creep, shrinkage, temperature and moisture.
5. Loads used in the design of masonry.
6. Specified compressive strength of masonry at stated ages or stages of construction for which masonry is designed, except where specifically exempted by this code.
7. Details of anchorage of masonry to structural members, frames, and other construction, including the type, size, and location of connectors.
8. Size and location of conduits, pipes, and sleeves.
9. The minimum level of testing and inspection as defined in Chapter 17, or an itemized testing and inspection program that meets or exceeds the requirements of Chapter 17.

2. Delete without substitution:

SECTION 2102
DEFINITIONS AND NOTATIONS

~~**BOND REINFORCING.** The adhesion between steel reinforcement and mortar or grout.~~

~~**BUTTRESS.** A projecting part of a masonry wall built integrally therewith to provide lateral stability.~~

~~**COLUMN, MASONRY.** An isolated vertical member whose horizontal dimension measured at right angles to its thickness does not exceed three times its thickness and whose height is at least four times its thickness.~~

~~**COMPOSITE ACTION.** Transfer of stress between components of a member designed so that in resisting loads, the combined components act together as a single member.~~

~~**COMPOSITE MASONRY.** Multiwythe masonry members acting with composite action.~~

~~**DIAPHRAGM.** A roof or floor system designed to transmit lateral forces to shear walls or other lateral-load-resisting elements.~~

~~**EFFECTIVE HEIGHT.** For braced members, the effective height is the clear height between lateral supports and is used for calculating the slenderness ratio. The effective height for unbraced members is calculated in accordance with engineering mechanics.~~

~~**HEADER (Bonder).** A masonry unit that connects two or more adjacent wythes of masonry.~~

~~**MEAN DAILY TEMPERATURE.** The average daily temperature of temperature extremes predicted by a local weather bureau for the next 24 hours.~~

~~**PLASTIC HINGE.** The zone in a structural member in which the yield moment is anticipated to be exceeded under loading combinations that include earthquakes.~~

3. Revise as follows:

NOTATIONS

~~A_n = Net cross-sectional area of masonry, square inches (mm^2).~~
~~b = Effective width of rectangular member or width of flange for T and I sections, inches (mm).~~
~~f_y = Specified yield stress of the reinforcement or the anchor bolt, psi (MPa).~~
~~L_w = Length of wall, inches (mm).~~
~~l_{de} = Embedment length of reinforcement, inches (mm).~~
~~P_w = Weight of wall tributary to section under consideration, pounds (N).~~
~~t = Specified wall thickness dimension or the least lateral dimension of a column, inches (mm).~~
~~V_n = Nominal shear strength, pounds (N).~~
~~V_u = Required shear strength due to factored loads, pounds (N).~~
~~W = Wind load, or related internal moments in forces.~~
~~γ = Reinforcement size factor.~~
~~ρ_n = Ratio of distributed shear reinforcement on plane perpendicular to plane of A_{mv}.~~
~~ρ_{max} = Maximum reinforcement ratio.~~
~~ϕ = Strength reduction factor.~~

P = The applied load at failure, pounds (N).
S_t = Thickness of the test specimen measured parallel to the direction of load, inches (mm).
S_w = Width of the test specimen measured parallel to the loading cylinder, inches (mm).

2103.8 Mortar. Mortar for use in masonry construction shall conform to ASTM C 270 ~~and shall conform to the proportion specifications of Table 2103.8(1) or the property specifications of Table 2103.8(2)~~. Type S or N mortar conforming to ASTM C 270 shall be used for glass unit masonry. ~~The amount of water used in mortar for glass unit masonry shall be adjusted to account for the lack of absorption. Retempering of mortar for glass unit masonry shall not be permitted after initial set. Unused mortar shall be discarded within 2½ hours after initial mixing, except that unused mortar for glass unit masonry shall be discarded within 1½ hours after initial mixing.~~

4. Delete without substitution:

~~TABLE 2103.8(1)~~
~~MORTAR PROPORTIONS~~

~~MORTAR~~	~~TYPE~~	~~PROPORTIONS BY VOLUME (cementitious materials)~~								~~AGGREATE MEASURED IN A DAMP, LOOSE CONDITION~~
		~~Portland cement[a] or blended cement[b]~~	~~Mortar cement[d]~~			~~Masonry cement[c]~~			~~HYDRATED LIME[e] OR LIME PUTTY~~	
			~~M~~	~~S~~	~~N~~	~~M~~	~~S~~	~~N~~		
~~Cement-Lime~~	~~M~~	~~1~~	—	—	—	—	—	—	~~$^1/_4$~~	~~Not less than $2^1/_4$ and not more than 3 times the sum of the separate volumes of cementitious materials~~
	~~S~~	~~1~~	—	—	—	—	—	—	~~over $^1/_4$ to $^1/_2$~~	
	~~N~~	~~1~~	—	—	—	—	—	—	~~over $^1/_2$ to $1^1/_4$~~	
	~~O~~	~~1~~	—	—	—	—	—	—	~~over $1^1/_4$ to $2^1/_2$~~	
~~Mortar Cement~~	~~M~~	~~1~~	—	—	~~1~~	—	—	—		
	~~M~~	—	~~1~~	—	—	—	—	—		
	~~S~~	~~$^1/_2$~~	—	—	~~1~~	—	—	—		
	~~S~~	—	—	~~1~~	—	—	—	—		
	~~N~~	—	—	—	~~1~~	—	—	—		
	~~O~~	—	—	—	~~1~~	—	—	—		
~~Masonry Cement~~	~~M~~	~~1~~	—	—	—	—	—	~~1~~		
	~~M~~	—	—	—	—	~~1~~	—	—		
	~~S~~	~~$^1/_2$~~	—	—	—	—	—	~~1~~		
	~~S~~	—	—	—	—	—	~~1~~	—		
	~~N~~	—	—	—	—	—	—	~~1~~		
	~~O~~	—	—	—	—	—	—	~~1~~		

~~a. Portland cement conforming to the requirements of ASTM C 150.~~
~~b. Blended cement conforming to the requirements of ASTM C 595.~~
~~c. Masonry cement conforming to the requirements of ASTM C 91.~~
~~d. Mortar cement conforming to the requirements of ASTM C 1329.~~
~~e. Hydrated lime conforming to the requirements of ASTM C 207.~~

~~TABLE 2103.8(2)~~
~~MORTAR PROPERTIES~~[a]

~~MORTAR~~	~~TYPE~~	~~AVERAGE COMPRESSIVE~~[b] ~~STRENGTH AT 28 DAYS minimum (psi)~~	~~WATER RETENTION minimum (%)~~	~~AIR CONTENT maximum (%)~~
~~Cement-Lime~~	~~M~~	~~2,500~~	~~75~~	~~12~~
	~~S~~	~~1,800~~	~~75~~	~~12~~
	~~N~~	~~750~~	~~75~~	~~14~~[c]
	~~O~~	~~350~~	~~75~~	~~14~~[c]
~~Mortar Cement~~	~~M~~	~~2,500~~	~~75~~	~~12~~
	~~S~~	~~1,800~~	~~75~~	~~12~~
	~~N~~	~~750~~	~~75~~	~~14~~[c]
	~~O~~	~~350~~	~~75~~	~~14~~[c]
~~Masonry Cement~~	~~M~~	~~2,500~~	~~75~~	~~18~~
	~~S~~	~~1,800~~	~~75~~	~~18~~
	~~N~~	~~750~~	~~75~~	~~20~~[d]
	~~O~~	~~350~~	~~75~~	~~20~~[d]

~~For SI: 1 inch = 25.4 mm, 1 pound per square inch = 6.895kPa.~~

a. ~~This aggregate ratio (measured in damp, loose condition) shall not be less than~~ $2^1/_4$ ~~and not more than 3 times the sum of the separate volumes of cementitious materials.~~

b. ~~Average of three 2-inch cubes of laboratory-prepared mortar, in accordance with ASTM C 270.~~

c. ~~When structural reinforcement is incorporated in cement-lime or mortar cement mortars, the maximum air content shall not exceed 12 percent.~~

d. ~~When structural reinforcement is incorporated in masonry cement mortar, the maximum air content shall not exceed 18 percent.~~

5. Revise as follows:

2103.11 Mortar for AAC masonry. Thin-bed mortar for AAC masonry shall comply with Article 2.1 C.1 of TMS 602/ACI 530.1/ASCE 6~~Section 2103.11.1. Mortar for leveling courses of AAC masonry shall comply with Section 2103.11.2~~. Mortar used for the leveling courses of AAC masonry shall comply with Article 2.1 C.2 of TMS 602/ACI 530.1/ASCE 6.

6. Delete without substitution:

~~**2103.11.1 Thin-bed mortar for AAC masonry.** Thin-bed mortar for AAC masonry shall be specifically manufactured for use with AAC masonry. Testing to verify mortar properties shall be conducted by the thin-bed mortar manufacturer and confirmed by an independent testing agency:~~

1. ~~The compressive strength of thin-bed mortar, as determined by ASTM C 109, shall meet or exceed the strength of the AAC masonry units.~~
2. ~~The shear strength of thin-bed mortar shall meet or exceed the shear strength of the AAC masonry units for wall assemblages tested in accordance with ASTM E 519.~~
3. ~~The flexural tensile strength of thin-bed mortar shall not be less than the modulus of rupture of the masonry units. Flexural strength shall be determined by testing in accordance with ASTM E 72 (transverse load test), ASTM E 518 Method A (flexural bond strength test) or ASTM C 1072 (flexural bond strength test).~~
   - 3.1. ~~For conducting flexural strength tests in accordance with ASTM E 518, at least five test specimens shall be constructed as stack-bonded prisms at least 32 inches (810 mm) high. The type of mortar specified by the AAC unit manufacturer shall be used.~~
   - 3.2. ~~For flexural strength tests in accordance with ASTM C 1072, test specimens shall be constructed as stack-bonded prisms comprised with at least three bed joints. A total of at least five joints shall be tested using the type of mortar specified by the AAC unit manufacturer.~~
4. ~~The splitting tensile strength of AAC masonry assemblages composed of two AAC masonry units bonded with one thin-bed mortar joint shall be determined in accordance with ASTM C 1006 and shall equal or exceed~~ $\cancel{2.4\sqrt{f'_{AAC}}}$.

7. Revise as follows:

2103.12 Grout. Grout shall comply with Article 2.1 C.1 of TMS 602/ACI 530.1/ASCE 6. ~~conform to Table 2103.12 or to ASTM C 476. When grout conforms to ASTM C 476, the grout shall be specified by proportion requirements or property requirements.~~

8. Delete without substitution:

~~TABLE 2103.12~~
~~GROUT PROPORTIONS BY VOLUME FOR MASONRY CONSTRUCTION~~

~~TYPE~~	**~~PARTS BY VOLUME OF PORTLAND CEMENT OR BLENDED CEMENT~~**	**~~PARTS BY VOLUME OF HYDRATED LIME OR LIME PUTTY~~**	**~~AGGREGATE, MEASURED IN A DAMP, LOOSE CONDITION~~**	
			~~Fine~~	~~Coarse~~
~~Fine Grout~~	~~1~~	~~0-$^{1}/_{10}$~~	~~$2^{1}/_{4}$–3 times the sum of the volumes of the cementitious materials~~	—
~~Coarse Grout~~	~~1~~	~~0-$^{1}/_{10}$~~	~~$2^{1}/_{4}$–3 times the sum of the volumes of the cementitious materials~~	~~1–2 times the sum of the volumes of the cementitious materials~~

9. Revise as follows:

2103.13 Metal reinforcement and accessories. Metal reinforcement and accessories shall conform to ~~Sections 2103.13.1 through 2103.13.8~~ Article 2.4 of TMS 602/ACI 530.1/ASCE 6 Where unidentified reinforcement is approved for use, not less than three tension and three bending tests shall be made on representative specimens of the reinforcement from each shipment and grade of reinforcing steel proposed for use in the work.

10. Delete without substitutions:

~~**2103.13.1 Deformed reinforcing bars.** Deformed reinforcing bars shall conform to one of the following standards:~~

~~ASTM A 615 for deformed and plain billet-steel bars for concrete reinforcement; ASTM A 706 for low-alloy steel deformed bars for concrete reinforcement; ASTM A 767 for zinc-coated reinforcing steel bars; ASTM A 775 for epoxy-coated reinforcing steel bars; and ASTM A 996 for rail and axle steel-deformed bars for concrete reinforcement.~~

~~**2103.13.2 Joint reinforcement.** Joint reinforcement shall comply with ASTM A 951. The maximum spacing of cross wires in ladder-type joint reinforcement and point of connection of cross wires to longitudinal wires of truss-type reinforcement shall be 16 inches (400 mm).~~

~~**2103.13.3 Deformed reinforcing wire.** Deformed reinforcing wire shall conform to ASTM A 496.~~

~~**2103.13.4 Wire fabric.** Wire fabric shall conform to ASTM A 185 for plain steel-welded wire fabric for concrete reinforcement or ASTM A 497 for welded deformed steel wire fabric for concrete reinforcement.~~

~~**2103.13.5 Anchors, ties and accessories.** Anchors, ties and accessories shall conform to the following standards:~~

~~ASTM A 36 for structural steel; ASTM A 82 for plain steel wire for concrete reinforcement; ASTM A 185 for plain steel-welded wire fabric for concrete reinforcement; ASTM A 240 for chromium and chromium-nickel stainless steel plate, sheet and strip; ASTM A 307 Grade A for anchor bolts; ASTM A 480 for flat rolled stainless and heat-resisting steel plate, sheet and strip; and ASTM A 1008 for cold-rolled carbon steel sheet.~~

~~**2103.13.6 Prestressing tendons.** Prestressing tendons shall conform to one of the following standards:~~

1. ~~Wire...ASTMA421~~
2. ~~Low-relaxation wire.......................ASTMA421~~
3. ~~Strand..ASTMA416~~
4. ~~Low-relaxation strand...................ASTMA416~~
5. ~~Bar...ASTMA722~~

~~Exceptions:~~

1. ~~Wire, strands and bars not specifically listed in ASTM A 421, ASTM A 416 or ASTM A 722 are permitted, provided they conform to the minimum requirements in ASTM A 421, ASTM A 416 or ASTM A 722 and are approved by the architect/engineer.~~

~~2. Bars and wires of less than 150 kips per square inch (ksi) (1034 MPa) tensile strength and conforming to ASTM A 82, ASTM A 510, ASTM A 615, ASTM A 996 or ASTM A 706 are permitted to be used as prestressed tendons, provided that:~~
~~2.1. The stress relaxation properties have been assessed by tests according to ASTM E 328 for the maximum permissible stress in the tendon.~~
~~2.2. Other nonstress-related requirements of ACI 530/ASCE 5/TMS 402, Chapter 4, addressing prestressing tendons are met.~~

~~**2103.13.7 Corrosion protection.** Corrosion protection for prestressing tendons shall comply with the requirements of ACI 530.1/ASCE 6/TMS 602, Article 2.4G. Corrosion protection for prestressing anchorages, couplers and end blocks shall comply with the requirements of ACI 530.1/ASCE 6/TMS 602, Article 2.4H. Corrosion protection for carbon steel accessories used in exterior wall construction or interior walls exposed to a mean relative humidity exceeding 75 percent shall comply with either Section 2103.13.7.2 or 2103.13.7.3. Corrosion protection for carbon steel accessories used in interior walls exposed to a mean relative humidity equal to or less than 75 percent shall comply with either Section 2103.13.7.1, 2103.13.7.2 or 2103.13.7.3.~~

~~**2103.13.7.1 Mill galvanized.** Mill galvanized coatings shall be applied as follows:~~

1. ~~For joint reinforcement, wall ties, anchors and inserts, a minimum coating of 0.1 ounce per square foot ($31 g/m^2$) complying with the requirements of ASTM A 641 shall be applied.~~
2. ~~For sheet metal ties and sheet metal anchors, a minimum coating complying with Coating Designation G-60 according to the requirements of ASTM A 653 shall be applied.~~
3. ~~For anchor bolts, steel plates or bars not exposed to the earth, weather or a mean relative humidity exceeding 75 percent, a coating is not required.~~

~~**2103.13.7.2 Hot-dipped galvanized.** Hot-dipped galvanized coatings shall be applied after fabrication as follows:~~

1. ~~For joint reinforcement, wall ties, anchors and inserts, a minimum coating of 1.5 ounces per square foot ($458 g/m^2$) complying with the requirements of ASTM A 153, Class B shall be applied.~~
2. ~~For sheet metal ties and anchors, the requirements of ASTM A 153, Class B shall be met.~~
3. ~~For steel plates and bars, the requirements of either ASTM A 123 or ASTM A 153, Class B shall be met.~~

~~**2103.13.7.3 Epoxy coatings.** Carbon steel accessories shall be epoxy coated as follows:~~

1. ~~For joint reinforcement, the requirements of ASTM A 884, Class A, Type 1 having a minimum thickness of 7 mils (175 *μm*) shall be met.~~
2. ~~For wire ties and anchors, the requirements of ASTM A 899, Class C having a minimum thickness of 20 mils (508 *μm*) shall be met.~~
3. ~~For sheet metal ties and anchors, a minimum thickness of 20 mils (508 μ*m*) per surface shall be provided or a minimum thickness in accordance with the manufacturer's specification shall be provided.~~

~~**2103.13.8 Tests.** Where unidentified reinforcement is approved for use, not less than three tension and three bending tests shall be made on representative specimens of the reinforcement from each shipment and grade of reinforcing steel proposed for use in the work.~~

11. Revise as follows:

2104.1 Masonry construction. Masonry construction shall comply with the requirements of Sections 2104.1.1 through 2104.5 and with ~~ACI 530.1/ASCE 6/TMS 602~~ TMS 602/ACI 530.1/ASCE 6.

2104.1.1 Tolerances. Masonry, except masonry veneer, shall be constructed within the tolerances specified in ~~ACI 530.1/ASCE 6/TMS 602~~ TMS 602/ACI 530.1/ASCE 6.

2104.1.2 Placing mortar and units. Placement of mortar, grout, and clay, concrete, glass, and AAC masonry ~~and concrete~~ units shall comply with ~~Sections 2104.1.2.1, 2104.1.2.2, 2104.1.2.3 and 2104.1.2.5. Placement of mortar and glass unit masonry shall comply with Sections 2104.1.2.4 and 2104.1.2.5. Placement of thin-bed mortar and AAC masonry shall comply with Section 2104.1.2.6~~ TMS 602/ACI 530.1/ASCE 6.

12. Delete without substitution:

~~**2104.1.2.1 Bed and head joints.** Unless otherwise required or indicated on the construction documents, head and bed joints shall be $^3/_8$ inch (9.5 mm) thick, except that the thickness of the bed joint of the starting course placed over foundations shall not be less than ¼ inch (6.4 mm) and not more than ¼ inch (19.1 mm).~~

~~**2104.1.2.1.1 Open-end units.** Open-end units with beveled ends shall be fully grouted. Head joints of open-end units with beveled ends need not be mortared. The beveled ends shall form a grout key that permits grouts within $^5/_8$ inch (15.9 mm) of the face of the unit. The units shall be tightly butted to prevent leakage of the grout.~~

~~**2104.1.2.2 Hollow units.** Hollow units shall be placed such that face shells of bed joints are fully mortared. Webs shall be fully mortared in all courses of piers, columns, pilasters, in the starting course on foundations where adjacent cells or cavities are to be grouted, and where otherwise required. Head joints shall be mortared a minimum distance from each face equal to the face shell thickness of the unit.~~

~~**2104.1.2.3 Solid units.** Unless otherwise required or indicated on the construction documents, solid units shall be placed in fully mortared bed and head joints. The ends of the units shall be completely buttered. Head joints shall not be filled by slushing with mortar. Head joints shall be constructed by shoving mortar tight against the adjoining unit. Bed joints shall not be furrowed deep enough to produce voids.~~

~~**2104.1.2.4 Glass unit masonry.** Glass units shall be placed so head and bed joints are filled solidly. Mortar shall not be furrowed. Unless otherwise required, head and bed joints of glass unit masonry shall be $^1/_4$ inch (6.4 mm) thick, except that vertical joint thickness of radial panels shall not be less than $^1/_8$ inch (3.2 mm). The bed joint thickness tolerance shall be minus 1/16 inch (1.6 mm) and plus 1/8 inch (3.2 mm). The head joint thickness tolerance shall be plus or minus $^1/_8$ inch (3.2 mm).~~

~~**2104.1.2.5 Placement in mortar.** Units shall be placed while the mortar is soft and plastic. Any unit disturbed to the extent that the initial bond is broken after initial positioning shall be removed and relaid in fresh mortar.~~

~~**2104.1.2.6 Thin-bed mortar and AAC masonry units.** AAC masonry construction shall begin with a leveling course of masonry meeting the requirements of Section 2104.1.2. Subsequent courses of AAC masonry units shall be laid with thin-bed mortar using a special notched trowel manufactured for use with thin-bed mortar to spread the mortar so that it completely fills the bed joints. Unless otherwise specified, the head joints shall be similarly filled. Joints in AAC masonry shall be approximately $^1/_{16}$ inch (1.5 mm) and shall be formed by striking on the ends and tops of AAC masonry units with a rubber mallet. Minor adjustments in unit position shall be made while the mortar is still soft and plastic by tapping it into the proper position. Minor sanding of the exposed faces of AAC masonry shall be permitted to provide a smooth and plumb surface.~~

~~**2104.1.2.7 Grouted masonry.** Between grout pours, a horizontal construction joint shall be formed by stopping all wythes at the same elevation and with the grout stopping a minimum of $1^1/_2$ inches (38 mm) below a mortar joint, except at the top of the wall. Where bond beams occur, the grout pour shall be stopped a minimum of ½ inch (12.7 mm) below the top of the masonry.~~

13. Revise as follows:

2104.1.3 Installation of wall ties. ~~The ends of wall ties shall be embedded in mortar joints. Wall tie ends shall engage outer face shells of hollow units by at least ½ inch 12.7 mm). Wire wall ties shall be embedded at least 1½ inches (38 mm) into the mortar bed of solid masonry units or solid-grouted hollow units. Wall ties shall not be bent after being embedded in grout or mortar~~ Wall ties shall be installed in accordance with TMS 602/ACI 530.1/ASCE 6.

2104.1.5 Lintels. The design for lintels shall be in accordance with the masonry design provisions of either Section 2107 or 2108. ~~Minimum length of end support shall be 4 inches (102 mm).~~

14. Delete without substitution:

~~**2104.1.7 Masonry protection.** The top of unfinished masonry work shall be covered to protect the masonry from the weather.~~

~~**2104.1.8 Weep holes.** Weep holes provided in the outside wythe of masonry walls shall be at a maximum spacing of 33 inches (838 mm) on center (o.c.).Weep holes shall not be less than $^3/_{16}$ inch (4.8 mm) in diameter.~~

15. Delete and substitute as follows:

~~**2104.2 Corbeled masonry.** Except for corbels designed per Section 2107 or 2108, the following shall apply:~~

1. ~~Corbels shall be constructed of solid masonry units.~~
2. ~~The maximum corbeled projection beyond the face of the wall shall not exceed:~~

~~2.1. One-half of the wall thickness for multiwythe walls bonded by mortar or grout and wall ties or masonry headers or~~
~~2.2. One-half the wythe thickness for single wythe walls, masonry bonded hollow walls, multiwythe walls with open collar joints and veneer walls.~~

~~3. The maximum projection of one unit shall not exceed:~~
~~3.1. One-half the nominal unit height of the unit or~~
~~3.2. One-third the nominal thickness of the unit or wythe.~~

~~4. The back surface of the corbelled section shall remain within 1 inch (25 mm) of plane.~~

2104.2 Corbeled masonry. Corbeled masonry shall comply with the requirements of Section 1.12 of TMS 402/ACI 530/ASCE 5.

16. Revise as follows:

2104.3 Cold weather construction. The cold weather construction provisions of ~~ACI 530.1/ASCE 6/TMS 602~~ TMS 602/ACI 530.1/ASCE 6, Article 1.8 C, ~~or the following procedures~~ shall be implemented when ~~either~~ the ambient temperature falls below 40°F (4°C) ~~or the temperature of masonry units is below 40°F (4°C)~~.

17. Delete without substitution:

~~**2104.3.1 Preparation.**~~

1. ~~Temperatures of masonry units shall not be less than 20°F (-7°C) when laid in the masonry. Masonry units containing frozen moisture, visible ice or snow on their surface shall not be laid.~~
2. ~~Visible ice and snow shall be removed from the top surface of existing foundations and masonry to receive new construction. These surfaces shall be heated to above freezing, using methods that do not result in damage.~~

~~**2104.3.2 Construction.** The following requirements shall apply to work in progress and shall be based on ambient temperature.~~

~~**2104.3.2.1Construction requirements for temperatures between 40°F (4°C) and 32°F (0°C).** The following construction requirements shall be met when the ambient temperature is between 40°F (4°C) and 32°F (0°C):~~

1. ~~Glass unit masonry shall not be laid.~~
2. ~~Water and aggregates used in mortar and grout shall not be heated above 140°F (60°C).~~
3. ~~Mortar sand or mixing water shall be heated to produce mortar temperatures between 40°F (4°C) and 120°F (49°C) at the time of mixing. When water and aggregates for grout are below 32°F(0°C), they shall be heated.~~

~~**2104.3.2.2 Construction requirements for temperatures between 32°F (0°C) and 25°F (-4°C).** The requirements of Section 2104.3.2.1 and the following construction requirements shall be met when the ambient temperature is between 32°F (0°C) and 25°F (-4°C):~~

1. ~~The mortar temperature shall be maintained above freezing until used in masonry.~~
2. ~~Aggregates and mixing water for grout shall be heated to produce grout temperature between 70°F (21°C) and 120°F (49°C) at the time of mixing. Grout temperature shall be maintained above 70°F (21°C) at the time of grout placement.~~
3. ~~Heat AAC masonry units to a minimum temperature of 40°F (4°C) before installing thin-bed mortar.~~

~~**2104.3.2.3 Construction requirements for temperatures between 25°F (-4°C) and 20°F (-7°C).** The requirements of Sections 2104.3.2.1 and 2104.3.2.2 and the following construction requirements shall be met when the ambient temperature is between 25°F (-4°C) and 20°F (-7°C):~~

1. ~~Masonry surfaces under construction shall be heated to 40°F (4°C).~~
2. ~~Wind breaks or enclosures shall be provided when the wind velocity exceeds 15 miles per hour (mph) (24 km/h).~~
3. ~~Prior to grouting, masonry shall be heated to a minimum of 40°F (4°C).~~

~~**2104.3.2.4 Construction requirements for temperatures below 20°F (-7°C).** The requirements of Sections 2104.3.2.1, 2104.3.2.2 and 2104.3.2.3 and the following construction requirement shall be met when the ambient temperature is below 20°F (-7°C): Enclosures and auxiliary heat shall be provided to maintain air temperature within the enclosure to above 32°F (0°C).~~

~~**2104.3.3 Protection.** The requirements of this section and Sections 2104.3.3.1 through 2104.3.3.5 apply after the masonry is placed and shall be based on anticipated minimum daily temperature for grouted masonry and anticipated mean daily temperature for ungrouted masonry.~~

~~**2104.3.3.1 Glass unit masonry.** The temperature of glass unit masonry shall be maintained above 40°F (4°C) for 48 hours after construction.~~

~~**2104.3.3.2 AAC masonry.** The temperature of AAC masonry shall be maintained above 32°F (0°C) for the first 4 hours after thin-bed mortar application.~~

~~**2104.3.3.3 Protection requirements for temperatures between 40°F (4°C) and 25°F (-4°C).** When the temperature is between 40°F (4°C) and 25°F (-4°C), newly constructed masonry shall be covered with a weather-resistive membrane for 24 hours after being completed.~~

~~**2104.3.3.4 Protection requirements for temperatures between 25°F (-4°C) and 20°F (-7°C).** When the temperature is between 25°F (-4°C) and 20°F (-7°C), newly constructed masonry shall be completely covered with weather-resistive insulating blankets, or equal protection, for 24 hours after being completed. The time period shall be extended to 48 hours for grouted masonry, unless the only cement in the grout is Type III portland cement.~~

~~**2104.3.3.5 Protection requirements for temperatures below20°F (-7°C).** When the temperature is below20°F (-7°C), newly constructed masonry shall be maintained at a temperature above 32°F (0°C) for at least 24 hours after being completed by using heated enclosures, electric heating blankets, infrared lamps or other acceptable methods. The time period shall be extended to 48 hours for grouted masonry, unless the only cement in the grout is Type III portland cement.~~

18. Revise as follows:

2104.4 Hot weather construction. The hot weather construction provisions of ~~ACI 530.1/ASCE 6/TMS 602~~ TMS 602/ACI 530.1/ASCE 6, Article 1.8 D, ~~or the following procedures~~ shall be implemented when the ~~temperature or the temperature and wind-velocity limits of this section are exceeded~~ ambient air temperature exceeds 100°F (37.8°C), or exceeds 90°F (32.2°C) with a wind velocity greater than 8 mph (12.9 km/hr).

19. Delete without substitution:

~~**2104.4.1 Preparation.** The following requirements shall be met prior to conducting masonry work.~~

~~**2104.4.1.1 Temperature.** When the ambient temperature exceeds 100°F (38°C), or exceeds 90°F (32°C) with a wind velocity greater than 8 mph (3.5 m/s):~~

1. ~~Necessary conditions and equipment shall be provided to produce mortar having a temperature below 120°F (49°C).~~
2. ~~Sand piles shall be maintained in a damp, loose condition.~~

~~**2104.4.1.2 Special conditions.** When the ambient temperature exceeds 115°F (46°C), or 105°F (40°C) with a wind velocity greater than 8 mph (3.5 m/s), the requirements of Section 2104.4.1.1 shall be implemented, and materials and mixing equipment shall be shaded from direct sunlight.~~

~~**2104.4.2 Construction.** The following requirements shall be met while masonry work is in progress.~~

~~**2104.4.2.1 Temperature.** When the ambient temperature exceeds 100°F (38°C), or exceeds 90°F (32°C) with a wind velocity greater than 8 mph (3.5 m/s):~~

1. ~~The temperature of mortar and grout shall be maintained below 120°F (49°C).~~
2. ~~Mixers, mortar transport containers and mortar boards shall be flushed with cool water before they come into contact with mortar ingredients or mortar.~~
3. ~~Mortar consistency shall be maintained by retempering with cool water.~~
4. ~~Mortar shall be used within 2 hours of initial mixing.~~
5. ~~Thin-bed mortar shall be spread no more than 4 feet (1219 mm) ahead of AAC masonry units.~~
6. ~~AAC masonry units shall be placed within one minute after spreading thin-bed mortar.~~

~~**2104.4.2.2 Special conditions.** When the ambient temperature exceeds 115°F (46°C), or exceeds 105°F (40°C) with a wind velocity greater than 8 mph (3.5 m/s), the requirements of Section 2104.4.2.1 shall be implemented and cool mixing water shall be used for mortar and grout. The use of ice shall be permitted in the mixing water prior to use. Ice shall not be permitted in the mixing water when added to the other mortar or grout materials.~~

~~**2104.4.3 Protection.** When the mean daily temperature exceeds 100°F (38°C) or exceeds 90°F (32°C) with a wind velocity greater than 8 mph (3.5 m/s), newly constructed masonry shall be fog sprayed until damp at least three times a day until the masonry is three days old.~~

~~**2104.5 Wetting of brick.** Brick (clay or shale) at the time of laying shall require wetting if the unit's initial rate of water absorption exceeds 30 grams per 30 square inches (19 355 mm^2) per minute or 0.035 ounce per square inch (1 g/645mm^2) per minute, as determined by ASTM C 67.~~

20. Revise as follows:

2105.2.2.1.1 Clay masonry. The compressive strength of masonry shall be determined based on the strength of the units and the type of mortar specified using Table 2105.2.2.1.1, provided:

1. Units ~~conform to~~ <u>are sampled and tested to verify conformance with</u> ASTM C 62, ASTM C 216 or ASTM C 652.
2. Thickness of bed joints does not exceed 5/8 inch (15.9 mm).
3. For grouted masonry, the grout meets one of the following requirements:
   3.1. Grout conforms to ~~ASTM C 476~~ <u>Article 2.2 of TMS 602/ACI 530.1/ASCE 6</u>.
   3.2. Minimum grout compressive strength equals or exceeds f_m but not less than 2,000 psi (13.79MPa). The compressive strength of grout shall be determined in accordance with ASTM C 1019.

TABLE 2105.2.2.1.1
COMPRESSIVE STRENGTH OF CLAY MASONRY

NET AREA COMPRESSIVE STRENGTH OF CLAY MASONRY UNITS (psi)		NET AREA COMPRESSIVE STRENGTH OF MASONRY (psi)
Type M or S mortar	**Type N mortar**	
1,700	2,100	1,000
3,350	4,150	1,500
4,950	6,200	2,000
6,600	8,250	2,500
8,250	10,300	3,000
9,900	___	3,500
~~13,200~~<u>11,500</u>	___	4,000

For SI: 1 pound per square inch = 0.00689MPa.

2105.2.2.1.2 Concrete masonry. The compressive strength of masonry shall be determined based on the strength of the unit and type of mortar specified using Table 2105.2.2.1.2, provided:

1. Units ~~conform to~~ <u>are sampled and tested to verify conformance with</u> ASTM C 55 or ASTM C 90 ~~and are sampled and tested in accordance with ASTM C 140~~.
2. Thickness of bed joints does not exceed 5/8 inch (15.9 mm).
3. For grouted masonry, the grout meets one of the following requirements:
   3.1. Grout conforms to ~~ASTM C 476~~ <u>Article 2.2 of TMS 602/ACI 530.1/ASCE 6</u>.
   3.2. Minimum grout compressive strength equals or exceeds f_m but not less than 2,000 psi (13.79 MPa). The compressive strength of grout shall be determined in accordance with ASTM C 1019.

2105.2.2.1.3 AAC masonry. The compressive strength of AAC masonry shall be based on the strength of the AAC masonry unit only and the following shall be met:

1. Units conform to ASTM C 1386.
2. Thickness of bed joints does not exceed 1/8 inch (3.2 mm).
3. For grouted masonry, the grout meets one of the following requirements:
   3.1. Grout conforms to ~~ASTM C 476~~ <u>Article 2.2 of TMS 602/ACI 530.1/ASCE 6</u>.
   3.2. Minimum grout compressive strength equals or exceeds f'_{AAC} but not less than 2,000 psi (13.79 MPa). The compressive strength of grout shall be determined in accordance with ASTM C 1019.

21. Delete and substitute:

~~**2106.1 Seismic design requirements for masonry.** Masonry structures and components shall comply with the requirements in Section 1.14.2.2 and Section 1.14.3, 1.14.4, 1.14.5, 1.14.6 or 1.14.7 of ACI 530/ASCE 5/TMS 402 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 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.

22. Delete without substitution:

~~**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 1.14.2.2 of ACI 530/ASCE 5/TMS 402 or with Section 2106.1.1.1, 2106.1.1.2 or 2106.1.1.3.~~

~~**2106.1.1.1 Ordinary plain prestressed masonry shear walls.** Ordinary plain prestressed masonry shear walls shall comply with the requirements of Chapter 4 of ACI 530/ASCE 5/TMS 402.~~

~~**2106.1.1.2 Intermediate prestressed masonry shear walls.** Intermediate prestressed masonry shear walls shall comply with the requirements of Section 1.14.2.2.4 of ACI 530/ASCE 5/TMS 402 and shall be designed by Chapter 4, Section 4.4.3, of ACI 530/ASCE 5/TMS 402 for flexural strength and by Section 3.3.4.1.2 of ACI 530/ASCE 5/TMS 402 for shear strength. Sections 1.14.2.2.5, 3.3.3.5 and 3.3.4.3.2(c) of ACI 530/ASCE 5/TMS 402 shall be applicable for reinforcement. Flexural elements subjected to load reversals shall be symmetrically reinforced. The nominal moment strength at any section along a member shall not be less than one-fourth the maximum moment strength. The cross-sectional area of bonded tendons shall be considered to contribute to the minimum reinforcement in Section 1.14.2.2.4 of ACI 530/ASCE 5/TMS 402. Tendons shall be located in cells that are grouted the full height of the wall.~~

~~**2106.1.1.3 Special prestressed masonry shear walls.** Special prestressed masonry shear walls shall comply with the requirements of Section 1.14.2.2.5 of ACI 530/ASCE 5/TMS 402 and shall be designed by Chapter 4, Section 4.4.3, of ACI 530/ASCE 5/TMS 402 for flexural strength and by Section 3.3.4.1.2 of ACI 530/ASCE 5/TMS 402 for shear strength. Sections 1.14.2.2.5(a), 3.3.3.5 and 3.3.4.3.2(c) of ACI 530/ASCE 5/TMS 402 shall be applicable for reinforcement. Flexural elements subjected to load reversals shall be symmetrically reinforced. The nominal moment strength at any section along a member shall not be less than one-fourth the maximum moment strength. The cross-sectional area of bonded tendons shall be considered to contribute to the minimum reinforcement in Section 1.14.2.2.5 of ACI 530/ASCE 5/TMS 402.~~

~~**2106.1.1.3.1 Prestressing tendons.** Prestressing tendons shall consist of bars conforming to ASTM A 722.~~

~~**2106.1.1.3.2 Grouting.** All cells of the masonry wall shall be grouted.~~

~~**2106.2 Anchorage of masonry walls.** Masonry walls shall be anchored to the roof and floors that provide lateral support for the wall in accordance with Section 1604.8.2.~~

~~**2106.3 Seismic Design Category B.** Structures assigned to Seismic Design Category B shall conform to the requirements of Section 1.14.4 of ACI 530/ASCE 5/TMS 402 and to the additional requirements of this section.~~

~~**2106.3.1 Masonry walls not part of the lateral-force-resisting system.** Masonry partition walls, masonry screen walls and other masonry elements that are not designed to resist vertical or lateral loads, other than those induced by their own mass, shall be isolated from the structure so that the vertical and lateral forces are not imparted to these elements. Isolation joints and connectors between these elements and the structure shall be designed to accommodate the design story drift.~~

~~**2106.4 Additional requirements for structures in Seismic Design Category C.** Structures assigned to Seismic Design Category C shall conform to the requirements of Section 2106.3, Section 1.14.5 of ACI 530/ASCE 5/TMS 402 and the additional requirements of this section.~~

~~**2106.4.1 Design of discontinuous members that are part of the lateral-force-resisting system.** Columns and pilasters that are part of the lateral-force-resisting system and that support reactions from discontinuous stiff members such as walls shall be provided with transverse reinforcement spaced at no more than one-fourth of the~~

~~least nominal dimension of the column or pilaster. The minimum transverse reinforcement ratio shall be 0.0015. Beams supporting reactions from discontinuous walls or frames shall be provided with transverse reinforcement spaced at no more than one-half of the nominal depth of the beam. The minimum transverse reinforcement ratio shall be 0.0015.~~

~~**2106.5 Additional requirements for structures in Seismic Design Category D.** Structures assigned to Seismic Design Category D shall conform to the requirements of Section 2106.4, Section 1.14.6 of ACI 530/ASCE 5/TMS 402 and the additional requirements of this section.~~

~~**2106.5.1 Loads for shear walls designed by the allowable stress design method.** When calculating in-plane shear or diagonal tension stresses by the allowable stress design method, shear walls that resist seismic forces shall be designed to resist 1.5 times the seismic forces required by Chapter 16. The 1.5 multiplier need not be applied to the overturning moment.~~

~~**2106.5.2 Shear wall shear strength.** For a shear wall whose nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength, two shear regions exist.~~

~~For all cross sections within a region defined by the base of the shear wall and a plane at a distance~~ $\cancel{L_w}$ ~~above the base of the shear wall, the nominal shear strength shall be determined by Equation 21-1.~~

$\cancel{V_n=A_{n\,n} f_y}$ ~~**(Equation 21-1)**~~

~~The required shear strength for this region shall be calculated at a distance~~ $\cancel{L_w/2}$ ~~above the base of the shear wall, but not to exceed one-half story height. For the other region, the nominal shear strength of the shear wall shall be determined from Section 2108.~~

~~**2106.6 Additional requirements for structures in Seismic Design Category E or F.** Structures assigned to Seismic Design Category E or F shall conform to the requirements of Section 2106.5 and Section 1.14.7 of ACI 530/ASCE 5/TMS 402.~~

23. Revise as follows:

2107.1 General. The design of masonry structures using allowable stress design shall comply with Section 2106 and the requirements of Chapters 1 and 2 of ~~ACI 530/ASCE 5/TMS 402~~ <u>TMS 402/ACI 530/ASCE 5</u> except as modified by Sections 2107.2 through ~~2107.8~~ <u>2107.5</u>.

2107.2 ~~ACI 530/ASCE 5/TMS 402~~ <u>TMS 402/ACI 530/ASCE 5</u>, Section 2.1.2, load combinations. Delete Section 2.1.2.1.

24. Delete without substitution:

~~**2107.3 ACI 530/ASCE 5/TMS 402, Section 2.1.3, design strength.** Delete Sections 2.1.3.4 through 2.1.3.4.3.~~

~~**2107.4 ACI 530/ASCE 5/TMS 402, Section 2.1.6, columns.** Add the following text to Section 2.1.6:~~
~~2.1.6.6 Light-frame construction. Masonry columns used only to support light-frame roofs of carports, porches, sheds or similar structures with a maximum area of 450 square feet (41.8~~ $\cancel{m_2}$~~) assigned to Seismic Design Category A, B or Care permitted to be designed and constructed as follows:~~

1. ~~Concrete masonry materials shall be in accordance with Section 2103.1 of the *International Building Code*. Clay or shale masonry units shall be in accordance with Section 2103.2 of the *International Building Code*.~~
2. ~~The nominal cross-sectional dimension of columns shall not be less than 8 inches (203 mm).~~
3. ~~Columns shall be reinforced with not less than one No. 4 bar centered in each cell of the column.~~
4. ~~Columns shall be grouted solid.~~
5. ~~Columns shall not exceed 12 feet (3658 mm) in height.~~
6. ~~Roofs shall be anchored to the columns. Such anchorage shall be capable of resisting the design loads specified in Chapter 16 of the *International Building Code*.~~
7. ~~Where such columns are required to resist uplift loads, the columns shall be anchored to their footings with two No. 4 bars extending a minimum of 24 inches (610 mm) into the columns and bent horizontally a minimum of 15 inches (381 mm) in opposite directions into the footings. One of these bars is permitted to be the reinforcing bar specified in Item 3 above. The total weight of a column and its footing shall not be less than 1.5 times the design uplift load.~~

25. Revise as follows:

~~2107.5~~ 2107.3 ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5, Section 2.1.10.7.1.1, lap splices. Modify Section ~~2.1.10.7.1.1~~ 2.1.9.7.1.1 as follows:

~~2.1.10.7.1.1~~ 2.1.9.7.1.1 The minimum length of lap splices for reinforcing bars in tension or compression, l_d, shall be

$l_d = 0.002 d_b f_s$ **(Equation ~~21-2~~ 21-1)**

For SI: $l_d = 0.29 d_b f_s$

but not less than 12 inches (305 mm). In no case shall the length of the lapped splice be less than 40 bar diameters.

where:

d_b = Diameter of reinforcement, inches (mm).

f_s = Computed stress in reinforcement due to design loads, psi (MPa).

In regions of moment where the design tensile stresses in the reinforcement are greater than 80 percent of the allowable steel tension stress, F_s, the lap length of splices shall be increased not less than 50 percent of the minimum required length. Other equivalent means of stress transfer to accomplish the same 50 percent increase shall be permitted. Where epoxy coated bars are used, lap length shall be increased by 50 percent.

~~2107.6~~ 2107.4 ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5, Section ~~2.1.10.7~~ 2.1.9.7, splices of reinforcement. Modify Section 2.1.10.7 as follows:

~~2.1.10.7~~ 2.1.9.7 Splices of reinforcement. Lap splices, welded splices or mechanical splices are permitted in accordance with the provisions of this section. All welding shall conform to AWS D1.4. Welded splices shall be of ASTM A 706 steel reinforcement. Reinforcement larger than No. 9 (M #29) shall be spliced using mechanical connections in accordance with Section 2.1.10.7.3.

~~2107.7~~ 2107.5 ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5, Section 2.3.6, maximum bar size. Add the following to Chapter 2:

2.3.6 Maximum bar size. The bar diameter shall not exceed one-eighth of the nominal wall thickness and shall not exceed one-quarter of the least dimension of the cell, course or collar joint in which it is placed.

26. Delete without substitution:

~~**2107.8 ACI 530/ASCE 5/TMS 402, Section 2.3.7, maximum reinforcement percentage.** Add the following text to Chapter 2:~~

~~2.3.7 Maximum reinforcement percentage. Special reinforced masonry shear walls having a shear span ratio,~~ ~~*M/Vd*, equal to or greater than 1.0 and having an axial load, *P*, greater than 0.05~~ $\cancel{f'_m A_n}$ ~~that are subjected to in-plane forces shall have a maximum reinforcement ratio,~~ $\cancel{\rho_{max}}$~~, not greater than that computed as follows:~~

$$\cancel{\rho_{\max} = \frac{n f'_m}{2 f_y \left(n + \frac{f_y}{f'_m} \right)}}$$ ~~**(Equation 21-3)**~~

~~The maximum reinforcement ratio does not apply in the out-of-plane direction.~~

27. Revise as follows:

2108.1 General. The design of masonry structures using strength design shall comply with Section 2106 and the requirements of Chapters 1 and 3 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5, except as modified by Sections 2108.2 through 2108.4.

> **Exception:** AAC masonry shall comply with the requirements of Chapter 1 and Appendix A of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

2108.2 ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5, Section 3.3.3.3 development. Add the following text to Section 3.3.3.3:

The required development length of reinforcement shall be determined by Equation (3-15), but shall not be less than 12 inches (305 mm) and need not be greater than 72 d_b.

2108.3 ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5, Section 3.3.3.4, splices. Modify items (b) and (c) of Section 3.3.3.4 as follows:

3.3.3.4 (b). A welded splice shall have the bars butted and welded to develop at least 125 percent of the yield strength, f_y, of the bar in tension or compression, as required. Welded splices shall be of ASTM A 706 steel reinforcement. Welded splices shall not be permitted in plastic hinge zones of intermediate or special reinforced walls or special moment frames of masonry. 3.3.3.4 (c). Mechanical splices shall be classified as Type 1 or 2 according to Section 21.2.6.1 of ACI 318. Type 1 mechanical splices shall not be used within a plastic hinge zone or within a beam-column joint of intermediate or special reinforced masonry shear walls or special moment frames. Type 2 mechanical splices are permitted in any location within a member.

28. Delete without substitution:

~~**2108.4 ACI 530/ASCE 5/TMS 402, Section 3.3.3.5, maximum areas of flexural tensile reinforcement.** Add the following text to Section 3.3.3.5:~~

~~3.3.3.5.5 For special prestressed masonry shear walls, strain in all prestressing steel shall be computed to be compatible with a strain in the extreme tension reinforcement equal to five times the strain associated with the reinforcement yield stress,~~ ~~f_y~~~~. The calculation of the maximum reinforcement shall consider forces in the prestressing steel that correspond to these calculated strains.~~

29. Delete Section 2109 in its entirety and substitute as follows:

SECTION 2109
EMPIRICAL DESIGN OF MASONRY

2109.1 General. Empirically designed masonry shall conform to the requirements of Chapter 5 of TMS 402/ACI 530/ASCE 5, except where otherwise noted in this section.

2109.1.1 Limitations. The use of empirical design of masonry shall be limited as noted in Section 5.1.2 of TMS 402/ACI 530/ASCE 5. In buildings that exceed one or more of the limitations of Section 5.1.2 of TMS 402/ACI 530/ASCE 5, masonry shall be designed in accordance with the engineered design provisions of Section 2101.2.1, 2101.2.2, or 2101.2.3 or the foundation wall provisions of Section 1805.5.

2109.2 Surface-bonded walls. Dry-stacked, surface-bonded concrete masonry walls shall comply with the requirements of Chapter 5 of TMS 402/ACI 530/ASCE 5, except where otherwise noted in this section.

2109.2.1 Strength. Dry-stacked, surface-bonded concrete masonry walls shall be of adequate strength and proportions to support all superimposed loads without exceeding the allowable stresses listed in Table 2109.2.1. Allowable stresses not specified in Table 2109.2.1 shall comply with the requirements of TMs 402/ACI 530/ASCE 5.

TABLE 2109.2.1
ALLOWABLE STRESS GROSS CROSS-SECTIONAL AREA
FOR DRY-STACKED, SURFACE-BONDED CONCRETE MASONRY WALLS

DESCRIPTION	MAXIMUM ALLOWABLE STRESS (psi)
Compression standard block	45
Flexural tension	
Horizontal span	30
Vertical span	18
Shear	10

For SI: 1 pound per square inch = 0.006895MPa.

2109.2.2 Construction. Construction of dry-stacked, surface-bonded masonry walls, including stacking and leveling of units, mixing and application of mortar and curing and protection shall comply with ASTM C 946.

2109.3 Adobe construction. Adobe construction shall comply with this section and shall be subject to the requirements of this code for Type V construction, Chapter 5 of TMS 402/ACI 530/ASCE 5, and this section.

2109.3.1 Unstabilized adobe.

2109.3.1.1 Compressive strength. Adobe units shall have an average compressive strength of 300 psi (2068 kPa) when tested in accordance with ASTM C 67. Five samples shall be tested and no individual unit is permitted to have a compressive strength of less than 250 psi (1724 kPa).

2109.3.1.2 Modulus of rupture. Adobe units shall have an average modulus of rupture of 50 psi (345 kPa) when tested in accordance with the following procedure. Five samples shall be tested and no individual unit shall have a modulus of rupture of less than 35 psi (241 kPa).

2109.3.1.2.1 Support conditions. A cured unit shall be simply supported by 2-inch-diameter (51 mm) cylindrical supports located 2 inches (51 mm) in from each end and extending the full width of the unit.

2109.3.1.2.2 Loading conditions. A 2-inch-diameter (51 mm) cylinder shall be placed at midspan parallel to the supports.

2109.3.1.2.3 Testing procedure. A vertical load shall be applied to the cylinder at the rate of 500 pounds per minute (37 N/s) until failure occurs.

2109.3.1.2.4 Modulus of rupture determination. The modulus of rupture shall be determined by the equation:

$f_r = 3\, PL_s / 2S_w (S_t^2)$ **(Equation 21-2)**

where, for the purposes of this section only:

S_w = Width of the test specimen measured parallel to the loading cylinder, inches (mm).
f_r = Modulus of rupture, psi (MPa).
L_s = Distance between supports, inches (mm).
S_t = Thickness of the test specimen measured parallel to the direction of load, inches (mm).
P = The applied load at failure, pounds (N).

2109.3.1.3 Moisture content requirements. Adobe units shall have a moisture content not exceeding 4 percent by weight.

2109.3.1.4 Shrinkage cracks. Adobe units shall not contain more than three shrinkage cracks and any single shrinkage crack shall not exceed 3 inches (76 mm) in length or $^1/_8$ inch (3.2 mm) in width.

2109.3.2 Stabilized adobe.

2109.3.2.1 Material requirements. Stabilized adobe shall comply with the material requirements of unstabilized adobe in addition to Sections 2109.3.2.1.1 and 2109.3.2.1.2.

2109.3.2.1.1 Soil requirements. Soil used for stabilized adobe units shall be chemically compatible with the stabilizing material.

2109.3.2.1.2 Absorption requirements. A 4-inch (102 mm) cube, cut from a stabilized adobe unit dried to a constant weight in a ventilated oven at 212°F to 239°F (100°C to 115°C), shall not absorb more than $2^1/_2$ percent moisture by weight when placed upon a constantly water-saturated, porous surface for seven days. A minimum of five specimens shall be tested and each specimen shall be cut from a separate unit.

2109.3.3 Allowable stress. The allowable compressive stress based on gross cross-sectional area of adobe shall not exceed 30 psi (207 kPa).

2109.3.3.1 Bolts. Bolt values shall not exceed those set forth in Table 2109.3.3.1.

TABLE 2109.3.3.1
ALLOWABLE SHEAR ON BOLTS IN ADOBE MASONRY

DIAMETER OF BOLTS (inches)	MINIMUM EMBEDMENT (inches)	SHEAR (pounds)
$^{1}/_{2}$	—	—
$^{5}/_{8}$	12	200
$^{3}/_{4}$	15	300
$^{7}/_{8}$	18	400
1	21	500
$1^{1}/_{8}$	24	600

For SI: 1 inch = 25.4 mm, 1 pound = 4.448 N.

2109.3.4 Construction.

2109.3.4.1 General.

2109.3.4.1.1 Height restrictions. Adobe construction shall be limited to buildings not exceeding one story, except that two-story construction is allowed when designed by a registered design professional.

2109.3.4.1.2 Mortar restrictions. Mortar for stabilized adobe units shall comply with Chapter 21 or adobe soil. Adobe soil used as mortar shall comply with material requirements for stabilized adobe. Mortar for unstabilized adobe shall be portland cement mortar.

2109.3.4.1.3 Mortar joints. Adobe units shall be laid with full head and bed joints and in full running bond.

2109.3.4.1.4 Parapet walls. Parapet walls constructed of adobe units shall be waterproofed.

2109.3.4.2 Wall thickness. The minimum thickness of exterior walls in one-story buildings shall be 10 inches (254 mm). The walls shall be laterally supported at intervals not exceeding 24 feet (7315 mm). The minimum thickness of interior load-bearing walls shall be 8 inches (203 mm). In no case shall the unsupported height of any wall constructed of adobe units exceed 10 times the thickness of such wall.

2109.3.4.3 Foundations.

2109.3.4.3.1 Foundation support. Walls and partitions constructed of adobe units shall be supported by foundations or footings that extend not less than 6 inches (152 mm) above adjacent ground surfaces and are constructed of solid masonry (excluding adobe) or concrete. Footings and foundations shall comply with Chapter 18.

2109.3.4.3.2 Lower course requirements. Stabilized adobe units shall be used in adobe walls for the first 4 inches (102 mm) above the finished first-floor elevation.

2109.3.4.4 Isolated piers or columns. Adobe units shall not be used for isolated piers or columns in a load-bearing capacity. Walls less than 24 inches (610 mm) in length shall be considered isolated piers or columns.

2109.3.4.5 Tie beams. Exterior walls and interior load-bearing walls constructed of adobe units shall have a continuous tie beam at the level of the floor or roof bearing and meeting the following requirements.

2109.3.4.5.1 Concrete tie beams. Concrete tie beams shall be a minimum depth of 6 inches (152 mm) and a minimum width of 10 inches (254 mm). Concrete tie beams shall be continuously reinforced with a minimum of two No. 4 reinforcing bars. The ultimate compressive strength of concrete shall be at least 2,500 psi (17.2 MPa) at 28 days.

2109.3.4.5.2 Wood tie beams. Wood tie beams shall be solid or built up of lumber having a minimum nominal thickness of 1 inch (25 mm), and shall have a minimum depth of 6 inches (152 mm) and a minimum width of 10 inches (254 mm). Joints in wood tie beams shall be spliced a minimum of 6 inches (152 mm). No splices shall be allowed within 12 inches (305 mm) of an opening. Wood used in tie beams shall be approved naturally decay-resistant or pressure-treated wood.

2109.3.4.6 Exterior finish. Exterior walls constructed of unstabilized adobe units shall have their exterior surface covered with a minimum of two coats of portland cement plaster having a minimum thickness of 3/4 inch (19.1

mm) and conforming to ASTM C 926. Lathing shall comply with ASTM C 1063. Fasteners shall be spaced at 16 inches (406 mm) o.c. maximum. Exposed wood surfaces shall be treated with an approved wood preservative or other protective coating prior to lath application.

2109.3.4.7 Lintels. Lintels shall be considered structural members and shall be designed in accordance with the applicable provisions of Chapter 16.

30. Delete Section 2110 in its entirety and substitute as follows:

SECTION 2110
GLASS UNIT MASONRY

2110.1 General. Glass unit masonry construction shall comply with Chapter 7 of TMS 402/ACI 530/ASCE 5 and this section.

2110.1.1 Limitations. Solid or hollow approved glass block shall not be used in fire walls, party walls, fire barriers, fire partitions or smoke barriers, or for load-bearing construction. Such blocks shall be erected with mortar and reinforcement in metal channel-type frames, structural frames, masonry or concrete recesses, embedded panel anchors as provided for both exterior and interior walls or other approved joint materials. Wood strip framing shall not be used in walls required to have a fire-resistance rating by other provisions of this code.

Exceptions:

1. Glass-block assemblies having a fire protection rating of not less than 3/4 hour shall be permitted as opening protectives in accordance with Section 715 in fire barriers, fire partitions, and smoke barriers that have a required fire-resistance rating of 1 hour or less and do not enclose exit stairways, exit ramps, or exit passageways.
2. Glass-block assemblies as permitted in Section 404.5, Exception 2.

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

Reason: The primary purpose of this code change proposal is to update the existing reference standards for the design and construction of masonry from the 2005 editions to the 2008 editions. The revisions proposed in this code change, while relatively minor in nature, reflect revisions incorporated into the 2008 edition of the Building Code Requirements for Masonry Structures and Specification for Masonry Structures. This code change proposal is one of several to harmonize the design and construction requirements for masonry within the IBC with those in the reference standard. A complete list of revisions incorporated into the referenced standards is available for download at www.masonrystandards.org.

An editorial change to the titles of ACI 530.1 and ASCE/SEI 6 as shown in Chapter 35 is also proposed.

With the publication of the 2008 edition of the Building Code Requirements for Masonry Structures and Specification for Masonry Structures, The Masonry Society (TMS) has become the lead sponsoring organization of the Masonry Standards Joint Committee (MSJC), which is charged with reviewing and maintaining the provisions in the referenced standards. As such, the official designation of these standards has changed from ACI 530/ASCE 5/TMS 402 to TMS 402/ACI 530/ASCE 5 and from ACI 530.1/ASCE 6/TMS 602 to TMS 602/ACI 530.1/ASCE 6 as reflected in the above proposed modifications.

The additional information proposed to be included in Section 2101.3 simply reflect revisions to the reference standards regarding the minimum information to be included on the construction documents. The proposed additions are consistent in intent with existing requirements in Chapters 1 and 16 as well as other material chapters.

The remainder of the changes proposed herein remove provisions transcribed into the IBC from the MSJC standards. While this series of changes may appear substantive, it is instead intended to be editorial as the provisions proposed for removal are already contained in the reference standards. For several years the ICC has been moving toward removing provisions transcribed into the body of the I-Codes, and where applicable, referencing a governing standard. This change proposes just that, with the one exception related to Section 2105. The provisions of this section are retained for the benefit of inspectors and regulators.

Some have voiced a concern in the past that by removing provisions from the IBC it forces them to purchase numerous reference standards. In the case of the masonry design and construction provisions, this statement would be true regardless of whether the transcribed provisions proposed to be removed by this change proposal were accepted or not as only a very small fraction of the MSJC provisions are transcribed. It can be argued that maintaining the transcribed provisions in the current form is a disservice to the safety and welfare of the general public; who may assume or are told that this Chapter contains to complete body of information related to masonry construction, materials, and design. Whether one assumes the role of contractor, designer, building official, inspector, plan checker, or specifier, they still must have the reference document to perform their tasks.

Cost Impact: The code change proposal will not increase the cost of construction.

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

Individual Consideration Agenda

This item is on the agenda for individual consideration because public comments were submitted.

Public Comment 1:

Jim McClintic, Sandy City, representing Utah Chapter of ICC, requests Disapproval.

Commenter's Reason: This is the same basic proposal that was voted down in Rochester as a show of force to stop a disturbing trend to eliminate basic construction code requirements from the IBC. Moving such requirements to reference standards leaves ICC members with little or no control over the outcome of any changes within each standard. Many smaller jurisdictions do not have the resources to purchase all of the new standards referenced in the IBC. Taking tools out of the toolbox and putting them in a separate place with a new price to use them doesn't seem to be consistent with the reason ICC exists. Don't let the IBC become a reference manual for standards. This code is for all of us to use and help develop.

Public Comment 2:

Richard vonWeller, Park City, representing Utah Chapter of ICC, requests Disapproval.

Commenter's Reason: This comment to E175 and items S182, S184, S188, and S190 all are related to the same general issue which was heavily debated disapproved at the Rochester final action hearings. Several are actually identical to those previously defeated. These are prime examples of a disturbing trend in the IBC to eliminate basic construction code requirements from the IBC. There has been a wholesale effort to take critical elements out of our base code and move them to reference standards where the voting members of ICC have little or no control over the outcome of any changes. In the reason for S190 the proponent is wanting to remove numerous provisions from the ICC process thereby "reducing the burdens on code officials in overseeing often esoteric, technical or specialized provisions". Please don't do code officials any such favor.

Once a requirement moves out of ICC's system what hope is there we will regain our opportunity to effect positive change in the public interest? The very reason ICC exists is because of our efforts to bring together of all the stakeholders in a single forum to discuss, debate and decide in a process where the outcome is determined by those with no financial interest.

The proponent agues the technical requirements are still incorporated through the reference standard, but the practicality of use and enforcement becomes extremely cumbersome, costly and less effective. Needed information must be readily available to enforcement personnel. Is it really a good idea to eliminate the basic requirements for grout, reinforcement, corrosion protection, mortar, wall ties, cold and hot weather protection, seismic resistance, and columns for light frame construction from the IBC? ICC does not even test on the reference standards for certifications. Shouldn't a certified inspector or plans examiner be able to answer simple questions about these subjects?

The voting members must take a stand and disapprove these changes to ensure the IBC is a useful tool for code officials to help protect the health, safety and welfare of our citizens.

Final Action: AS AM AMPC____ D

S179-07/08
Table 2103.8(2)

Proposed Change as Submitted:

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards

Revise table as follows:

TABLE 2103.8(2)
MORTAR PROPERTIES[a]

For SI: 1 inch = 25.4 mm, 1 pound per square inch = 6.895kPa.

a. This aggregate ratio (measured in damp, loose condition) shall not be less than 2¼ and not more than ~~3~~ 3½ times the sum of the separate volumes of cementitious materials.

(Portions of table and footnotes not shown remain unchanged)

Reason: Footnote a in Table 2103.8(2) of the IBC is incorrect. The proposed revision provides the correct proportioning requirements for masonry sand and exactly matches the requirements contained in ASTM C 270, Standard Specification for Mortar for Unit Masonry, as was originally intended.

Cost Impact: The code change proposal will not increase the cost of construction.

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**

Individual Consideration Agenda

This item is on the agenda for individual consideration because a public comment was submitted.

Public Comment:

Phil Samblanet, The Masonry Society, and Jason Thompson, National Concrete Masonry Association, representing The Masonry Alliance for Codes and Standards, request Approval as Submitted.

Commenter's Reason: If code change proposal S175-07/08 is ultimately approved as modified as recommended by the committee, this change proposal becomes redundant and unnecessary.

Final Action: AS AM AMPC____ D

S182-07/08
2104.1

Proposed Change as Submitted:

Proponent: Phillip J. Samblanet, The Masonry Society

1. Revise as follows:

2104.1.2 Placing mortar and units. Placement of ~~mortar and clay and concrete~~ units and mortar shall comply with TMS 602/ACI 530.1/ASCE 6 and Section 2104.1.2.1 ~~Sections 2104.1.2.1, 2104.1.2.2, 2104.1.2.3 and 2104.1.2.5. Placement of mortar and glass unit masonry shall comply with Sections 2104.1.2.4 and 2104.1.2.5. Placement of thin-bed mortar and AAC masonry shall comply with Section 2104.1.2.6~~.

~~**2104.1.2.1 Bed and head joints.** Unless otherwise required or indicated on the construction documents, head and bed joints shall be 3/8 inch (9.5 mm) thick, except that the thickness of the bed joint of the starting course placed over foundations shall not be less than 1/4 inch (6.4 mm) and not more than 3/4 inch (19.1 mm).~~

2104.1.2.1~~.1~~ Open-end units. Open-end units with beveled ends shall be fully grouted. Head joints of open-end units with beveled ends need not be mortared. The beveled ends shall form a grout key that permits grouts within 5/8 inch (15.9 mm) of the face of the unit. The units shall be tightly butted to prevent leakage of the grout.

2. Delete without substitution:

~~**2104.1.2.2 Hollow units.** Hollow units shall be placed such that face shells of bed joints are fully mortared. Webs shall be fully mortared in all courses of piers, columns, pilasters, in the starting course on foundations where adjacent cells or cavities are to be grouted, and where otherwise required. Head joints shall be mortared a minimum distance from each face equal to the face shell thickness of the unit.~~

~~**2104.1.2.3 Solid units.** Unless otherwise required or indicated on the construction documents, solid units shall be placed in fully mortared bed and head joints. The ends of the units shall be completely buttered. Head joints shall not be filled by slushing with mortar. Head joints shall be constructed by shoving mortar tight against the adjoining unit. Bed joints shall not be furrowed deep enough to produce voids.~~

~~**2104.1.2.4 Glass unit masonry.** Glass units shall be placed so head and bed joints are filled solidly. Mortar shall not be furrowed. Unless otherwise required, head and bed joints of glass unit masonry shall be 1/4 inch (6.4 mm) thick, except that vertical joint thickness of radial panels shall not be less than 1/8 inch (3.2 mm). The bed joint thickness tolerance shall be minus 1/16 inch (1.6 mm) and plus 1/8 inch (3.2 mm). The head joint thickness tolerance shall be plus or minus 1/8 inch (3.2 mm).~~

~~**2104.1.2.5 Placement in mortar.** Units shall be placed while the mortar is soft and plastic. Any unit disturbed to the extent that the initial bond is broken after initial positioning shall be removed and relaid in fresh mortar.~~

~~**2104.1.2.6 Thin-bed mortar and AAC masonry units.** AAC masonry construction shall begin with a leveling course of masonry meeting the requirements of Section 2104.1.2. Subsequent courses of AAC masonry units shall be laid with thin-bed mortar using a special notched trowel manufactured for use with thin-bed mortar to spread the mortar so that it completely fills the bed joints. Unless otherwise specified, the head joints shall be similarly filled. Joints inAAC masonry shall be approximately 1/16 inch (1.5 mm) and shall be formed by striking on the ends and tops of AAC masonry units with a rubber mallet. Minor adjustments in unit position shall be made while the mortar is still soft and plastic by tapping it into the proper position. Minor sanding of the exposed faces of AAC masonry shall be permitted to provide a smooth and plumb surface.~~

~~**2104.1.2.7 Grouted masonry.** Between grout pours, a horizontal construction joint shall be formed by stopping all wythes at the same elevation and with the grout stopping a minimum of 11/2 inches (38 mm) below a mortar joint, except at the top of the wall. Where bond beams occur, the grout pour shall be stopped a minimum of 1/2 inch (12.7 mm) below the top of the masonry.~~

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

Reason: The provisions in Section 2104.1.2 contain some, but not all requirements for placement of units and mortar (for instance, requirements for filling collar joints are not included in the IBC). As such, a reference is needed to TMS 602/ACI 530.1/ASCE 6 in keeping with Section 2104.1.1.

In considering the entire section however, it seems reasonable to only show the modifications to the reference standard similar to what is done in Sections 2107 and 2108. As such, sections 2104.1.2.1, 2104.1.2.2, 2104.1.2.3, 2104.1.2.4, 2104.1.2.5, 2104.1.2.6, and 2107.1.2.7 are proposed to be deleted, while Section 2102.1.1.1 is proposed to be left in because it modifies the referenced standard. This will simplify things greatly for the inspector and building official as they can quickly see what is required and what the IBC modifications are.

Cost Impact: The code change proposal will not increase the cost of construction.

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**

Individual Consideration Agenda

This item is on the agenda for individual consideration because public comments were submitted.

Public Comment 1:

Phil Samblanet and Jason Thompson, The Masonry Society and National Concrete Masonry Association, representing The Masonry Society and The Masonry Alliance for Codes and Standards, requests Approval as Modified by the Code Committee as Published in the ROH

Commenter's Reason If code change proposal S175-07/08 is ultimately approved as modified as recommended by the committee, this change proposal becomes redundant and unnecessary:

Public Comment 2:

Richard vonWeller, Park City, representing Utah Chapter of ICC, requests Disapproval.

Commenter's Reason: This comment to E175 and items S182, S184, S188, and S190 all are related to the same general issue which was heavily debated disapproved at the Rochester final action hearings. Several are actually identical to those previously defeated. These are prime examples of a disturbing trend in the IBC to eliminate basic construction code requirements from the IBC. There has been a wholesale effort to take critical elements out of our base code and move them to reference standards where the voting members of ICC have little or no control over the outcome of any changes. In the reason for S190 the proponent is wanting to remove numerous provisions from the ICC process thereby "reducing the burdens on code officials in overseeing often esoteric, technical or specialized provisions". Please don't do code officials any such favor.

Once a requirement moves out of ICC's system what hope is there we will regain our opportunity to effect positive change in the public interest? The very reason ICC exists is because of our efforts to bring together of all the stakeholders in a single forum to discuss, debate and decide in a process where the outcome is determined by those with no financial interest.

The proponent agues the technical requirements are still incorporated through the reference standard, but the practicality of use and enforcement becomes extremely cumbersome, costly and less effective. Needed information must be readily available to enforcement personnel. Is it really a good idea to eliminate the basic requirements for grout, reinforcement, corrosion protection, mortar, wall ties, cold and hot weather protection, seismic resistance, and columns for light frame construction from the IBC? ICC does not even test on the reference standards for certifications. Shouldn't a certified inspector or plans examiner be able to answer simple questions about these subjects?

The voting members must take a stand and disapprove these changes to ensure the IBC is a useful tool for code officials to help protect the health, safety and welfare of our citizens.

Public Comment 3:

Can Xiao, City of Phoenix, AZ, representing herself, requests Disapproval.

Commenter's Reason: There is very limited information in ACI 530.1/ASCE 6/TMS 602 about masonry construction. It is important to still have Section 2104 in IBC code.

Final Action: AS AM AMPC____ D

S184-07/08
2102, 2104.3, 2104.4, Table 1704.5.1, Table 1704.5.3

Proposed Change as Submitted:

Proponent: Phillip J. Samblanet, The Masonry Society

1. Delete without substitution:

SECTION 2102
DEFINITIONS AND NOTATIONS

~~**MEAN DAILY TEMPERATURE.** The average daily temperature of temperature extremes predicted by a local weather bureau for the next 24 hours.~~

~~**2104.3 Cold weather construction.** The cold weather construction provisions of ACI 530.1/ASCE 6/TMS 602, Article 1.8 C, or the following procedures shall be implemented when either the ambient temperature falls below 40°F (4°C) or the temperature of masonry units is below 40°F (4°C).~~

~~**2104.3.1 Preparation.** 1. Temperatures of masonry units shall not be less than 20°F (-7°C) when laid in the masonry. Masonry units containing frozen moisture, visible ice or snow on their surface shall not be laid. 2. Visible ice and snow shall be removed from the top surface of existing foundations and masonry to receive new construction. These surfaces shall be heated to above freezing, using methods that do not result in damage.~~

~~**2104.3.2 Construction.** The following requirements shall apply to work in progress and shall be based on ambient temperature.~~

~~**2104.3.2.1Construction requirements for temperatures between 40°F (4°C) and 32°F (0°C).** The following construction requirements shall be met when the ambient temperature is between 40°F (4°C) and 32°F (0°C): 1. Glass unit masonry shall not be laid. 2. Water and aggregates used in mortar and grout shall not be heated above 140°F (60°C). 3. Mortar sand or mixing water shall be heated to produce mortar temperatures between 40°F (4°C) and 120°F (49°C) at the time of mixing. When water and aggregates for grout are below 32°F(0°C), they shall be heated.~~

~~**2104.3.2.2 Construction requirements for temperatures between 32°F (0°C) and 25°F (-4°C).** The requirements of Section 2104.3.2.1 and the following construction requirements shall be met when the ambient temperature is between 32°F (0°C) and 25°F (-4°C): 1. The mortar temperature shall be maintained above freezing until used in masonry. 2. Aggregates and mixing water for grout shall be heated to produce grout temperature between 70°F (21°C) and 120°F (49°C) at the time of mixing. Grout temperature shall be maintained above 70°F (21°C) at the time of grout placement. 3. Heat AAC masonry units to a minimum temperature of 40°F (4°C) before installing thin-bed mortar.~~

~~**2104.3.2.3 Construction requirements for temperatures between 25°F (-4°C) and 20°F (-7°C).** The requirements of Sections 2104.3.2.1 and 2104.3.2.2 and the following construction requirements shall be met when the ambient temperature is between 25°F (-4°C) and 20°F (-7°C): 1. Masonry surfaces under construction shall be heated to 40°F (4°C). 2. Wind breaks or enclosures shall be provided when the wind velocity exceeds 15 miles per hour (mph) (24 km/h). 3. Prior to grouting, masonry shall be heated to a minimum of 40°F (4°C).~~

~~**2104.3.2.4 Construction requirements for temperatures below 20°F (-7°C).** The requirements of Sections 2104.3.2.1, 2104.3.2.2 and 2104.3.2.3 and the following construction requirement shall be met when the ambient temperature is below 20°F (-7°C): Enclosures and auxiliary heat shall be provided to maintain air temperature within the enclosure to above 32°F (0°C).~~

~~**2104.3.3 Protection.** The requirements of this section and Sections 2104.3.3.1 through 2104.3.3.5 apply after the masonry is placed and shall be based on anticipated minimum daily temperature for grouted masonry and anticipated mean daily temperature for ungrouted masonry.~~

~~**2104.3.3.1 Glass unit masonry.** The temperature of glass unit masonry shall be maintained above 40°F (4°C) for 48 hours after construction.~~

~~**2104.3.3.2 AAC masonry.** The temperature of AAC masonry shall be maintained above 32°F (0°C) for the first 4 hours after thin-bed mortar application.~~

~~**2104.3.3.3 Protection requirements for temperatures between 40°F (4°C) and 25°F (-4°C).** When the temperature is between 40°F (4°C) and 25°F (-4°C), newly constructed masonry shall be covered with a weather-resistive membrane for 24 hours after being completed.~~

~~**2104.3.3.4 Protection requirements for temperatures between 25°F (-4°C) and 20°F (-7°C).** When the temperature is between 25°F (-4°C) and 20°F (-7°C), newly constructed masonry shall be completely covered with weather-resistive insulating blankets, or equal protection, for 24 hours after being completed. The time period shall be extended to 48 hours for grouted masonry, unless the only cement in the grout is Type III Portland cement.~~

~~**2104.3.3.5 Protection requirements for temperatures below20°F (-7°C).** When the temperature is below20°F (-7°C), newly constructed masonry shall be maintained at a temperature above 32°F (0°C) for at least 24 hours after being completed by using heated enclosures, electric heating blankets, infrared lamps or other acceptable methods. The time period shall be extended to 48 hour for grouted masonry, unless the only cement in the grout is Type III portland cement.~~

~~**2104.4 Hot weather construction.** The hot weather construction provisions of ACI 530.1/ASCE 6/TMS 602, Article 1.8 D, or the following procedures shall be implemented when the temperature or the temperature and wind-velocity limits of this section are exceeded.~~

~~**2104.4.1 Preparation.** The following requirements shall be met prior to conducting masonry work.~~

~~**2104.4.1.1 Temperature.** When the ambient temperature exceeds 100°F (38°C), or exceeds 90°F (32°C) with a wind velocity greater than 8 mph (3.5 m/s): 1. Necessary conditions and equipment shall be provided to produce mortar having a temperature below 120°F (49°C). 2. Sand piles shall be maintained in a damp, loose condition.~~

~~**2104.4.1.2 Special conditions.** When the ambient temperature exceeds 115°F (46°C), or 105°F (40°C) with a wind velocity greater than 8 mph (3.5 m/s), the requirements of Section 2104.4.1.1 shall be implemented, and materials and mixing equipment shall be shaded from direct sunlight.~~

~~**2104.4.2 Construction.** The following requirements shall be met while masonry work is in progress.~~

~~**2104.4.2.1 Temperature.** When the ambient temperature exceeds 100°F (38°C), or exceeds 90°F (32°C) with a wind velocity greater than 8 mph (3.5 m/s):~~

1. ~~The temperature of mortar and grout shall be maintained below 120°F (49°C).~~
2. ~~Mixers, mortar transport containers and mortar boards shall be flushed with cool water before they come into contact with mortar ingredients or mortar.~~
3. ~~Mortar consistency shall be maintained by retempering with cool water.~~
4. ~~Mortar shall be used within 2 hours of initial mixing.~~
5. ~~Thin-bed mortar shall be spread no more than 4 feet (1219 mm) ahead of AAC masonry units.~~
6. ~~AAC masonry units shall be placed within one minute after spreading thin-bed mortar.~~

~~**2104.4.2.2 Special conditions.** When the ambient temperature exceeds 115°F (46°C), or exceeds 105°F (40°C) with a wind velocity greater than 8 mph (3.5 m/s), the requirements of Section 2104.4.2.1 shall be implemented and cool mixing water shall be used for mortar and grout. The use of ice shall be permitted in the mixing water prior to use. Ice shall not be permitted in the mixing water when added to the other mortar or grout materials.~~

~~**2104.4.3 Protection.** When the mean daily temperature exceeds 100°F (38°C) or exceeds 90°F (32°C) with a wind velocity greater than 8 mph (3.5 m/s), newly constructed masonry shall be fog sprayed until damp at least three times a day until the masonry is three days old.~~

~~**2104.5 Wetting of brick.** Brick (clay or shale) at the time of laying shall require wetting if the unit's initial rate of water absorption exceeds 30 grams per 30 square inches (19 355 mm2) per minute or 0.035 ounce per square inch (1 g/645mm^2) per minute, as determined by ASTM C 67.~~

TABLE 1704.5.1
LEVEL 1 SPECIAL INSPECTION

INSPECTION TASK	FREQUENCY OF INSPECTION		REFERENCE FOR CRITERIA		
	Continuous during task listed	Periodically during task listed	IBC section	ACI 530/ASCE 5/TMS 402a	ACI 530.1/ASCE 6/TMS 602a
1. As masonry construction begins, the following shall be verified to ensure compliance:					
a. Proportions of site-prepared mortar.	—	X	—	—	Art. 2.6A
b. Construction of mortar joints.	—	X	—	—	Art. 3.3B
c. Location of reinforcement, connectors, prestressing tendons and anchorages.	—	X	—	—	Art. 3.4, 3.6A
d. Prestressing technique.	—	X	—	—	Art. 3.6B
e. Grade and size of prestressing tendons and anchorages.	—	X	—	—	Art. 2.4B, 2.4H
2. The inspection program shall verify:					
a. Size and location of structural elements.	—	X	—	—	Art. 3.3G
b. Type, size and location of anchors, including other details of anchorage of masonry to structural members, frames or other construction.	—	X	—	Sec. 1.2.2(e), 2.1.4, 3.1.6	—
c. Specified size, grade and type of reinforcement.	—	X	—	Sec. 1.13	Art. 2.4, 3.4
d. Welding of reinforcing bars.	X	—	—	Sec. 2.1.10.7.2, 3.3.3.4(b)	—
e. Protection of masonry during cold weather (temperature below 40°F) or hot weather (temperature above 90°F).	—	X	~~Sec. 2104.3, 2104.4~~	—	Art. 1.8C, 1.8D
f. Application and measurement of prestressing force.	—	X	—	—	Art. 3.6B
3. Prior to grouting, the following shall be verified to ensure compliance:					
a. Grout space is clean.	—	X	—	—	Art. 3.2D
b. Placement of reinforcement and connectors and prestressing tendons and anchorages.	—	X	—	Sec. 1.13	Art. 3.4
c. Proportions of site-prepared grout and prestressing grout for bonded tendons.	—	X	—	—	Art. 2.6B
d. Construction of mortar joints.	—	X	—	—	Art. 3.3B
4. Grout placement shall be verified to ensure compliance with code and construction document provisions.	X	—	—	—	Art 3.5
a. Grouting of prestressing bonded tendons.	X	—	—	—	Art. 3.6C
5. Preparation of any required grout specimens, mortar specimens and/or prisms shall be observed.	X	—	Sec. 2105.2.2, 2105.3	—	Art. 1.4
6. Compliance with required inspection provisions of the construction documents and the approved submittals shall be verified.	—	X	—	—	Art. 1.5

(Portions of table not shown remain unchanged)

TABLE 1704.5.3
LEVEL 2 SPECIAL INSPECTION

INSPECTION TASK	FREQUENCY OF INSPECTION		REFERENCE FOR CRITERIA		
	Continuous during task listed	Periodically during task listed	IBC section	ACI 530/ ASCE 5/ TMS 402a	ACI 530.1/ ASCE 6/ TMS 602a
1. From the beginning of masonry construction, 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	—	Sec. 1.13	Art. 3.4, 3.6A
d. Grout space prior to grouting.	X	—	—	—	Art. 3.2D
e. Placement of grout.	X	—	—	—	Art. 3.5
f. Placement of prestressing grout.	X	—	—	—	Art. 3.6C
2. The inspection program shall verify:					
a. Size and location of structural elements.	—	X	—	—	Art. 3.3G
b. Type, size and location of anchors, including other details of anchorage of masonry to structural members, frames or other construction.	X	—	—	Sec. 1.2.2(e), 2.1.4, 3.1.6	—
c. Specified size, grade and type of reinforcement.		X	—	Sec. 1.13	Art. 2.4, 3.4
d. Welding of reinforcing bars.	X	—	—	Sec. 2.1.10.7.2, 3.3.3.4(b)	—
e. Protection of masonry during cold weather (temperature below 40°F) or hot weather (temperature above 90°F).	—	X	~~Sec. 2104.3, 2104.4~~	—	Art. 1.8C, 1.8D
f. Application and measurement of prestressing force.	X	—	—	—	Art. 3.6B
3. Preparation of any required grout specimens, mortar specimens and/or prisms shall be observed.	X	—	Sec. 2105.2.2, 2105.3	—	Art. 1.4
4. Compliance with required inspection provisions of the construction documents and the approved submittals shall be verified.	—	X	—	—	Art. 1.5

(Portions of table not shown remain unchanged)

Add updates to referenced standards:

ACI
ACI 530.1-~~05~~<u>08</u> Specification~~s~~ for Masonry Structures

ASCE/SEI
ASCE/SEI 6-~~05~~<u>08</u> Specification~~s~~ for Masonry Structures

TMS
TMS 602-~~05~~<u>08</u> Specification for Masonry Structures

Reason: Section 2104 of the IBC is increasingly causing confusion and aggravation among contractors, inspectors and building officials because the leading section (2104.1) requires compliance with Section 2104 of the IBC AND the ACI 530.1/ASCE 6/TMS 602. The intent of this is appropriate because the referenced standard contains most of the needed construction requirements and then Section 2104 contains additional requirements for systems that are included in the IBC but not the referenced standard. However, this Section 2104 does not just add additional requirements, but it also contains some transcribed provisions from the referenced standard. As such, contractors, inspectors and building officials struggle with having to read similar and essentially identical requirements in both the referenced standard and the IBC to try to determine what are the differences (none for the sections proposed to be deleted in this section) between these requirements. There is no need to make them do this, and keeping the transcription only causes confusion, aggravation, and the potential for future conflict if the IBC is not kept updated with the referenced standard. This change simplifies the provisions for users while eliminating the risk of that IBC and the referenced standard become out of phase.

This proposal is essentially identical to a proposal submitted during the last supplement cycle for the IBC, which was recommended for approval by the IBC Structural Subcommittee, and which was narrowly overturned on the floor in Rochester. It is being brought back for reconsideration because as noted the change greatly simplifies the Code for contractors, building officials, and inspectors. Opposition to the change was procedural (globally), not technical. The primary opposition related to concerns about taking critical provisions out of the I-Codes. While this is a valid concern, and while the proponents share the goal to have building officials involved in the development of all provisions in, or referenced by the I-Codes, the use of referenced standards are nevertheless widely used throughout the I-Codes for many good reasons (including reducing burdens on code officials in overseeing often esoteric, technical, or specialized provisions and instead rely on consensus forums to develop such provisions with experts on a balanced committee that properly considers all comments related to the provisions).

This change is proposed again with the hopes that those good reasons are still supported and with the intent of simplifying the Code for contractors, building officials, inspectors and designers. The proposed revisions will remove duplicate provisions, thus simplifying the IBC and reducing the chance that the provisions in the IBC and the referenced standard vary unnecessarily.

To the best of the proponents' knowledge, this change has no technical impact.

One final clarification. During the Rochester hearings, opposition to this change noted concerns with the high cost of having to purchase numerous referenced standards to be able to effectively use the IBC. The proponents share this concern, and note that this proposed change does not require an additional reference because Section 2104.1 already requires the referenced standard to determine appropriateness of tolerances, placement procedures, and other aspects of masonry construction. Thus this change will not increase the cost of construction because it is simply removing redundant provisions), nor does it increase the cost of references that contractors, inspectors, building officials, and designers need.

We are hopeful that this change will be approved as it will simplify and streamline the code, making application easier for contractors, building officials, inspectors and inspectors.

Cost Impact: The code change proposal will not increase the cost of construction.

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

Individual Consideration Agenda

This item is on the agenda for individual consideration because public comments were submitted.

Public Comment 1:

Phil Samblanet and Jason Thompson, The Masonry Society and National Concrete Masonry Association, representing The Masonry Society and The Masonry Alliance for Codes and Standards, request Approval as Submitted.

Commenter's Reason: If code change proposal S175-07/08 is ultimately approved as modified as recommended by the committee, this change proposal becomes redundant and unnecessary.

Public Comment 2:

Richard vonWeller, Park City, representing Utah Chapter of ICC, requests Disapproval.

Commenter's Reason: This comment to E175 and items S182, S184, S188, and S190 all are related to the same general issue which was heavily debated disapproved at the Rochester final action hearings. Several are actually identical to those previously defeated. These are prime examples of a disturbing trend in the IBC to eliminate basic construction code requirements from the IBC. There has been a wholesale effort to take critical elements out of our base code and move them to reference standards where the voting members of ICC have little or no control over the outcome of any changes. In the reason for S190 the proponent is wanting to remove numerous provisions from the ICC process thereby "reducing the burdens on code officials in overseeing often esoteric, technical or specialized provisions". Please don't do code officials any such favor.

Once a requirement moves out of ICC's system what hope is there we will regain our opportunity to effect positive change in the public interest? The very reason ICC exists is because of our efforts to bring together of all the stakeholders in a single forum to discuss, debate and decide in a process where the outcome is determined by those with no financial interest.

The proponent agues the technical requirements are still incorporated through the reference standard, but the practicality of use and enforcement becomes extremely cumbersome, costly and less effective. Needed information must be readily available to enforcement personnel. Is it really a good idea to eliminate the basic requirements for grout, reinforcement, corrosion protection, mortar, wall ties, cold and hot weather protection, seismic resistance, and columns for light frame construction from the IBC? ICC does not even test on the reference standards for certifications. Shouldn't a certified inspector or plans examiner be able to answer simple questions about these subjects?

The voting members must take a stand and disapprove these changes to ensure the IBC is a useful tool for code officials to help protect the health, safety and welfare of our citizens.

Public Comment 3:

Can Xiao, City of Phoenix, AZ, representing herself, requests Disapproval.

Commenter's Reason: The similar information was not provided in ACI 530.1/ASCE 6/TMS 602. It is important to still have that information stay in the IBC.

Final Action: AS AM AMPC____ D

S186-07/08
2106, 2102

Proposed Change as Submitted:

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards

1. Revise as follows:

2106.1 Seismic design requirements for masonry. Masonry structures and components shall comply with the requirements in Section 1.<u>17</u>~~14.2.2~~ ~~and Section 1.14.3, 1.14.4, 1.14.5, 1.14.6 or 1.14.7~~ of ~~ACI 530/ASCE 5/~~<u>TMS 402/ACI 530/ASCE 5</u> 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.

Delete Section 2106.1.1 Basic seismic-force-resisting system through Section 2106.6 Additional requirements for structures in Seismic Design Category E or F without substitution.

2. Delete without substitution:

SECTION 2102
DEFINITIONS AND NOTATIONS

NOTATIONS.

~~*Lw* = Length of wall, inches (mm).~~
~~*Vn* = Nominal shear strength, pounds (N).~~
~~ρ*n* = Ratio of distributed shear reinforcement on plane perpendicular to plane of *Amv*.~~

Add updates to referenced standards as follows:

ACI
ACI 530-~~05~~<u>08</u> Building Code Requirements for Masonry Structures

ASCE/SEI
ASCE/SEI 5-~~05~~<u>08</u> Building Code Requirements for Masonry Structures

TMS
TMS 402-~~05~~<u>08</u> Building Code Requirements for Masonry Structures

Reason: The revisions proposed in this code change reflect editorial and substantive revisions incorporated into the 2008 edition of the Building Code Requirements for Masonry Structures (TMS 402/ACI 530/ASCE 5), commonly referred to as the Masonry Standard Joint Committee (MSJC) Code. This code change proposal is one of several to harmonize the design and construction requirements for masonry within the IBC with those in the reference standard. A complete list of revisions incorporated into the reference standard is available for download at www.masonrystandards.org.

While on the surface this change may appear quite substantive, it actually employs little technical change. Over the course of the 2008 update cycle for the MSJC the entire seismic design and detailing requirements for masonry were rewritten for clarity and the modifications included in Section 2106 were reviewed and their intent incorporated into the MSJC.

The deletion of notations for L_w, V_n, and ρ_n are proposed as well. With the removal of Equation 21-1 currently in Section 2106, these terms are no longer used in Chapter 21.

Through the consolidation of these requirements into a single location, the use and enforcement of these provisions will be simplified.

Cost Impact: The code change proposal will not increase the cost of construction.

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**

Individual Consideration Agenda

This item is on the agenda for individual consideration because public comments were submitted.

Public Comment 1:

Phil Samblanet and Jason Thompson, The Masonry Society and National Concrete Masonry Association, representing The Masonry Society and The Masonry Alliance for Codes and Standards, request Approval as Modified by the Code Committee as Published in the ROH.

Commenter's Reason: If code change proposal S175-07/08 is ultimately approved as modified as recommended by the committee, this change proposal becomes redundant and unnecessary.

Public Comment 2:

Can Xiao, City of Phoenix, Arizona, AZ, representing herself, requests Disapproval.

Commenter's Reason: There are detailing requirements and shear wall strength analysis information provided in IBC Section 2106 for seismic category C and D that are not covered under ACI 530/ASCE 5-08/TMS 402**.**

Final Action: AS AM AMPC____ D

S187-07/08

2102, 2106.1 through 2106.1.1.3 through 2106.4, 2106.5 through 2106.6

Proposed Change as Submitted:

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards

Revise as follows:

SECTION 2102
DEFINITIONS AND NOTATIONS

NOTATIONS

M_n = nominal moment strength, in.-lb (N-mm)

~~L_w = Length of wall, inches (mm).~~

~~ρ_n = Ratio of distributed shear reinforcement on plane perpendicular to plane of A_{mv}.~~

2106.1 Seismic design requirements for masonry. Masonry structures and components shall comply with the requirements in Section ~~1.14.2.2~~ 1.17.2 and 1.17.3 and Section ~~1.14.3~~ 1.17.4.1, ~~1.14.4~~ 1.17.4.2, ~~1.14.5~~ 1.17.4.3, ~~1.14.6~~ 1.17.4.4 or ~~1.14.7~~ 1.17.4.5 of ~~ACI 530/ASCE 5/TMS 402~~ 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 ~~1.14.2.2~~ 1.17.3.2 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 or with Section 2106.1.1.1, 2106.1.1.2 or 2106.1.1.3.

2106.1.1.1 Ordinary plain prestressed masonry shear walls. Ordinary plain prestressed masonry shear walls shall comply with the requirements of Chapter 4 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

2106.1.1.2 Intermediate prestressed masonry shear walls. Intermediate prestressed masonry shear walls shall comply with the requirements of Section ~~1.14.2.2.4~~ 1.17.3.2.11 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 and shall be designed by Chapter 4, Section 4.4.3, of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 for flexural strength and by Section 3.3.4.1.2 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 for shear strength. Sections ~~1.14.2.2.5~~ 1.17.3.2.6(a), 1.17.3.2.6(b), 3.3.3.5 or 3.3.6.5, and ~~3.3.4.3.2(c)~~ 3.3.4.2.3(c) of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 shall be applicable for reinforcement. Flexural elements subjected to load reversals shall be symmetrically reinforced. The nominal moment strength at any section along a member shall not be less than one-fourth the maximum moment strength. The cross-sectional area of bonded tendons shall be considered to contribute to the minimum reinforcement in Section ~~1.14.2.2.4~~ 1.17.3.2.3.1, 1.17.3.2.6(a), and 1.17.3.2.6(b) of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5. Tendons shall be located in cells that are grouted the full height of the wall.

2106.1.1.3 Special prestressed masonry shear walls. Special prestressed masonry shear walls shall comply with the requirements of Section ~~1.14.2.2.5~~ 1.17.3.2.12 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 and shall be designed by Chapter 4, Section 4.4.3, of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 for flexural strength and by Section ~~3.3.4.1.2~~ 4.6 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 for shear strength. Sections ~~1.14.2.2.5(a)~~ 1.17.3.2.6(a), 1.17.3.2.6(b), 3.3.3.5 or 3.3.6.5, and ~~3.3.4.3.2(c)~~ 3.3.4.2.3(c) of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 shall be applicable for reinforcement. Flexural elements subjected to load reversals shall be symmetrically reinforced. The nominal moment strength at any section along a member shall not be less than one-fourth the maximum moment strength. The cross-sectional area of bonded tendons shall be considered to contribute to the minimum reinforcement in Section ~~1.14.2.2.5~~ 1.17.3.2.3.1, 1.17.3.2.6(a), and 1.17.3.2.6(b) of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

2106.3 Seismic Design Category B. Structures assigned to Seismic Design Category B shall conform to the requirements of Section ~~1.14.4~~ 1.17.4.2 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 ~~and to the additional requirements of this section~~.

~~**2106.3.1 Masonry walls not part of the lateral-force-resisting system.** Masonry partition walls, masonry screen walls and other masonry elements that are not designed to resist vertical or lateral loads, other than those induced by their own mass, shall be isolated from the structure so that the vertical and lateral forces are not imparted to these elements. Isolation joints and connectors between these elements and the structure shall be designed to accommodate the design story drift.~~

2106.4 Additional requirements for structures in Seismic Design Category C. Structures assigned to Seismic Design Category C shall conform to the requirements of Section 2106.3, Section ~~1.14.5~~ 1.17.4.3 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 and the additional requirements of this section.

2106.5 Additional requirements for structures in Seismic Design Category D. Structures assigned to Seismic Design Category D shall conform to the requirements of Section 2106.4, Section ~~1.14.6~~ 1.17.4.4 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5 and the additional requirements of this section.

2106.5.1 Loads for shear walls designed by the allowable stress design method. When calculating in-plane shear or diagonal tension stresses by the allowable stress design method in accordance with Section 2107, special reinforced masonry, shear walls that resist seismic forces shall be designed to resist 1.5 times the seismic forces required by Chapter 16. The 1.5 multiplier need not be applied to the overturning moment.

2106.5.2 Shear wall shear strength. ~~For a shear wall whose nominal shear strength exceeds the shear corresponding to development of its nominal flexural strength, two shear regions exist.~~

~~For all cross sections within a region defined by the base of the shear wall and a plane at a distance *Lw* above the base of the shear wall, the nominal shear strength shall be determined by Equation 21-1.~~

$\cancel{V_n=A_n \rho_n f_y}$ ~~**(Equation 21-1)**~~

~~The required shear strength for this region shall be calculated at a distance $L_w/2$ above the base of the shear wall, but not to exceed one-half story height.~~

~~For the other region, the nominal shear strength of the shear wall shall be determined from Section 2108.~~ Where designing special reinforced masonry shear walls in accordance with Section 3.3 or A.3 or Chapter 4 of TMS 402/ACI 530/ASCE 5, the design shear strength, ϕV_n, shall exceed the shear corresponding to the development of 1.25 times the nominal flexural strength, M_n, of the element, except that the nominal shear strength, V_n, need not exceed 2.5 times required shear strength, V_u.

2106.6 Additional requirements for structures in Seismic Design Category E or F. Structures assigned to Seismic Design Category E or F shall conform to the requirements of Section 2106.5 and Section ~~1.14.7~~ 1.17.4.5 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

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

Reason: The revisions proposed in this code change reflect editorial and substantive revisions incorporated into the 2008 edition of the Building Code Requirements for Masonry Structures (TMS 402/ACI 530/ASCE 5), commonly referred to as the Masonry Standard Joint Committee (MSJC) Code. This code change proposal is one of several to harmonize the design and construction requirements for masonry within the IBC with those in the reference standard. A complete list of revisions incorporated into the reference standard is available for download at www.masonrystandards.org.

The majority of the changes proposed simply update section numbers that have changed in the reference standard. Substantive revisions include:

1) Section 2106.3.1 is proposed to be deleted as it conflicts with Section 2106.1. Section 2106.1 requires that all masonry elements that are not part of the seismic force-resisting system be isolated, regardless of SDC. Section 2106.3.1, conversely, triggers this requirement for SDC B and higher. Language similar to that in Section 2106.3.1 in included in the reference standard for all SDCs.

2) Section 2106.5.2 is proposed to be replaced with the corresponding design provisions from the 2008 MSJC. The language is also clarified that this design check is applicable to the strength design of masonry, which isn't clear in the original IBC language. In reviewing the design provisions of Section 2106.5.2, the MSJC did not think it was appropriate to ignore the contribution of the masonry to the nominal shear strength of a shear wall. The MSJC did agree, however, that encouraging flexural limit states in masonry shear walls was preferable over shear controlled failures, and as such, incorporated the shear capacity check proposed to be included in Section 2106.5.2.

3) The deletion of notations for L_w and ρ_n are proposed as well. With the removal of Equation 21-1 currently in Section 2106, these terms are no longer used in Chapter 21. The revised Section 2106.5.2 introduces the term M_n, nominal moment strength. As such, a corresponding definition is proposed.

Cost Impact: The code change proposal will not increase the cost of construction.

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**

Individual Consideration Agenda

This item is on the agenda for individual consideration because public comments were submitted.

Public Comment 1:

Phil Samblanet and Jason Thompson, The Masonry Society and National Concrete Masonry Association, representing The Masonry Society and The Masonry Alliance for Codes and Standards, request Approved as Submitted.

Commenter's Reason: If code change proposal S175-07/08 is ultimately approved as modified as recommended by the committee, this change proposal becomes redundant and unnecessary.

Public Comment 2:

Can Xiao, City of Phoenix, AZ, representing herself, requests Disapproval.

Commenter's Reason: There are detailing requirements and shear wall strength analysis information provided in IBC Section 2106 for seismic category D that are not covered under ACI 530/ASCE 5-08/TMS 402.

Final Action: AS AM AMPC____ D

S188-07/08
2102, 2107

Proposed Change as Submitted:

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards

Revise as follows:

SECTION 2102
DEFINITIONS AND NOTATIONS

NOTATIONS
~~p_{max} = Maximum reinforcement ratio.~~

2107.1 General. The design of masonry structures using allowable stress design shall comply with Section 2106 and the requirements of Chapters 1 and 2 of ~~ACI 530/ASCE 5/TMS 402~~ <u>TMS 402/ACI 530/ASCE 5</u> except as modified by Sections 2107.2 through ~~2107.8~~ <u>2107.5</u>.

2107.2 ~~ACI 530/ASCE 5/TMS 402~~ <u>TMS 402/ACI 530/ASCE 5</u>, Section 2.1.2, load combinations. Delete Section 2.1.2.1.

~~**2107.3 ACI 530/ASCE 5/TMS 402, Section 2.1.3, design strength.** Delete Sections 2.1.3.4 through 2.1.3.4.3.~~

~~**2107.4 ACI 530/ASCE 5/TMS 402, Section 2.1.6, columns.** Add the following text to Section 2.1.6:~~
~~2.1.6.6 Light-frame construction. Masonry columns used only to support light-frame roofs of carports, porches, sheds or similar structures with a maximum area of 450 square feet (41.8 m2) assigned to Seismic Design Category A, B or Care permitted to be designed and constructed as follows:~~

1. ~~Concrete masonry materials shall be in accordance with Section 2103.1 of the *International Building Code*. Clay or shale masonry units shall be in accordance with Section 2103.2 of the *International Building Code*.~~
2. ~~The nominal cross-sectional dimension of columns shall not be less than 8 inches (203 mm).~~
3. ~~Columns shall be reinforced with not less than one No. 4 bar centered in each cell of the column.~~
4. ~~Columns shall be grouted solid.~~
5. ~~Columns shall not exceed 12 feet (3658 mm) in height.~~
6. ~~Roofs shall be anchored to the columns. Such anchorage shall be capable of resisting the design loads specified in Chapter 16 of the *International Building Code*.~~
7. ~~Where such columns are required to resist uplift loads, the columns shall be anchored to their footings with two No. 4 bars extending a minimum of 24 inches (610 mm) into the columns and bent horizontally a minimum of 15 inches (381 mm) in opposite directions into the footings. One of these bars is permitted to be the reinforcing bar specified in Item 3 above. The total weight of a column and its footing shall not be less than 1.5 times the design uplift load.~~

~~2107.5~~ <u>2107.3</u> ~~ACI 530/ASCE 5/TMS 402~~ <u>TMS 402/ACI 530/ASCE 5</u>, Section 2.1.~~10~~<u>9</u>.7.1.1, lap splices.
Modify Section 2.1.~~10~~<u>9</u>.7.1.1 as follows:

2.1.~~10~~<u>9</u>.7.1.1 The minimum length of lap splices for reinforcing bars in tension or compression, l_d, shall be

$l_d = 0.002 d b f s$ **(Equation 21-2)**

For SI: $l_d = 0.29 d_b f_s$ but not less than 12 inches (305 mm). In no case shall the length of the lapped splice be less than 40 bar diameters.

where:
d_b = Diameter of reinforcement, inches (mm).
f_s = Computed stress in reinforcement due to design loads, psi (MPa).

In regions of moment where the design tensile stresses in the reinforcement are greater than 80 percent of the allowable steel tension stress, *Fs*, the lap length of splices shall be increased not less than 50 percent of the minimum required length. Other equivalent means of stress transfer to accomplish the same 50 percent increase shall be permitted. Where epoxy coated bars are used, lap length shall be increased by 50 percent.

~~2107.6~~ 2107.4 ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5, Section ~~2.1.10.7~~ 2.1.9.7, splices of reinforcement. Modify Section ~~2.1.10.7~~ 2.1.9.7 as follows:

~~2.1.10.7~~ 2.1.9.7 Splices of reinforcement. Lap splices, welded splices or mechanical splices are permitted in accordance with the provisions of this section. All welding shall conform to AWS D1.4. Welded splices shall be of ASTM A 706 steel reinforcement. Reinforcement larger than No. 9 (M #29) shall be spliced using mechanical connections in accordance with Section 2.1.10.7.3.

~~2107.7~~ 2107.5 ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5, Section 2.3.6, maximum bar size. Add the following to Chapter 2:

2.3.6 Maximum bar size. The bar diameter shall not exceed one-eighth of the nominal wall thickness and shall not exceed one-quarter of the least dimension of the cell, course or collar joint in which it is placed.

~~**2107.8 ACI 530/ASCE 5/TMS 402, Section 2.3.7, maximum reinforcement percentage.** Add the following text to Chapter 2:~~

~~2.3.7 Maximum reinforcement percentage. Special reinforced masonry shearwalls having a shear span ratio, *M/Vd*, equal to or greater than 1.0 and having an axial load, *P*, greater than 0.05~~ $\cancel{f'_m A_n}$ ~~that are subjected to in-plane forces shall have a maximum reinforcement ratio,~~ $\cancel{\rho}$~~max, not greater than that computed as follows:~~

$$\cancel{\rho_{\max} = \frac{nf'_m}{2f_y\left(n+\frac{f_y}{f'_m}\right)}} \qquad \text{(Equation 21-3)}$$

~~The maximum reinforcement ratio does not apply in the out-of-plane direction.~~

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

Reason: The revisions proposed in this code change reflect editorial and substantive revisions incorporated into the 2008 edition of the Building Code Requirements for Masonry Structures (TMS 402/ACI 530/ASCE 5), commonly referred to as the Masonry Standard Joint Committee (MSJC) Code. This code change proposal is one of several to harmonize the design and construction requirements for masonry within the IBC with those in the reference standard. A complete list of revisions incorporated into the reference standard is available for download at www.masonrystandards.org.

Specific revisions proposed above include:

1) Section 2107.3 is proposed to be deleted without replacement. This section deleted Sections 2.1.3.4 through 2.1.3.4.3 of the 2005 MSJC, which contained a pseudo-strength design procedure for masonry whereby allowable stresses were scaled-up to corresponding strength-levels. These provisions have been removed from the 2008 edition of the MSJC. As such, there is no longer a need to delete these provisions.

2) Section 2107.4 is proposed to be deleted without replacement. This section included an alternative design and construction option for lightly loaded columns. Nearly identical provisions have been incorporated into the 2008 MSJC, and as such, are proposed for deletion from the IBC. There are, however, two substantive differences between the IBC provisions and those adopted into the 2008 MSJC:

a) The 450 ft^2 trigger for the maximum supported area was changed to a maximum load of 2,000 pounds (service level). Because design loads can vary significantly for a given tributary area (for example, a design snow load of 10 psf versus 60 psf), the MSJC opted to be more clear in the limits of this alternative. The axial load limit of 2,000 pounds was developed based on the flexure capacity of a nominal 8 inch by 8 inch by 12 foot high column with one No. 4 reinforcing bar in the center and f'_m of 1350 psi. An axial load of 2,000 pounds at the edge of the member will result in a moment that is approximately equal to the moment capacity of this member.

b) The MSJC language clarifies that such elements must still be designed using the strength design or allowable stress design procedures and comply with all design and modeling assumptions and inherent load path requirements. The IBC language has been interpreted by some as a deemed-to-comply, prescriptive detailing option that if met, does not require engineering analysis. The MSJC disagreed with this interpretation and instead opted to clarify that such elements be designed as if they were any other part of the structure.

3) Section 2107.6 (Section 2107.4 as proposed above) introduces a requirement that welded splices use ASTM A 706 reinforcement intended for such applications. This material limitation mirrors that required in Section 2108.3 of the IBC for the strength design of masonry.

4) Section 2107.8 is proposed to be deleted without replacement. An identical requirement for limiting the amount of reinforcement for the allowable stress design of special reinforced shear walls has been incorporated into the 2008 MSJC. As such, this modification is no longer required.

5) The definition for ρ_{max} is proposed to be deleted, as with the removal of Equation 21-3 the term is no longer used in Chapter 21.

The remaining changes editorially update the section references.

Cost Impact: The code change proposal will not increase the cost of construction.

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

Individual Consideration Agenda

This item is on the agenda for individual consideration because public comments were submitted.

Public Comment 1:

Phil Samblanet and Jason Thompson, The Masonry Society and National Concrete Masonry Association, representing The Masonry Society and The Masonry Alliance for Codes and Standards, request Approval as Submitted.

Commenter's Reason: If code change proposal S175-07/08 is ultimately approved as modified as recommended by the committee, this change proposal becomes redundant and unnecessary.

Public Comment 2:

Richard vonWeller, Park City, representing Utah Chapter of ICC, requests Disapproval.

Commenter's Reason: This comments toS175 and items S182, S184, S188, and S190 all are related to the same general issue which was heavily debated disapproved at the Rochester final action hearings. Several are actually identical to those previously defeated. These are prime examples of a disturbing trend in the IBC to eliminate basic construction code requirements from the IBC. There has been a wholesale effort to take critical elements out of our base code and move them to reference standards where the voting members of ICC have little or no control over the outcome of any changes. In the reason for S190 the proponent is wanting to remove numerous provisions from the ICC process thereby "reducing the burdens on code officials in overseeing often esoteric, technical or specialized provisions". Please don't do code officials any such favor.

Once a requirement moves out of ICC's system what hope is there we will regain our opportunity to effect positive change in the public interest? The very reason ICC exists is because of our efforts to bring together of all the stakeholders in a single forum to discuss, debate and decide in a process where the outcome is determined by those with no financial interest.

The proponent agues the technical requirements are still incorporated through the reference standard, but the practicality of use and enforcement becomes extremely cumbersome, costly and less effective. Needed information must be readily available to enforcement personnel. Is it really a good idea to eliminate the basic requirements for grout, reinforcement, corrosion protection, mortar, wall ties, cold and hot weather protection, seismic resistance, and columns for light frame construction from the IBC? ICC does not even test on the reference standards for certifications. Shouldn't a certified inspector or plans examiner be able to answer simple questions about these subjects?

The voting members must take a stand and disapprove these changes to ensure the IBC is a useful tool for code officials to help protect the health, safety and welfare of our citizens.

Public Comment 3:

Cam Xiao, City of Phoenix, AZ, representing herself, requests Disapproval.

Commenter's Reason: The requirement for lightly-loaded columns in TMS402/ACI 530/ASCE 5-08 is less restrictive than IBC section 2107's requirement for light-frame construction columns. Masonry code removed the material, roof anchorage to column and uplifting resistance requirements, which are three important requirements for this type of columns design. It will jeopardize public safety to remove those two requirements from building code. Other steel reinforcement lap splice, maximum bar size and maximum reinforcement percentage requirement in Section 2107 can not be found in masonry code either

Final Action: AS AM AMPC____ D

S189-07/08
2108

Proposed Change as Submitted:

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards

Revise as follows:

SECTION 2108
STRENGTH DESIGN OF MASONRY

2108.1 General. The design of masonry structures using strength design shall comply with Section 2106 and the requirements of Chapters 1 and 3 of TMS 402/ACI 530/ASCE 5~~/TMS 402~~, except as modified by Sections 2108.2 through 2108.3~~4~~.

> **Exception:** AAC masonry shall comply with the requirements of Chapter 1 and Appendix A of TMS 402/ACI 530/ASCE 5~~/TMS 402~~.

2108.2 TMS 402/ACI 530/ASCE 5~~/TMS 402~~, Section 3.3.3.3 development. Add the following text to Section 3.3.3.3:

> The required development length of reinforcement shall be determined by Equation (3-15), but shall not be less than 12 inches (305 mm) and need not be greater than 72 d_b.

2108.3 TMS 402/ACI 530/ASCE 5~~/TMS 402~~, Section 3.3.3.4, splices. Modify items (b) and (c) of Section 3.3.3.4 as follows:

> 3.3.3.4 (b). A welded splice shall have the bars butted and welded to develop at least 125 percent of the yield strength, f_y, of the bar in tension or compression, as required.Welded splices shall be of ASTM A 706 steel reinforcement. Welded splices shall not be permitted in plastic hinge zones of intermediate or special reinforced walls or special moment frames of masonry.
>
> 3.3.3.4 (c). Mechanical splices shall be classified as Type 1 or 2 according to Section 21.2.6.1 of ACI 318. Type 1 mechanical splices shall not be used within a plastic hinge zone or within a beam-column joint of intermediate or special reinforced masonry shear walls or special moment frames. Type 2 mechanical splices are permitted in any location within a member.

~~**2108.4 ACI 530/ASCE 5/TMS 402, Section 3.3.3.5, maximum areas of flexural tensile reinforcement.** Add the following text to Section 3.3.3.5:~~

> ~~3.3.3.5.5 For special prestressed masonry shearwalls, strain in all prestressing steel shall be computed to be compatible with a strain in the extreme tension reinforcement equal to five times the strain associated with the reinforcement yield stress, f_y. The calculation of the maximum reinforcement shall consider forces in the prestressing steel that correspond to these calculated strains.~~

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

Reason: The revisions proposed in this code change reflect editorial and substantive revisions incorporated into the 2008 edition of the Building Code Requirements for Masonry Structures (TMS 402/ACI 530/ASCE 5), commonly referred to as the Masonry Standard Joint Committee (MSJC) Code. This code change proposal is one of several to harmonize the design and construction requirements for masonry within the IBC with those in the reference standard. A complete list of revisions incorporated into the reference standard is available for download at www.masonrystandards.org.

Specific revisions proposed above include:

IBC Section 2108.4 introduces maximum reinforcement limitations for special prestressed masonry shear walls. An identical set of requirements has been incorporated into the 2008 MSJC. As such, this modification is no longer required.

Cost Impact: The code change proposal will not increase the cost of construction.

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**

Individual Consideration Agenda

This item is on the agenda for individual consideration because public comments were submitted.

Public Comment 1:

Phil Samblanet and Jason Thompson, The Masonry Society and National Concrete Masonry Association, representing The Masonry Society and The Masonry Alliance for Codes and Standards, requests Approval as Submitted.

Commenter's Reason: If code change proposal S175-07/08 is ultimately approved as modified as recommended by the committee, this change proposal becomes redundant and unnecessary.

Public Comment 2:

Can Xiao, City of Phoenix, AZ, representing herself, requests Approval as Modified by this Public Comment.

Modify proposal as follows:

<u>**2108.4 TMS 402/ACI 530/ASCE 5/, Section 3.3.3.5, maximum areas of flexural tensile reinforcement.** Add the following text to Section 3.3.3.5:</u>

<u>3.3.3.5.5 For special prestressed masonry shearwalls, strain in all prestressing steel shall be computed to be compatible with a strain in the extreme tension reinforcement equal to five times the strain associated with the reinforcement yield stress, f_y. The calculation of the maximum reinforcement shall consider forces in the prestressing steel that correspond to these calculated strains.</u>

(Portions of the proposal not shown remain unchanged)

Commenter's Reason: There are no similar requirements for special pre-stressed masonry shear wall in TMS402/ACI 530/ASCE 5-08. Section 2108.4 needs to stay.

Final Action: AS AM AMPC____ D

S190-07/08

1704.5.1, 1704.5.2, 1704.5.3, 1708.1.1, 1708.1.2, 1708.1.3, 1708.1.4, 2104.4, 2101.2.5, 2101.2.5.1 (New), 2101.2.7 (New), 2101.2.8 (New), 2109, 2109.1, 2109.1.1, 2110

Proposed Change as Submitted:

Proponent: Phillip J. Samblanet, The Masonry Society

1. Revise as follows:

2101.2.4 Empirical design. Masonry designed by the empirical design method shall comply with the provisions of ~~Sections 2106 and 2109 or~~ Chapter 5 of ACI 530/ASCE 5/TMS 402. <u>In buildings that exceed one or more of the limitations in Section 5.1.2 of ACI 530/ASCE 5/TMS 402, masonry shall be designed in accordance with the engineered design provisions of Section 2101.2.1, 2101.2.2, 2101.2.3 or the foundation wall provisions of Section 1805.5.</u>

2101.2.5 Glass unit masonry. Glass unit masonry shall comply with the provisions of ~~Section 2110 or~~ Chapter 7 of ACI 530/ASCE 5/ TMS 402 and this section.

2. Add new text as follows:

2101.2.5.1 Limitations. Solid or hollow approved glass block shall not be used in fire walls, party walls, fire barriers, fire partitions, or smoke barriers, or for load-bearing construction. Such blocks shall be erected with mortar and reinforcement in metal channel-type frames, structural frames, masonry or concrete recesses, embedded panel anchors as provided for both exterior and interior walls or other approved joint materials. Wood strip framing shall not be used in walls required to have a fire-resistance rating by other provisions of this code.

Exceptions:

1. Glass-block assemblies having a fire protection rating of not less than 3/4 hour shall be permitted as opening protectives in accordance with Section 715 in fire barriers, fire partitions and smoke barriers that have a required fire-resistance rating of 1 hour or less and do not enclose exit stairways, exit ramps, or exit passageways.
2. Glass-block assemblies as permitted in Section 404.5, Exception 2.

2101.2.7 Surface-bonded masonry. Dry-stacked, surface-bonded masonry shall comply with the provisions of Section 2109.

2101.2.8 Adobe masonry. Adobe masonry shall comply with the provisions of Section 2110.

3. Revise as follows:

SECTION 2109
~~EMPIRICAL DESIGN OF MASONRY~~ SURFACE-BONDED MASONRY

2109.1 General. ~~Empirically designed~~ Dry-stacked, surface-bonded masonry shall conform ~~to this chapter or~~ Chapter 5 of ACI 530/ASCE 5/TMS 402 except as modified in this section.

2109.1.1 Limitations. The use of ~~empirical design~~ of dry-stacked, surface-bonded masonry shall be limited as ~~follows:~~ noted in Section 5.1.2 of ACI 530/ASCE 5/TMS 402. The use of dry-stacked, surface-bonded masonry shall be prohibited in Occupancy Category IV structures.

1. ~~Empirical design shall not be used for buildings assigned to Seismic Design Category D, E or F as specified in Section 1613, nor for the design of the seismic-force-resisting system for buildings assigned to Seismic Design Category B or C.~~
2. ~~Empirical design shall not be used for masonry elements that are part of the lateral-force-resisting system where the basic wind speed exceeds 110 mph (79 m/s).~~
3. ~~Empirical design shall not be used for interior masonry elements that are not part of the lateral force-resisting system in buildings other than enclosed buildings as defined in Chapter 6 of ASCE 7 in:~~
   ~~3.1. Buildings over 180 feet (55 100 mm) in height.~~
   ~~3.2. Buildings over 60 feet (18 400 mm) in height where the basic wind speed exceeds 90 mph (40 m/s).~~
   ~~3.3. Buildings over 35 feet (10 700 mm) in height where the basic wind speed exceeds 100 mph (45 m/s).~~
   ~~3.4. Where the basic wind speed exceeds 110 mph (79 m/s).~~
4. ~~Empirical design shall not be used for exterior masonry elements that are not part of the lateral force-resisting system and that are more than 35 feet (10 700 mm) above ground:~~
   ~~4.1. Buildings over 180 feet (55 100 mm) in height.~~
   ~~4.2. Buildings over 60 feet (18 400 mm) in height where the basic wind speed exceeds 90 mph (40 m/s).~~
   ~~4.3. Buildings over 35 feet (10 700 mm) in height where the basic wind speed exceeds 100 mph (45 m/s).~~
5. ~~Empirical design shall not be used for exterior masonry elements that are less than or equal to 35 feet (10 700 mm) above ground where the basic wind speed exceeds 110 mph (79 m/s).~~
6. ~~Empirical design shall only be used when the resultant of gravity loads is within the center third of the wall thickness and within the central area bounded by lines at one-third of each cross-sectional dimension of foundation piers.~~
7. ~~Empirical design shall not be used for AAC masonry.~~

In buildings that exceed one or more of the ~~above~~ limitations in Section 5.1.2 of ACI 530/ASCE 5/TMS 402, masonry shall be designed in accordance with the engineered design provisions of Section ~~2107 or 2108~~ 2101.2.1, 2101.2.2, 2101.2.3 or the foundation wall provisions of Section 1805.5.

4. Delete without substitution:

~~**2109.2 Lateral stability.**~~

~~**2109.2.1 Shear walls.** Where the structure depends upon masonry walls for lateral stability, shear walls shall be provided parallel to the direction of the lateral forces resisted.~~

~~**2109.2.2 Roofs.** The roof construction shall be designed so as not to impart out-of-plane lateral thrust to the walls under roof gravity load.~~

~~**2109.2.3 Surface-bonded walls.** Dry-stacked, surface-bonded concrete masonry walls shall comply with the requirements of this code for masonry wall construction, except where otherwise noted in this section.~~

5. Revise as follows:

~~**2109.2.3.1**~~ **2109.2 Strength.** Dry-stacked, surface-bonded concrete masonry walls shall be of adequate strength and proportions to support all superimposed loads without exceeding the allowable stresses listed in Table ~~2109.2.3.1~~ 2109.2. Allowable stresses not specified in Table ~~2109.2.3.1~~ 2109.2 shall comply with the requirements of ACI 530/ASCE 5/TMS 402.

TABLE ~~2109.2.3.1~~ 2109.2
ALLOWABLE STRESS GROSS CROSS-SECTIONAL AREA FOR DRY-STACKED, SURFACE-BONDED CONCRETE MASONRY WALLS

(Portions of table not shown remain unchanged)

~~**2109.2.3.2**~~ **2109.3 Construction.** Construction of dry-stacked, surface-bonded masonry walls, including stacking and leveling of units, mixing and application of mortar and curing and protection shall comply with ASTM C 946.

6. Delete without substitution:

~~**2109.3 Compressive stress requirements.**~~

~~**2109.3.1 Calculations.** Compressive stresses in masonry due to vertical dead plus live loads, excluding wind or seismic loads, shall be determined in accordance with Section 2109.3.2.1. Dead and live loads shall be in accordance with Chapter 16, with live load reductions as permitted in Section 1607.9.~~

~~**2109.3.2 Allowable compressive stresses.** The compressive stresses in masonry shall not exceed the values given in Table 2109.3.2. Stress shall be calculated based on specified rather than nominal dimensions.~~

~~**2109.3.2.1 Calculated compressive stresses.** Calculated compressive stresses for single wythe walls and for multiwythe composite masonry walls shall be determined by dividing the design load by the gross cross-sectional area of the member. The area of openings, chases or recesses in walls shall not be included in the gross cross-sectional area of the wall.~~

~~**2109.3.2.2 Multiwythe walls.** The allowable stress shall be as given in Table 2109.3.2 for the weakest combination of the units used in each wythe.~~

~~**2109.4 Lateral support.**~~

~~**2109.4.1 Intervals.** Masonry walls shall be laterally supported in either the horizontal or vertical direction at intervals not exceeding those given in Table 2109.4.1.~~

~~**TABLE 2109.4.1**~~
~~**WALL LATERAL SUPPORT REQUIREMENTS**~~

~~**2109.4.2 Thickness.** Except for cavity walls and cantilever walls, the thickness of a wall shall be its nominal thickness measured perpendicular to the face of the wall. For cavity walls, the thickness shall be determined as the sum of the nominal thicknesses of the individual wythes. For cantilever walls, except for parapets, the ratio of height-to-nominal thickness shall not exceed 6 for solid masonry or 4 for hollow masonry. For parapets, see Section 2109.5.4.~~

~~**2109.4.3 Support elements.** Lateral support shall be provided by crosswalls, pilasters, buttresses or structural frame members when the limiting distance is taken horizontally, or by floors, roofs acting as diaphragms or structural frame members when the limiting distance is taken vertically.~~

~~**2109.5 Thickness of masonry.** Minimum thickness requirements shall be based on nominal dimensions of masonry.~~

~~**2109.5.1 Thickness of walls.** The thickness of masonry walls shall conform to the requirements of Section 2109.5.~~

~~**2109.5.2 Minimum thickness.**~~

~~**2109.5.2.1 Bearing walls.** The minimum thickness of masonry bearing walls more than one story high shall be 8 inches (203 mm). Bearing walls of one-story buildings shall not be less than 6 inches (152 mm) thick.~~

~~**2109.5.2.2 Rubble stone walls.** The minimum thickness of rough, random or coursed rubble stone walls shall be 16 inches (406 mm).~~

~~**2109.5.2.3 Shear walls.** The minimum thickness of masonry shear walls shall be 8 inches (203 mm).~~

~~**2109.5.2.4 Foundation walls.** The minimum thickness of foundation walls shall be 8 inches (203 mm) and as required by Section 2109.5.3.1.~~

~~TABLE 2109.3.2~~
~~ALLOWABLE COMPRESSIVE STRESSES FOR EMPIRICAL DESIGN OF MASONRY~~

~~**2109.5.2.5 Foundation piers.** The minimum thickness of foundation piers shall be 8 inches (203 mm).~~

~~**2109.5.2.6 Parapet walls.** The minimum thickness of parapet walls shall be 8 inches (203 mm) and as required by Section 2109.5.4.1.~~

~~**2109.5.2.7 Change in thickness.** Where walls of masonry of hollow units or masonry bonded hollow walls are decreased in thickness, a course or courses of solid masonry shall be interposed between the wall below and the thinner wall above, or special units or construction shall be used to transmit the loads from face shells or wythes above to those below.~~

~~**2109.5.3 Foundation walls.** Foundation walls shall comply with the requirements of Section 2109.5.3.1 or 2109.5.3.2.~~

~~**2109.5.3.1 Minimum thickness.** Minimum thickness for foundation walls shall comply with the requirements of Table 2109.5.3.1. The provisions of Table 2109.5.3.1 are only applicable where the following conditions are met:~~

1. ~~The foundation wall does not exceed 8 feet (2438 mm) in height between lateral supports;~~
2. ~~The terrain surrounding foundation walls is graded to drain surface water away from foundation walls;~~
3. ~~Backfill is drained to remove ground water away from foundation walls;~~
4. ~~Lateral support is provided at the top of foundation walls prior to backfilling;~~
5. ~~The length of foundation walls between perpendicular masonry walls or pilasters is a maximum of three times the basement wall height;~~
6. ~~The backfill is granular and soil conditions in the area are nonexpansive; and~~
7. ~~Masonry is laid in running bond using TypeMor S mortar.~~

~~TABLE 2109.5.3.1~~
~~FOUNDATION WALL CONSTRUCTION~~

~~**2109.5.3.2 Design requirements.** Where the requirements of Section 2109.5.3.1 are not met, foundation walls shall be designed in accordance with Section 1805.5.~~

~~**2109.5.4 Parapet walls.**~~

~~**2109.5.4.1 Minimum thickness.** The minimum thickness of unreinforced masonry parapets shall meet Section 2109.5.2.6 and their height shall not exceed three times their thickness.~~

~~**2109.5.4.2 Additional provisions.** Additional provisions for parapet walls are contained in Sections 1503.2 and 1503.3.~~

~~**2109.6 Bond.**~~

~~**2109.6.1 General.** The facing and backing of multiwythe masonry walls shall be bonded in accordance with Section 2109.6.2, 2109.6.3 or 2109.6.4.~~

~~**2109.6.2 Bonding with masonry headers.**~~

~~**2109.6.2.1 Solid units.** Where the facing and backing (adjacent wythes) of solid masonry construction are bonded by means of masonry headers, no less than 4 percent of the wall surface of each face shall be composed of headers extending not less than 3 inches (76 mm) into the backing. The distance between adjacent full-length headers shall not exceed 24 inches (610 mm) either vertically or horizontally. In walls in which a single header does not extend through the wall, headers from the opposite sides shall overlap at least 3 inches (76 mm), or headers from opposite sides shall be covered with another header course overlapping the header below at least 3 inches (76 mm).~~

~~**2109.6.2.2 Hollow units.** Where two or more hollow units are used to make up the thickness of a wall, the stretcher courses shall be bonded at vertical intervals not exceeding 34 inches (864 mm) by lapping at least 3 inches (76 mm) over the unit below, or by lapping at vertical intervals not exceeding 17 inches (432 mm) with units that are at least 50 percent greater in thickness than the units below.~~

~~**2109.6.2.3 Masonry bonded hollow walls.** In masonry bonded hollow walls, the facing and backing shall be bonded so that not less than 4 percent of the wall surface of each face is composed of masonry bonded units extending not less than 3 inches (76 mm) into the backing. The distance between adjacent bonders shall not exceed 24 inches (610 mm) either vertically or horizontally.~~

~~**2109.6.3 Bonding with wall ties or joint reinforcement.**~~

~~**2109.6.3.1 Bonding with wall ties.** Except as required by Section 2109.6.3.1.1, where the facing and backing (adjacent wythes) of masonry walls are bonded with wire size W2.8 (MW18) wall ties or metal wire of equivalent stiffness embedded in the horizontal mortar joints, there shall be at least one metal tie for each 41/2 square feet (0.42 m^2) of wall area. The maximum vertical distance between ties shall not exceed 24 inches (610 mm), and the maximum horizontal distance shall not exceed 36 inches (914 mm). Rods or ties bent to rectangular shape shall be used with hollow masonry units laid with the cells vertical. In other walls, the ends of ties shall be bent to 90-degree (1.57 rad) angles to provide hooks no less than 2 inches (51 mm) long. Wall ties shall be without drips. Additional bonding ties shall be provided at all openings, spaced not more than 36 inches (914 mm) apart around the perimeter and within 12 inches (305 mm) of the opening.~~

~~**2109.6.3.1.1 Bonding with adjustable wall ties.** Where the facing and backing (adjacent wythes) of masonry are bonded with adjustable wall ties, there shall be at least one tie for each 1.77 square feet (0.164 m^2) of wall area. Neither the vertical nor horizontal spacing of the adjustable wall ties shall exceed 16 inches (406 mm). The maximum vertical offset of bed joints from one wythe to the other shall be 11/4 inches (32 mm). The maximum clearance between connecting parts of the ties shall be 1/16 inch (1.6 mm). When pintle legs are used, ties shall have at least two wire size W2.8 (MW18) legs.~~

~~**2109.6.3.2 Bonding with prefabricated joint reinforcement.** Where the facing and backing (adjacent wythes) of masonry are bonded with prefabricated joint reinforcement, there shall be at least one cross wire serving as a tie for each 22/3 square feet (0.25m^2) of wall area. The vertical spacing of the joint reinforcing shall not exceed 24 inches (610 mm). Cross wires on prefabricated joint reinforcement shall not be less than W1.7 (MW11) and shall be without drips. The longitudinal wires shall be embedded in the mortar.~~

~~**2109.6.4 Bonding with natural or cast stone.**~~

~~**2109.6.4.1 Ashlar masonry.** In ashlar masonry, bonder units, uniformly distributed, shall be provided to the extent of not less than 10 percent of the wall area. Such bonder units shall extend not less than 4 inches (102 mm) into the backing wall.~~

~~**2109.6.4.2 Rubble stone masonry.** Rubble stone masonry 24 inches (610 mm) or less in thickness shall have bonder units with a maximum spacing of 36 inches (914 mm) vertically and 36 inches (914 mm) horizontally, and if the masonry is of greater thickness than 24 inches (610 mm), shall have one bonder unit for each 6 square feet (0.56 m^2) of wall surface on both sides.~~

~~**2109.6.5 Masonry bonding pattern.**~~

~~**2109.6.5.1 Masonry laid in running bond.** Each wythe of masonry shall be laid in running bond, head joints in successive courses shall be offset by not less than one-fourth the unit length or the masonry walls shall be reinforced longitudinally as required in Section 2109.6.5.2.~~

~~**2109.6.5.2 Masonry laid in stack bond.** Where unit masonry is laid with less head joint offset than in Section 2109.6.5.1, the minimum area of horizontal reinforcement placed in mortar bed joints or in bond beams spaced not more than 48 inches (1219 mm) apart, shall be 0.0003 times the vertical cross-sectional area of the wall.~~

~~**2109.7 Anchorage.**~~

~~**2109.7.1 General.** Masonry elements shall be anchored in accordance with Sections 2109.7.2 through 2109.7.4.~~

~~**2109.7.2 Intersecting walls.** Masonry walls depending upon one another for lateral support shall be anchored or bonded at locations where they meet or intersect by one of the methods indicated in Sections 2109.7.2.1 through 2109.7.2.5.~~

~~**2109.7.2.1 Bonding pattern.** Fifty percent of the units at the intersection shall be laid in an overlapping masonry bonding pattern, with alternate units having a bearing of not less than 3 inches (76 mm) on the unit below.~~

~~**2109.7.2.2 Steel connectors.** Walls shall be anchored by steel connectors having a minimum section of 1/4 inch (6.4 mm) by 11/2 inches (38 mm), with ends bent up at least 2 inches (51 mm) or with cross pins to form anchorage. Such anchors shall be at least 24 inches (610 mm) long and the maximum spacing shall be 48 inches (1219 mm).~~

~~**2109.7.2.3 Joint reinforcement.** Walls shall be anchored by joint reinforcement spaced at a maximum distance of 8 inches (203 mm). Longitudinal wires of such reinforcement shall be at least wire size W1.7 (MW 11) and shall extend at least 30 inches (762 mm) in each direction at the intersection.~~

~~**2109.7.2.4 Interior nonload-bearing walls.** Interior nonload-bearing walls shall be anchored at their intersection, at vertical intervals of not more than 16 inches (406 mm) with joint reinforcement or 1/4-inch (6.4 mm) mesh galvanized hardware cloth.~~

~~**2109.7.2.5 Ties, joint reinforcement or anchors.** Other metal ties, joint reinforcement or anchors, if used, shall be spaced to provide equivalent area of anchorage to that required by this section.~~

~~**2109.7.3 Floor and roof anchorage.** Floor and roof diaphragms providing lateral support to masonry shall comply with the live loads in Section 1607.3 and shall be connected to the masonry in accordance with Sections 2109.7.3.1 through 2109.7.3.3. Roof loading shall be determined in accordance with Chapter 16 and, when net uplift occurs, uplift shall be resisted entirely by an anchorage system designed in accordance with the provisions of Sections 2.1 and 2.3, Sections 3.1 and 3.3 or Chapter 4 of ACI 530/ASCE 5/TMS 402.~~

~~**2109.7.3.1Wood floor joists.** Wood floor joists bearing on masonry walls shall be anchored to the wall at intervals not to exceed 72 inches (1829 mm) by metal strap anchors. Joists parallel to thewall shall be anchored with metal straps spaced not more than 72 inches (1829 mm) o.c. extending over or under and secured to at least three joists. Blocking shall be provided between joists at each strap anchor.~~

~~**2109.7.3.2 Steel floor joists.** Steel floor joists bearing on masonry walls shall be anchored to the wall with 3/8-inch (9.5 mm) round bars, or their equivalent, spaced not more than 72 inches (1829 mm) o.c. Where joists are parallel to the wall, anchors shall be located at joist bridging.~~

~~**2109.7.3.3 Roof diaphragms.** Roof diaphragms shall be anchored to masonry walls with 1/2-inch-diameter (12.7 mm) bolts, 72 inches (1829 mm) o.c. or their equivalent. Bolts shall extend and be embedded at least 15 inches (381 mm) into the masonry, or be hooked or welded to not less than 0.20 square inch (129 mm2) of bond beam reinforcement placed not less than 6 inches (152 mm) from the top of the wall.~~

~~**2109.7.4 Walls adjoining structural framing.** Where walls are dependent upon the structural frame for lateral support, they shall be anchored to the structural members with metal anchors or otherwise keyed to the structural members. Metal anchors shall consist of 1/2-inch (12.7 mm) bolts spaced at 48 inches (1219 mm) o.c. embedded 4 inches (102 mm) into the masonry, or their equivalent area.~~

7. Revise as follows:

<u>**SECTION 2110**</u>
<u>**ADOBE MASONRY**</u>

~~**2109.8**~~ <u>**2110.1**</u> **Adobe construction.** Adobe construction shall comply with this section and shall be subject to the requirements of this code for Type V construction <u>and Chapter 5 of ACI 530/ASCE 5/TMS 402</u>.

8. Add new text as follows:

<u>**2110.1.1 Limitations.** The use of adobe masonry shall be limited as noted in Section 5.1.2 of ACI 530/ASCE 5/TMS 402. The use of adobe masonry shall be prohibited in Occupancy Category IV structures.</u>

<u>In buildings that exceed one or more of the limitations in Section 5.1.2 of ACI 530/ASCE 5/TMS 402, masonry shall be designed in accordance with the engineered design provisions of Section 2101.2.1, 2101.2.2, 2101.2.3 or the foundation wall provisions of Section 1805.5.</u>

Renumber remainder of Section 2109.8

9. Delete Section 2110.1 Scope through Section 2110.7 Reinforcement without substitution.

10. Revise as follows:

1704.5 Masonry construction. Masonry construction shall be inspected and evaluated in accordance with the requirements of Sections 1704.5.1 through 1704.5.3, 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<u>, surface-bonded masonry or adobe masonry</u> designed by Section ~~2109, 2110 or Chapter 14, respectively, or by Chapter 5, 7 or 6 of ACI 530/ASCE 5/TMS 402~~ <u>2101.2.4, 2101.2.5, 2101.2.6, 2101.2.7, or 2101.2.8</u>, 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.

1704.5.1 Empirically designed masonry, glass unit masonry and masonry veneer in Occupancy Category IV. The minimum special inspection program for 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~~ <u>2101.2.4, 2101.2.5, or 2101.2</u>, respectively, in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with Table 1704.5.1.

1704.5.2 Engineered masonry in Occupancy Category I, II or III. The minimum special inspection program for masonry designed by Section ~~2107 or 2108 or by chapters other than Chapters 5, 6 or 7 of ACI 530/ASCE 5/TMS 402~~ <u>2101.2.1, 2101.2.2 or 2101.2</u> in structures classified as Occupancy Category I, II or III, in accordance with Section 1604.5, shall comply with Table 1704.5.1.

1704.5.3 Engineered masonry in Occupancy Category IV. The minimum special inspection program for masonry designed by Section ~~2107 or 2108 or by chapters other than Chapters 5, 6 or 7 of ACI 530/ASCE 5/TMS 402~~ <u>2101.2.1, 2101.2.2 or 2101.2</u> in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with Table 1704.5.3.

1708.1.1 Empirically designed masonry<u>,</u> ~~and~~ glass unit masonry<u>, surface-bonded masonry and adobe masonry</u> in Occupancy Category I, II or III. For masonry designed by Section ~~2109 or 2110 or by Chapter 5 or 7 of ACI 530/ASCE 5/TMS 402~~ <u>2101.2.4, 2101.2.5, 2101.2.7, or 2101.2.8</u> in structures classified as Occupancy Category I, II or III, in accordance with Section 1604.5, certificates of compliance used in masonry construction shall be verified prior to construction.

1708.1.2 Empirically designed masonry and glass unit masonry in Occupancy Category IV. The minimum testing and verification prior to construction for masonry designed by Section ~~2109 or 2110 or by Chapter 5 or 7 of ACI 530/ASCE 5/TMS 402~~ 2101.2.4 or 2101.2.5 in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with the requirements of Table 1708.1.2.

1708.1.3 Engineered masonry in Occupancy Category I, II or III. The minimum testing and verification prior to construction for masonry designed by Section ~~2107 or 2108 or by chapters other than Chapter 5, 6 or 7 of ACI 530/ASCE 5/TMS 402~~ 2101.2.1, 2101.2.2 or 2101.2.3 in structures classified as Occupancy Category I, II or III, in accordance with Section 1604.5, shall comply with Table 1708.1.2.

1708.1.4 Engineered masonry in Occupancy Category IV. The minimum testing and verification prior to construction for masonry designed by Section ~~2107 or 2108 or by chapters other than Chapter 5, 6 or 7 of ACI 530/ASCE 5/TMS 402~~ 2101.2.1, 2101.2.2 or 2101.2.3 in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with Table 1708.1.4.

Reason: This proposal is essentially identical to a proposal submitted during the last supplement cycle for the IBC, which was recommended for approval as modified by the IBC Structural Subcommittee, and which was narrowly overturned on the floor in Rochester. It is being brought back for reconsideration because the change greatly simplifies the Code for building officials, designers, and inspectors. Opposition to the change was procedural (globally), not technical. The primary opposition related to concerns about taking critical provisions out of the I-Codes. While this is a valid concern, and while the proponents share the goal to have building officials involved in the development of all provisions in, or referenced by the I-Codes, the use of referenced standards are nevertheless widely used throughout the I-Codes for many good reasons (including having experts, overseen by consensus procedures for adequate balance and consideration of all issue, focusing on issues of interest, and reducing burdens on code officials in overseeing often esoteric, technical, or specialized provisions).

This change is proposed again with the hopes that those good reasons are still supported and with the intent of simplifying the Code for building officials, designers, contractors and inspectors. The proposed revisions will remove duplicate provisions, thus simplifying the IBC and reducing the chance that the provisions in the IBC and the referenced standard vary unnecessarily. In addition, for clarity, the provisions for Surface-bonded masonry and Adobe Masonry are proposed to be moved to separate chapters.

To maintain a clear and logical inspection path, section references are updated in Chapter 17 to be consistent with proposed revisions in Chapter 21. With the exceptions noted below, these revisions do not change the application of the inspection provisions, but rather more clearly and easily identify which requirements apply to which design method.

To the best of the proponents' knowledge, this change has no technical impact with one exception. Currently, it could be interpreted that the IBC permits the use of surface bonded masonry and adobe masonry in essential facilities classified as Occupancy Category IV. The proponents are not sure this was truly the intent of the IBC and thus have proposed prohibiting the use of these methods from Occupancy Category IV.

If this change is not approved, revisions made to TMS 402-08/ACI 530-08/ASCE 5-08 should be considered if these provisions are to remain consistent with the referenced standard. If not, the IBC may conflict the referenced standard causing design construction and inspection confusion.

In this change is approved, future updates to empirical requirements and glass unit masonry provisions will be balloted in the consensus Masonry Standards Joint Committee, which is charged with updating the TMS 402/ACI 530/ASCE 5. This forum welcomes input from all segments of the design and construction community, and would appreciate active participation by building officials interested in masonry. In addition, public comment forums are open to the general public by TMS, ACI and ASCE and provide building officials and others an opportunity to view and comment on proposed changes. All such comments must be considered before the next version of the provisions are finalized. Those interested in working with the committee or reviewing proposed changes to these consensus standards are encouraged to contact the proponents to be added to the committee contact list.

One final clarification. During the Rochester hearings, opposition to this change noted concerns with the high cost of having to purchase numerous referenced standards to be able to effectively use the IBC. The proponents share this concern, and note that this proposed change does not require a building official, inspector, designer, or anyone else using the I-Code to have any additional references above those already required.

We are hopeful that this change will be approved as it will simplify and streamline the code, making application easier for designers, contractors, building officials and inspectors.

Cost Impact: The code change proposal will not increase the cost of construction.

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**

Individual Consideration Agenda

This item is on the agenda for individual consideration because public comments were submitted.

Public Comment 1:

Phil Samblanet and Jason Thompson, The Masonry Society and National Concrete Masonry Association, representing The Masonry Society and The Masonry Alliance for Codes and Standards, request Approval as Submitted.

Commenter's Reason: If code change proposal S175-07/08 is ultimately approved as modified as recommended by the committee, this change proposal becomes redundant and unnecessary.

Public Comment 2:

Richard vonWeller, Park City, representing Utah Chapter of ICC, requests Disapproval.

Commenter's Reason: This comments to S175 and items S182, S184, S188, and S190 all are related to the same general issue which was heavily debated disapproved at the Rochester final action hearings. Several are actually identical to those previously defeated. These are prime examples of a disturbing trend in the IBC to eliminate basic construction code requirements from the IBC. There has been a wholesale effort to take critical elements out of our base code and move them to reference standards where the voting members of ICC have little or no control over the outcome of any changes. In the reason for S190 the proponent is wanting to remove numerous provisions from the ICC process thereby "reducing the burdens on code officials in overseeing often esoteric, technical or specialized provisions". Please don't do code officials any such favor.

Once a requirement moves out of ICC's system what hope is there we will regain our opportunity to effect positive change in the public interest? The very reason ICC exists is because of our efforts to bring together of all the stakeholders in a single forum to discuss, debate and decide in a process where the outcome is determined by those with no financial interest.

The proponent agues the technical requirements are still incorporated through the reference standard, but the practicality of use and enforcement becomes extremely cumbersome, costly and less effective. Needed information must be readily available to enforcement personnel. Is it really a good idea to eliminate the basic requirements for grout, reinforcement, corrosion protection, mortar, wall ties, cold and hot weather protection, seismic resistance, and columns for light frame construction from the IBC? ICC does not even test on the reference standards for certifications. Shouldn't a certified inspector or plans examiner be able to answer simple questions about these subjects?

The voting members must take a stand and disapprove these changes to ensure the IBC is a useful tool for code officials to help protect the health, safety and welfare of our citizens.

Public Comment 3:

Can Xiao, City of Phoenix, AZ, representing herself, requests Disapproval.

Commenter's Reason: Compared to section IBC 2109, Masonry code has much less information about detailing and construction requirement. Those requirements are important to maintain the construction quality and protect public safety. They need to stay.

Final Action: AS AM AMPC____ D

S191-07/08

2109.1, 2109.2.3.1, Table 2109.3.2, 2109.4.1, Figure 2109.4.1, 2109.4.2, 2109.6.2.1, 2109.6.2.2, 2109.6.3.1, 2106.3.1.1, 2109.6.3.2, 2109.7.3.2, 2109.7.3.3, 2109.7.3.4 (New), 2109.8.4.7

Proposed Change as Submitted:

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards

1. Revise as follows:

2109.1 General. Empirically designed masonry shall conform to this chapter or Chapter 5 of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

2109.2.1.1 Cumulative length of shear walls. In each direction in which shear walls are required for lateral stability, shear walls shall be positioned in at least two separate planes. The minimum cumulative length of shear walls provided shall be 0.4 ~~times~~ multiplied by the long dimension of the building. Cumulative length of shear walls shall not include openings or any element with a length that is less than one-half its height.

2109.2.3.1 Strength. Dry-stacked, surface-bonded concrete masonry walls shall be of adequate strength and proportions to support all superimposed loads without exceeding the allowable stresses listed in Table 2109.2.3.1. Allowable stresses not specified in Table 2109.2.3.1 shall comply with the requirements of ~~ACI 530/ASCE 5/TMS 402~~ TMS 402/ACI 530/ASCE 5.

TABLE 2109.3.2
ALLOWABLE COMPRESSIVE STRESSES FOR EMPIRICAL DESIGN OF MASONRY

CONSTRUCTION; COMPRESSIVE STRENGTH OF UNIT GROSS AREA (psi)	ALLOWABLE COMPRESSIVE STRESSES[a] GROSS CROSS-SECTIONAL AREA (psi)	
	Type M or S mortar	Type N mortar
Solid masonry of brick and other solid units of clay or shale; sand-lime or concrete brick:		
8,000 or greater	350	300
4,500	225	200
2,500	160	140
1,500	115	100
Grouted masonry, of clay or shale; sand-lime or concrete:		
4,500 or greater	225	200
2,500	160	140
1,500	115	100
Solid masonry of solid concrete masonry units:		
3,000 or greater	225	200
2,000	160	140
1,200	115	100
Masonry of hollow load-bearing units of clay or shale:		
2,000 or greater	140	120
1,500	115	100
1,000	75	70
700	60	55
Masonry of hollow load bearing concrete masonry units, up to and including 8 in. nominal thickness:		
2000 or greater	140	120
1500	115	100
1000	75	70
700	60	55
Masonry of hollow load bearing concrete masonry units, greater than 8 and up to 12 in. nominal thickness:		
2000 or greater	125	110
1500	105	90
1000	65	60
700	55	50
Masonry of hollow load bearing concrete masonry units, 12 in. nominal thickness and greater:		
2000 or greater	115	100
1500	95	85
1000	60	55
700	50	45
Hollow walls (noncomposite masonry bonded)[b]		
Solid units:		
2,500 or greater	160	140
1,500	115	100
Hollow units of clay or shale	75	70
Hollow units of concrete masonry of nominal thickness		
up to and including 8 in.	75	70
greater than 8 and up to 12 in.	70	65
12 in.and greater:	60	55
Stone ashlar masonry:		
Granite	720	640
Limestone or marble	450	400
Sandstone or cast stone	360	320
Rubble stone masonry		
Coursed, rough or random	120	100

For SI: 1 pound per square inch = 0.006895MPa.

a. Linear interpolation for determining allowable stresses for masonry units having compressive strengths which are intermediate between those given in the table is permitted.

b. Where floor and roof loads are carried upon one wythe, the gross cross-sectional area is that of the wythe under load; if both wythes are loaded, the gross cross-sectional area is that of the wall minus the area of the cavity between the wythes. Walls bonded with metal ties shall be considered as noncomposite walls unless collar joints are filled with mortar or grout.

2109.4.1 ~~Intervals~~ Maximum *l/t* and *h/t*. Masonry walls without openings shall be laterally supported in either the horizontal or vertical direction such that *l/t* or *h/t* does ~~at intervals~~ not exceeding ~~those~~ the values given in Table 2109.4.1. Masonry walls with single or multiple openings shall be laterally supported in either the horizontal or vertical direction such that *l/t* or *h/t* does not exceed the values given in Table 2109.4.1 divided by $\sqrt{W_T/W_S}$.

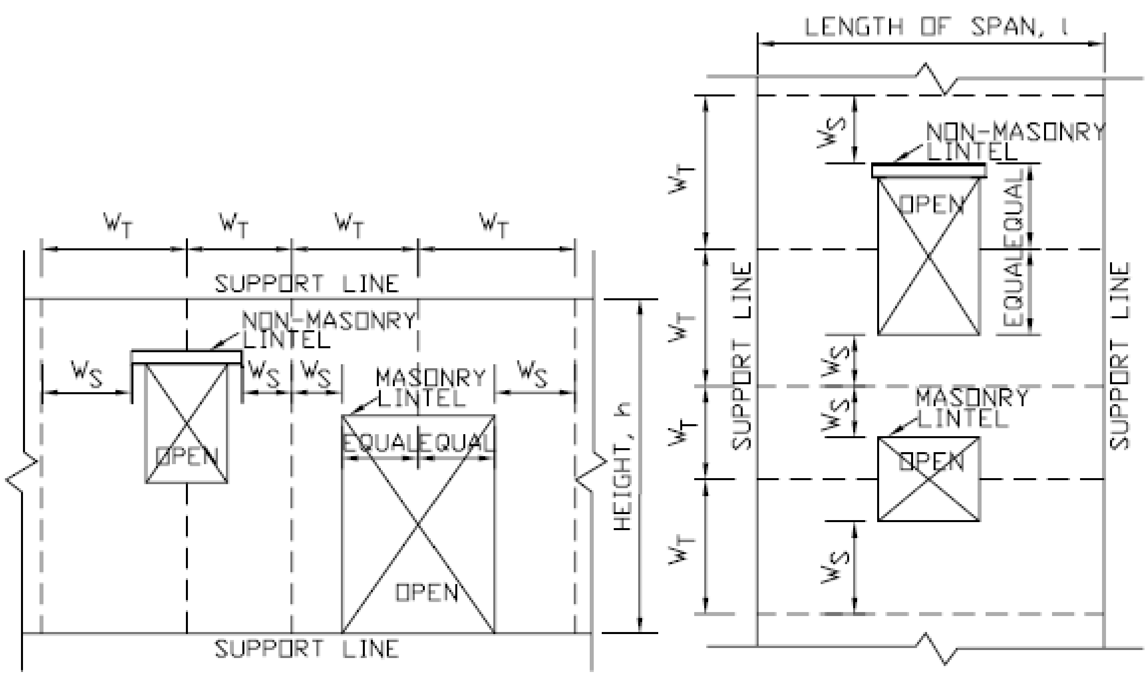
W_S is the dimension of the structural wall strip measured perpendicular to the span of the wall strip and perpendicular to the thickness as shown in Figure 2109.4.1. W_S is measured from the edge of the opening. W_S shall be no less than 3*t* on each side of each opening. Therefore, at walls with multiple openings, jambs shall be no less than 6*t* between openings. For design purposes, the effective W_S shall not be assumed to be greater than 6*t*. At non-masonry lintels, the edge of the opening shall be considered the edge of the non-masonry lintel. W_S shall occur uninterrupted over the full span of the wall.

W_T is the dimension, parallel to W_S, from the center of the opening to the opposite end of W_S as shown in Figure 2109.4.1. Where there are multiple openings perpendicular to W_S, W_T shall be measured from the center of a virtual opening that encompasses such openings. Masonry elements within the virtual opening must be designed in accordance with Section 2107 or 2108.

For walls with openings that span no more than 4 feet (1219 mm), parallel to W_S, if W_S is no less than 4 feet (1219 mm), then it shall be permitted to ignore the effect of those openings.

The span of openings, parallel to W_S, shall be limited such that the span divided by *t* does not exceed the values given in Table 2109.4.1.

In addition to these limitations, lintels shall be designed for gravity loads in accordance with Section 2109.8.4.7



W_S and W_T for Walls Spanning Vertically

W_S and W_T for Walls Spanning Horizontally

FIGURE 2109.4.1
GRAPHICAL REPRESENTATION OF W_S AND W_T

TABLE 2109.4.1
WALL LATERAL SUPPORT REQUIREMENTS

CONSTRUCTION	MAXIMUM WALL LENGTH TO THICKNESS OR WALL HEIGHT TO THICKNESS
Bearing walls	
Solid units or fully grouted	20
Other than solid units or fully grouted ~~All other~~	18
Nonbearing walls	
Exterior	18
Interior	36

2109.4.2 Thickness. ~~Except for cavity walls and cantilever walls, the thickness of a wall shall be its nominal thickness measured perpendicular to the face of the wall. For cavity walls, the thickness shall be determined as the sum of the nominal thicknesses of the individual wythes.~~ For cantilever walls, except for parapets, the ratio of height-to-nominal thickness shall not exceed 6 for solid masonry or 4 for hollow masonry. For parapets, see Section 2109.5.4.

2109.6.2.1 Solid units. Where ~~the facing and backing (~~adjacent wythes~~)~~ of solid masonry ~~construction~~ walls are bonded by means of masonry headers, no less than 4 percent of the wall surface area of each face shall be composed of headers extending not less than 3 inches (76 mm) into ~~the backing~~ each wythe. The distance between adjacent full-length headers shall not exceed 24 inches (610 mm) either vertically or horizontally. In walls in which a single header does not extend through the wall, headers from the opposite sides shall overlap at least 3 inches (76 mm), or headers from opposite sides shall be covered with another header course overlapping the header below at least 3 inches (76 mm).

2109.6.2.2 Hollow units. Where two or more wythes are constructed using hollow units ~~are used to make up the thickness of a wall~~, the stretcher courses shall be bonded at vertical intervals not exceeding 34 inches (864 mm) by lapping at least 3 inches (76 mm) over the unit below, or by lapping at vertical intervals not exceeding 17 inches (432 mm) with units that are at least 50 percent greater in thickness than the units below.

2109.6.3.1 Bonding with wall ties. Except as required by Section 2109.6.3.1.1, where ~~the facing and backing (~~adjacent wythes~~)~~ of masonry walls are bonded with wire size W2.8 (MW18) wall ties or metal wire of equivalent stiffness embedded in the horizontal mortar joints, there shall be at least one metal tie for each $4\frac{1}{2}$ square feet (0.42 m^2) of wall area. The maximum vertical distance between ties shall not exceed 24 inches (610 mm), and the maximum horizontal distance shall not exceed 36 inches (914 mm). Rods or ties bent to rectangular shape shall be used with hollow masonry units laid with the cells vertical. In other walls, the ends of ties shall be bent to 90-degree (1.57 rad) angles to provide hooks no less than 2 inches (51 mm) long. Wall ties shall be without drips. Additional bonding ties shall be provided at all openings, spaced not more than 36 inches (914 mm) apart around the perimeter and within 12 inches (305 mm) of the opening.

2109.6.3.1.1 Bonding with adjustable wall ties. Where ~~the facing and backing (~~adjacent wythes~~)~~ of masonry are bonded with adjustable wall ties, there shall be at least one tie for each 1.77 square feet (0.164 m^2) of wall area. Neither the vertical nor horizontal spacing of the adjustable wall ties shall exceed 16 inches (406 mm). The maximum vertical offset of bed joints from one wythe to the other shall be 1¼ inches (32 mm). The maximum clearance between connecting parts of the ties shall be $\frac{1}{16}$ inch (1.6 mm). When pintle legs are used, ties shall have at least two wire size W2.8 (MW18) legs.

2109.6.3.2 Bonding with prefabricated joint reinforcement. Where ~~the facing and backing (~~adjacent wythes~~)~~ of masonry are bonded with prefabricated joint reinforcement, there shall be at least one cross wire serving as a tie for each 2 2/3 square feet (0.25m^2) of wall area. The vertical spacing of the joint reinforcing shall not exceed 24 inches (610 mm). Cross wires on prefabricated joint reinforcement shall not be less than W1.7 (MW11) and shall be without drips. The longitudinal wires shall be embedded in the mortar.

2109.7.3.2 Steel ~~floor~~ joists. Steel ~~floor~~ joists ~~bearing on~~ masonry walls shall bear on and be connected to steel bearing plates. Maximum joist spacing shall be 6 ft (1829 mm) on center. Each bearing plate shall be anchored to the wall with ~~3/8-inch (9.5 mm) round bars~~ a minimum of two ½ in. (12.7 mm) diameter bolts, or their equivalent~~, spaced not more than 72 inches (1829 mm) o.c.~~ Where steel joists are parallel to the wall, anchors shall be located ~~at~~ where joist bridging terminates at the wall and additional anchorage shall be provided to comply with Section 2109.7.3.3.

2109.7.3.3 Roof and floor diaphragms. Roof and floor diaphragms shall be anchored to masonry walls with ~~1/2-inch-diameter~~ a minimum of ½ in. (12.7 mm) bolts, at a maximum spacing of 72 inches (1829 mm) o.c. or their equivalent. ~~Bolts shall extend and be embedded at least 15 inches (381 mm) into the masonry, or be hooked or welded to not less than 0.20 square inch (129 mm2) of bond beam reinforcement placed not less than 6 inches (152 mm) from the top of the wall.~~

2109.7.3.4 Bolts and anchors. Bolts and anchors required by Sections 2109.7.3.2 and 2109.7.3.3 shall comply with the following:

1. Bolts and anchors at steel floor joists and floor diaphragms shall be embedded in the masonry at least 6 in. (152 mm) or shall comply with Section 2109.7.3.4, item 3.
2. Bolts at steel roof joists and roof diaphragms shall be embedded in the masonry at least 15 in. (381 mm) or shall comply with Section 2109.7.3.4, item 3.
3. In lieu of the embedment lengths listed in Sections 21097.3.4, item 1 and 2109.7.3.4, item 2, bolts shall be permitted to be hooked or welded to not less than 0.20 in.2 (129 mm^2) of bond beam reinforcement placed not less than 6 in. (152 mm) below joist bearing or bottom of diaphragm.

2109.8.4.7 Lintels. Lintels shall be considered structural members and shall be designed in accordance with the applicable provisions of Chapter 16 and Sections 2107 or 2108.

Reason: The revisions proposed in this code change reflect editorial and substantive revisions incorporated into the 2008 edition of the Building Code Requirements for Masonry Structures (TMS 402/ACI 530/ASCE 5), commonly referred to as the Masonry Standard Joint Committee (MSJC) Code. This code change proposal is one of several to harmonize the design and construction requirements for masonry within the IBC with those in the reference standard. A complete list of revisions incorporated into the reference standard is available for download at www.masonrystandards.org.

Specific revisions proposed above include:

Section 2109 has undergone significant revisions during the 2008 Code Cycle. The IBC contains these same provisions transcribed from the MSJC and these sections of the IBC need to have the same revisions made to them to avoid a conflict between the code and the standard. The most significant of these changes serves to place additional limitations on the use of empirical design for walls with openings.

Cost Impact: The code change proposal will not increase the cost of construction.

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**

Individual Consideration Agenda

This item is on the agenda for individual consideration because public comments were submitted.

Public Comment 1:

Phil Samblanet and Jason Thompson, The Masonry Society and National Concrete Masonry Association, representing The Masonry Society and The Masonry Alliance for Codes and Standards, request Approval as Submitted.

Commenter's Reason:. If code change proposal S175-07/08 is ultimately approved as modified as recommended by the committee, this change proposal becomes redundant and unnecessary.

Public Comment 2:

Mark W. Cunningham, Earth Tech, representing himself, requests Disapproval.

Commenter's Reason: S190-07/08, Approved as Submitted, deletes the empirical masonry design provisions section from IBC, and refers to ACI 530 for empirical design instead. SECTION 2109 EMPRICAL DESIGN OF MASONRY is changed to SECTION 2109 SURFACE-BONDED MASONRY. Therefore, S191-07/08, which revises the original IBC SECTION 2109 empirical design provisions cannot be Approved as Submitted, because those provisions no longer exist in IBC.

Final Action: AS AM AMPC____ D

S192-07/08
2110.2.2, Figure 2110.3.1

Proposed Change as Submitted:

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards

Revise as follows:

2110.3.3 Interior panels. <u>Where the wind pressure does not exceed 10 psf (480 Pa), </u>~~T~~<u>t</u>he maximum area of each individual standard-unit panel shall be 250 square feet (23.2 m^2)~~. The~~ <u>and the</u> 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<u>. 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</u>
<u>Section 2110.3.2.</u>

FIGURE 2110.3.1
GLASS <u>UNIT</u> MASONRY DESIGN WIND LOAD RESISTANCE

(Portions of figure not shown remain unchanged)

Reason: The revisions proposed in this code change reflect editorial and substantive revisions incorporated into the 2008 edition of the Building Code Requirements for Masonry Structures (TMS 402/ACI 530/ASCE 5), commonly referred to as the Masonry Standard Joint Committee (MSJC) Code. This code change proposal is one of several to harmonize the design and construction requirements for masonry within the IBC with those in the reference standard. A complete list of revisions incorporated into the reference standard is available for download at www.masonrystandards.org.

Specific revisions proposed above include:

Section 2110 has had minor revisions during the 2008 Code Cycle. The IBC contains these same provisions transcribed from the MSJC and these sections of the IBC need to have the same revisions made to them to avoid a conflict between the code and the standard.

Cost Impact: The code change proposal will not increase the cost of construction.

Committee Action: **Approved as Modified**

Modify proposal as follows:

2110.3.3 Interior panels. <u>For interior panels w</u>here the wind pressure does not exceed 10 psf (480 Pa), <u>t</u>he 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**

Individual Consideration Agenda

This item is on the agenda for individual consideration because public comments were submitted.

Public Comment 1:

Phil Samblanet and Jason Thompson, The Masonry Society and National Concrete Masonry Association, representing The Masonry Society and The Masonry Alliance for Codes and Standards, requests Approval as Modified by the Code Committee as Published in the ROH.

Commenter's Reason: If code change proposal S175-07/08 is ultimately approved as modified as recommended by the committee, this change proposal becomes redundant and unnecessary. .

Public Comment 2:

Mark W. Cunningham, Earth Tech, representing himself, requests Disapproval.

Commenter's Reason: S190-07/08, Approved as Submitted, deletes the glass unit masonry design provisions from IBC, and refers to ACI 530 for glass unit masonry design instead. SECTION 2110 GLASS UNIT MASONRY is changed to SECTION 2110 ADOBE MASONRY. Therefore, S192-07/08, which revises the original IBC SECTION 2110 glass unit masonry design provisions cannot be Approved as Modified, because those provisions no longer exist in IBC.

Final Action: AS AM AMPC____ D

S207-07/08

2209.2, Chapter 35 (New)

Proposed Change as Submitted:

Proponent: Bonnie Manley, American Iron and Steel Institute, representing Steel Deck Institute

1. Revise as follows:

2209.2 Steel decks. The design and construction of cold-formed steel decks shall be in accordance with this section.

2209.2.1 Composite slabs on steel decks. Composite slabs of concrete and steel deck shall be permitted to be designed and constructed in accordance with ASCE 3 or ANSI/SDI-C1.0.

2209.2.2 Non-composite steel floor decks. Non-composite steel floor decks shall be permitted to be constructed in accordance with ANSI/SDI-NC1.0.

2209.2.3 Steel roof deck. Steel roof decks shall be permitted to be designed and constructed in accordance with ANSI/SDI-RD1.0.

2. Add standards to Chapter 35 as follows:

Steel Deck Institute

C1.0–06	Standard for Composite Steel Floor Deck
NC1.0–06	Standard for Non-Composite Steel Floor Deck
RD1.0–06	Standard for Steel Roof Deck

Reason: This code change proposal introduces three new SDI standards on cold-formed steel decks. It is intended that users be permitted to use these documents in lieu of the more formal approach of AISI S100, *North American Specification for the Design of Cold-Formed Steel Structural Members.* The scope of the documents is as follows:

ANSI/SDI C1.0: "This Specification for Composite Steel Deck shall govern the materials, design, and erection of cold formed steel deck which acts as a permanent form and as positive reinforcement for a structural concrete slab."

ANSI/SDI NC1.0: "This Specification for Non-Composite Steel Floor Deck shall govern the materials, design, and erection of cold formed non-composite steel deck used as a form for reinforced concrete slabs."

ANSI/SDI RD1.0: "This Specification for Steel Roof Deck shall govern the materials, design, and erection of cold formed steel deck used for the support of roofing materials, design live loads and SDI construction loads."

Cost Impact: The code change proposal will not increase the cost of construction.

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**

Individual Consideration Agenda

This item is on the agenda for individual consideration because public comments were submitted.

Public Comment 1:

Dr. Salah Altoubat, University of Sahrjah, UAE, representing himself, requests Approval as Submitted.

Commenter's Reason: What is unanimous about fibers?
Fiber reinforced concrete (FRC) has been emerged as a promising construction material with proven field performance in a variety of applications such as slabs on ground, pavement, precast panels and walls, Insulated concrete forms construction, septic tanks, pipes, tunnel linings and stabilization for underground construction. It is widely recognized that fibers enhance engineering properties of concrete, which includes <u>flexural toughness, impact resistance, cracking resistance and post-cracking load carrying capacity</u>.

What is a well established function of fibers?
It is well established that fibers control shrinkage cracking. Due to its distributed nature, fibers provide a spatial network of discrete reinforcement that control cracking immediately from the inception of micro-cracks in the concrete. Restrained shrinkage tests performed by a variety of researchers have shown considerable reduction in crack width. <u>Researchers have also established that fibers reduce the shrinkage cracks width and control shrinkage cracking</u>. <u>With this regard, fibers have been proven effective in slabs on ground, industrial flooring and pavement</u>.

What about using fibers in composite deck for crack control?
For composite deck applications, I believe that fibers can effectively be applied to control shrinkage and temperature cracking in the transverse direction normal to the corrugation. I also believe that FRC can effectively be applied as a substitute for welded wire mesh in composite steel decks. <u>The nature of the deck restraint at the bottom of the concrete slab</u> makes fibers an effective and attractive alternative because the amount of mobilized deformation contributed to a single crack due to temperature and shrinkage of concrete will be very much limited, <u>and significant multiple fine cracking will be facilitated by the deck</u>. Therefore the width of cracks will be sufficiently small. Since the suppression of cracks is a function of the volume fraction and type of fibers, <u>it is quite possible that the amount of fibers needed to control width of cracks to an acceptable level will be small</u> compared to that for other applications such as slabs on ground or pavement. This is also explained by the small percentage of steel 0.075% specified by the SDI.

Will the aggregate interlock be at risk when fibers are used?
The aggregate interlock and transverse distribution of load will not be an issue in composite decks again because the crack width is expected to be small due to the nature of the restraint at the bottom of the slab. Aggregate interlock is a function of crack width and research studies have shown that aggregate interlock almost vanished when the crack width exceeded 1.5 mm. The TR34 design guidelines "Concrete Industrial Ground Floors – A Guide to Their Design and Construction" for slabs on ground consider aggregate interlock to vanish as the crack opening approached 2 mm. The crack width due to temperature and shrinkage in composite deck <u>will be far below such critical levels due to the nature of restraint</u>. Therefore, <u>aggregate interlocks for shear transfer and load distribution in composite decks will not be compromised when fibers are used.</u>

Where performance stands when favoring WWF over FRC or vice versa?
Neither WWF nor FRC is intended to replace primary reinforcement in composite metal deck. Welded wire mesh has been traditionally used to control shrinkage and thermal cracking in composite decks. The WWF will be only engaged when the crack reached the level of the WWM. The location of the WWF within the slab depth influences its capability to control cracking, and that steel cover requirement will apply. This limitation will not be applied to fibers because of the distributed nature of the fibers and its three dimensional reinforcement capabilities within the slab. Since both WWF and fibers are not primarily used for structural purpose, the use of one over the other is reduced to economics and constructability issues. <u>However, performance is an issue that we must address</u>

Do we have performance test?
Although WWF has been traditionally used to control cracking, it relies mostly on its historical use, and not on published research, as justification for its use. On the other hand, fibers have been used in many applications <u>based on proven performance</u> and there are several standard tests to characterize the post-cracking strength of FRC, such as RILEM T162, ASTM C1609, and ASTM C1399. Each standard test provides important information that can be used to judge the post-cracking performance for FRC and thus can be engaged in design guidelines that have been evolving in recent years. The choice of WWF or fibers must be based on performance and not only in historical use or even perception. <u>A performance test that compares WWF to FRC with this regard must be established. A tensile post-cracking test can be, in principle, devised for both FRC and WWF to judge performance</u>.

FRC industry is moving beyond crack control to structural applications: Should we consider that?
In recent years, there has been a growing recognition that fibers offer structural contribution. Such structural contributions have been demonstrated by large scale testing of structural elements such as beams, slabs, bridge decks, joints, ..etc. Research on FRC has changed direction over the past ten years toward establishing design tools to include fiber reinforcement in the design of structures. Several design guidelines were developed to design structures with <u>FRC such as RILEM, TC 162, German Guidelines (DBV), Italian Guidelines, and TR34-2003</u>. International platforms were also organized to discuss design of FRC structures, among them the international workshop on FRC held in Italy (2004) and organized by RILEM, where North American and European experience were presented and discussed.

<u>If the industry is extending the use of fibers beyond crack control to structural applications, I believe we should not move a step back and stop fibers in composite deck for non-structural purposes</u>. However, we should continue working to agree on a performance test that will be suitable for metal deck such as a tensile post-cracking test and will be used as a basis for acceptance of either FRC or WWF or any other new reinforcing system.

Public Comment 2:

Nemy Banthia, Ph.D, P.Eng, representing himself, requests Approval as Submitted.

Commenter's Reason: The current version of IBC references ASCE 3-91 for design of composite metal deck. Unfortunately, ASCE 3-91 is obsolete and out of print. With a growing interest in the use of composite metal decks, an alternative is needed, and the ANSI/SDI code fills this gap. The ANSI/SDI code allows the use of steel or macro synthetic fibers as reinforcement to combat temperature and shrinkage related stresses. SDI also ventures into determining the appropriate levels of fiber dosage required to provide suitable reinforcement.

Objections were raised at the last code hearings that fibers are incapable of providing minimum temperature and shrinkage reinforcement, and that the resultant crack widths in fiber reinforced concrete would be substantially wider than those with concrete reinforced with WWF. Further, allegations were made that fibers should not be used in such decks because they may not be suitably inspected.

I find these objections unfounded and can confidently state that based on my personal research and experience, fibers can, in fact, provide a superior alternative to WWF reinforcement in composite metal decks. Fibers can also be easily inspected. I base these statements on the following facts:

1. Fibers are uniformly dispersed in concrete and hence provide a homogenous, volumetric reinforcement. This not only improves the fracture toughness and crack growth resistance of concrete but also enhance its fatigue endurance, abrasion resistance and energy absorption capacity under impact and impulsively applied loads. While plain concrete has no residual strength after cracking, fiber reinforced concrete has high post-cracking residual strength on account of fiber bridging and energy-intensive fiber pull-out processes. Cracks in fiber reinforced concrete--if they at all occur--remain narrow and high post-crack strength prevents them from growing.

   In the case of WWF, on the other hand, WWF only reinforces “locally” and is effective as reinforcement only under a prescribed external “structural” loading. Away from the WWF reinforcement, concrete is essentially unreinforced, brittle, easy to crack, and with poor durability.

2. Fibers are highly effective in controlling shrinkage cracking in concrete. A comprehensive study recently carried out at the University of British Columbia (1,2,3,4) clearly demonstrated the effectiveness of fibers in controlling shrinkage cracking under harsh environmental conditions including low humidity and high temperatures. In Figures 1a and b some date related to five types of fibers are plotted. Notice that depending on the type of fiber, cracking in concrete can be completely eliminated. In Figure 2, some photographs showing crack control effectiveness of fibers are presented.

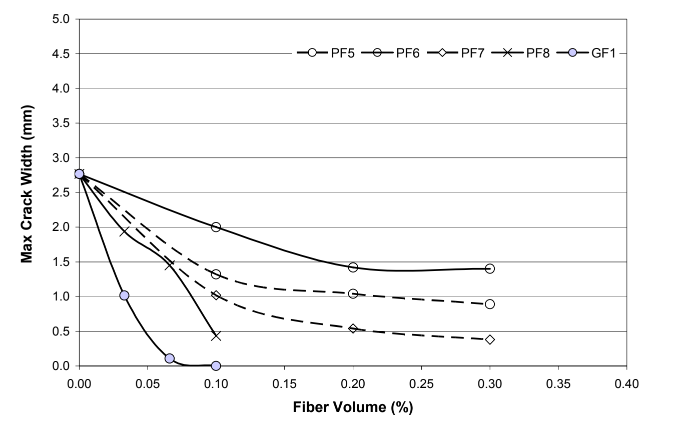


Fig. 1. Maximum Crack Width vs. Fiber Volume Fraction. PF5-8 and GF1 are different fiber types.

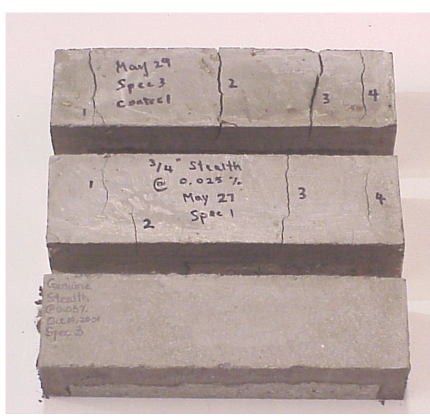
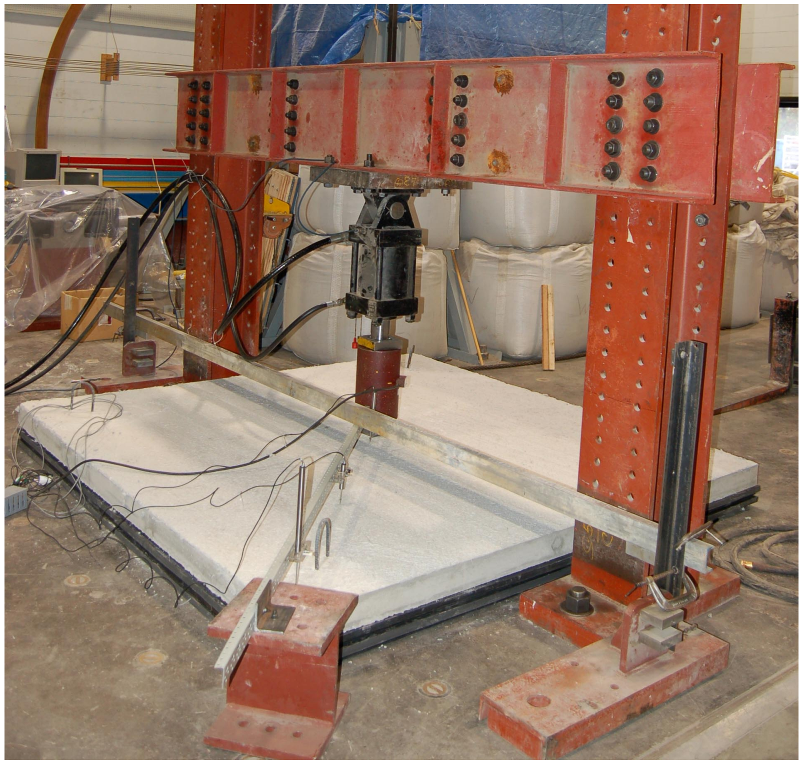


Figure 2. Control of Shrinkage Cracking with Fiber Reinforcement (Top: Plain; Middle: 0.1% Fiber; Bottom: 0.2% of Polymeric Fiber



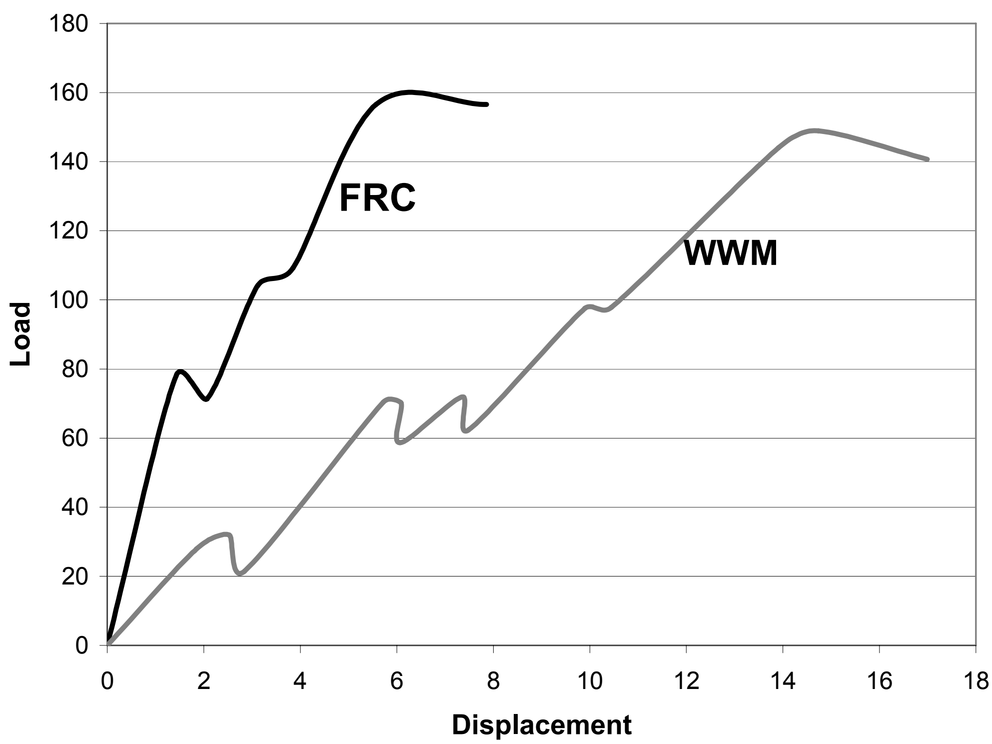


Figure 3 (a). Full-Scale Load Test on Slab-on Grade and 3(b) Load-Displacement Curves.

3. Even based on full-scale structural tests, the improved performance of slabs with fiber reinforcement over those with WWF can be seen. Some full-scale structural load responses are shown in Figure 3 (5). In these tests, 2.5 m x 2.5m x 0.1m slabs-on-grade were tested under a centrally applied load. One slab was reinforced with a welded wire mesh (WWM, or WWF) and the other with polymeric fiber at 3lb/yd^3. Rubber pads were used to provide a sub-base with a uniform elastic reaction. Results indicated that the slab with fiber reinforcement supported a higher load at failure and also depicted a stiffer response.

4. Finally, a few words on durability. The most important and pressing issue with decks reinforced with WWF today is the rampant corrosion of WWF that occurs due to ingress of chlorides. Corrosion of WWF in deck slabs has become a multi-billion dollar problem and one wonders why we continue to use this archaic type of reinforcement when superior alternatives (such as fiber reinforcement) are available. Fiber reinforcement is a far superior option in so far as durability is concerned as fiber reinforcement reduces concrete permeability under both unstressed and stressed conditions (Figure 4, Ref. 6) and even reduces the corrosion rates of reinforcing steel, in present (7).

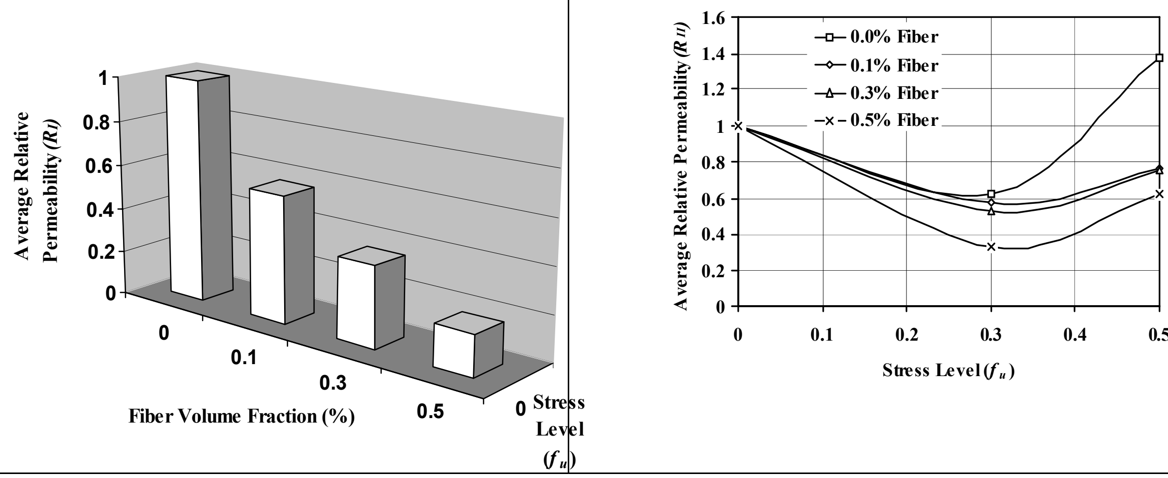


Figure 4. (a) Influence of Fiber Reinforcement on Permeability of Unstressed Concrete and (b) Influence of Stress on Relative Permeability of Plain and Fiber Reinforced Concrete. Notice the impermeable nature of fiber reinforced concrete under both stressed and unstressed conditions

References:
1, Banthia, N. and Gupta, R. Test Method for Evaluation of Plastic Shrinkage Cracking in Fiber Reinforced Cementitious Materials, Experimental Techniques, Nov-Dec 2007, pp. 44-48.
2. Banthia, N. and Gupta, R., Influence of Polypropylene Fiber Geometry on Plastic Shrinkage Cracking in Concrete, Cement and Concrete Research, 36 (7), July 2006, pp. 1263-1267.
3. Banthia, N. and Yan, C., Shrinkage Cracking in Polyolefin Fiber Reinforced Concrete, ACI Materials Journal, 97(4), 2000, pp. 432-437.
4. Banthia, N. and Gupta, R., Repairing with Fiber Reinforced Concrete Repairs, ACI Concrete International, 28(11), Nov 2006, pp. 36-40.
5. Banthia, N., Comparison of Load Carrying Capacity of Slab-on-Grade with Fibermesh 650 Fiber and Slab-on-Grade with Welded Wire Mesh Reinforcement, Technical Report, The University of British Columbia, Vancouver, Canada 2006.
6. Banthia, N. and Bhargava, A., Permeability of Stressed Concrete and Role of Fiber Reinforcement, American Concrete Institute, Materials Journal, 104(1), Jan-Feb, 2007, pp. 303-309.
7. Bhargava, A. and Banthia, N., Permeability of Concrete with Fiber Reinforcement and Service Life Predictions, RILEM, Materials and Structures, 41, Jan 2008, pp. 363-372.

Public Comment 3:

Timothy A. Durning, P.E., Grace Construction Products, requests Approval as Submitted.

Commenter's Reason: The current IBC references ASCE 3-91 for design of composite metal deck. This standard is obsolete and out of print. An alternative is needed and ANSI/SDI-C1.0 fills this gap. ANSI/SDI-C1.0 does include provisions where steel or macro synthetic fibers may be used to satisfy requirements for minimum temperature and shrinkage reinforcement at the discretion of the design engineer. Although this provision caused controversy at the Code Committee hearing, SDI performed the necessary diligence to determine appropriate levels of fiber dose to provide equivalent performance to that provided by the minimum specified levels of WWF.

It is the position of Grace Construction Products that macro synthetic and steel fibers are both suitable substitutes for WWF as reinforcement to control temperature and shrinkage cracking in composite steel floor decks as called for in ANSI/SDI-C1.0. This position is based upon the numerous research and technical papers which illustrate the effectiveness of macro fibers in controlling shrinkage cracking and reducing crack widths in concrete; as well as the abundant body of field performance data from around the world where fibers are used to provide minimum reinforcing in a myriad of applications.

Globally, fibers gained early acceptance as a replacement for heavy mesh reinforcement in tunnel and mine shaft shotcrete linings. Due to performance, economics and enhanced safety they have become the standard for providing crack control for concrete in underground construction worldwide. Above ground, macro fibers are now used in various parts of the world to provide minimum reinforcing, and in some case structural reinforcing for: slabs on ground, slab on piles, various thin walled precast concrete items, slope stabilization, drainage channels, pond linings and overlay and full depth pavements for roadways, airfields, parking lots, and numerous other applications. Millions of cubic yards of concrete structures have been designed and constructed successfully using macro synthetic and steel fibers. Of particular interest is that the UK and northern Europe have been successfully using macro fibers in compliance with local building codes for composite steel floor deck for the last 5 years.

These fibers provide a spatial network of discrete reinforcement that control cracking immediately from the inception of micro-cracks in the concrete. Fibers help maintain the integrity of the concrete materials and transfer thermal and shrinkage induced stresses. Since fibers dispersed throughout the concrete matrix have reinforcement capabilities in all directions, they have shown considerable reduction in crack width in restrained shrinkage tests as performed by a variety of researchers.

Below is a listing of references along with a brief summary of published technical research papers that clearly illustrate the value and effectiveness of fibers in reducing the crack widths and controlling shrinkage cracking in many applications.
Shah and co-workers used the ring test to study the influence of steel, synthetic and cellulose fibers on cracking of concrete along with welded wire mesh and concluded that addition of small amount of these fibers significantly reduced the crack width.

Shah, S.P., Sarigaphuti, M., and Karaguler, M.E., "Comparison of shrinkage cracking performance of different types of fibres and wiremesh", Fiber Reinforced Concrete Developments and Innovations, ACI, SP-142, 1–18, 1994.

Grzybowski and Shah have looked at the crack width of restrained concrete rings and found that steel fibers and polypropylene fibers were effective in controlling shrinkage cracking, reducing crack width significantly and facilitated multiple cracking.
Grzybowski, M., and Shah, S. P., "Shrinkage Cracking of Fiber Reinforced Concrete," ACI Materials Journal, 87 (2) 138-148, 1990.

Banthia and Yan have also looked at the crack width of concrete slab specimens cast over a hard substrate to represent slabs restrained at the bottom (overlays), and found that crack width reduced from 1.1 mm to 0.5 mm when 0.3% of polyolefin macro fibers were added to concrete.
Banthia, N., and Yan, C., "Shrinkage cracking in polyolefin fiber reinforced concrete," ACI Materials Journal 97(4) 432–437, 2000.

Carlswärd studied the effect of steel fibers on shrinkage induced cracking in thin concrete overlays and showed that crack widths may be reduced quite significantly by the addition of steel fibers.
Carlswärd, J., "Shrinkage cracking of steel fibre reinforced self compacting concrete overlays-Test methods and theoretical modeling", Doctoral Thesis, Luleå University of Technology; Department of Civil and Environmental Engineering, 2006.

Voigt et al (2004) have compared the performance of welded wire mesh and fibers in controlling shrinkage cracking of restrained concrete rings. The results suggested that fibers and welded wire mesh provide similar shrinkage crack control. For example, the W1.4 x W1.4 6 x 6 (a common mesh in composite decks) reduced the shrinkage crack width from 1.07 mm to 0.17 mm. The addition of 0.25% of steel fibers reduced crack width to about 0.14 to 0.21 mm and the addition of 0.3% of crimped polypropylene fiber reduced crack width to 0.4 mm.
Voigt, T., Bui, Van K., and Shah,S.P., "Drying Shrinkage of Concrete Reinforced with Fibers and Welded-Wire Fabric" ACI Materials Journal 234-241, 2004

Altoubat (2007) looked at the ability of synthetic fibers to control shrinkage cracking in composite floor systems under severe loading condition. The study monitored the crack width over the middle support of two-span continuous floor system under sustained loading (highly stressed area due to combined shrinkage and loading). The crack width for the fiber slab was in the order of 0.4 mm. The results showed that synthetic fibers can effectively limit the crack width in composite floors to an acceptable limit.
Altoubat, S. A. " Mitigating shrinkage cracking of concrete using shrinkage reducing admixtures and synthetic macro fibers," Presented in the 5th International Conference on Concrete Under Sever Conditions Environment and Loading (CONSEC'07), Tours, France, June 2007

A study of slab on ground by Bischoff and co-workers indicated that steel fibers are a suitable alternative to properly positioned WWR. The addition of 0.4% of fibrillated polypropylene fibers was found also suitable to replace the welded wire mesh. In the figure the addition of 0.4% of steel fibers is almost equivalent to double layer of steel mesh (0.4%) one at the bottom and one at the top of the slab. Clearly these tests show the value of fibers in terms of crack control at high loading regime much more than would be expected due to temperature and shrinkage in composite decks
Bischoff, P.H., ; Valsangkar, A. J.; and Irving, J, "Use of Fibers and Welded-Wire Reinforcement in Construction of Slabs on Ground", Practice Periodical On Structural Design And Construction (41-46) February 2003

Roesler et al (2006) have tested slabs on ground to failure. The results showed that synthetic macro fibers at 0.33% increased the slab capacity and kept the concrete slabs intact even after significant loading past their ultimate load capacity. While the plain concrete slab failed and broke into several pieces, the cracks of the FRC slab were tight and the slab had to be wetted with water for visibility.
ROESLER, J.R.; ALTOUBAT, S.; LANGE, D.; RIEDER, K.-A.; ULREICH, G., "Effect of Synthetic Fibers on Structural Behavior of Concrete Slabs on Ground". ACI Materials Journal, 103 (1) 3-10, 2006.

Mirza, F.A., Soroushian have shown that, alkali-resistant glass fibers were found to be highly effective in controlling restrained shrinkage cracking of lightweight concrete.
Mirza, F.A., Soroushian, P. "Effects of alkali-resistant glass fiber reinforcement on crack and temperature resistance of lightweight concrete", Cement & Concrete Composites 24 (2002) 223–227

MesbahU, H.A. and Bodin used synthetic and steel fibers to address larger shrinkage cracking observed in recycled aggregate mortars and have shown similar results of reduced crack widths.
MesbahU, H.A. and Bodin, F.B., "Efficiency of polypropylene and metallic fibres on control of shrinkage and cracking of recycled aggregate mortars", Construction and Building Materials 13(439-447), 1999.

Public Comment 4:

Dr. Dean Forgeron, PE, Dalhousie University, requests Approval as Submitted.

Commenter's Reason: A review of the out of date and out of print document (IBC references ASCE 3-91) for the design of composite metal deck revels it should be considered obsolete. An up to date replacement is required to fill this gap. In my opinion an excellent document to fill this gap is the ANSI/SDI code which include provisions for the use of steel or macro synthetic fibers in composite and non composite slabs on steel decks. Steel and macro synthetic fibers have been shown to satisfy the requirements for minimum temperature and shrinkage reinforcement through extensive testing. Based on my research on the structural behavior of macro-synthetic and steel fibers it is clear that SDI has practiced due diligence when determining appropriate fiber dosage levels to not only provide superior structural capacity as the specified WWF but also satisfy the other requirement of this application.

The proposed changes to section 2209.2.1 and 2209.2.2 are not only unfounded but based on allegations made at the committee code hearings " fibers (Steel and Macro-synthetics) are not suitable to provide minimum temperature and shrinkage reinforcement, because they are not capable of holding cracks closed as effectively as WWF" they were made by an uninformed party who has not researched the advances that have been made in fiber reinforced concrete technology and our understanding of its behavior. One of the largest American

Concrete Institute Committees is currently working on several documents to support the use of fiber reinforcement in structural applications to replace considerably more steel reinforcement than used in this application. Many lab and full scale tests support this work. I therefore strongly reject these allegations and confidently state that macro-synthetic and steel fibers, as specified in the existing document, are a superior alternative to the level of WWF specified and provide the added benefit of improved performance, economics, and considerably improvement placement safety.

In addition to the above mentioned benefits the uniform distribution of commercially available macro-synthetic and steel fibers ensures complete reinforcement whereas accurate placement in WWF is very difficult if not impossible as seen by anyone who has cored a WWF reinforced slab and found the reinforcement at the bottom of the slab. When one considers the effectiveness of conventional reinforcement depends strongly on the distance between the surface and the reinforcement level, specifying WWF over fiber reinforcement in this application is a greater risk due to the **high probability/certainty** of WWF reinforcement misplacement. The widespread use of fiber reinforcement in slabs on grade, precast products and shotcrete applications where strict quality control and inspections are required confirms that fibers in concrete are easily and have been successfully inspected like other concrete constituents in commercial applications.

Public Comment 5:

Andrew W Gayer, P.E., S.E., HOK, requests Approval as Submitted.

Commenter's Reason: I support this Code Change Proposal because ASCE 3 has not kept pace with the new trends in the industry. Specifically, SDI has kept abreast of the state of the industry, just like UL, and allowed uniformly distributed macro synthetic fiber reinforcing to be used as temperature and shrinkage reinforcing. Macro synthetic fiber reinforcing is better than welded wire reinforcement for the following reasons:

- Synthetic fiber reinforcing is non-corrosive.
- Synthetic fiber reinforcing is uniformly distributed and thus provides better performance than the discrete system of welded wire reinforcement.
- Synthetic fiber reinforcing is uniformly distributed and thus provides better performance than welded wire reinforcement which often ends up resting directly on the deck and thus cannot resist tension forces on the slab surface. No matter what the Contract Documents or design standards dictate, no Contractor properly supports welded wire reinforcement. Therefore the uniform distribution of synthetic fiber reinforcing is superior to welded wire reinforcement.

Also, because UL recognizes macro synthetic fiber reinforcement for adequate performance in fire situations, the building code also needs to explicitly allow the same fibers for construction of composite slabs.

Public Comment 6:

Mukhtar N. Giader, Opus Architects and Engineers, requests Approval as Submitted.

Commenter's Reason: Referencing ANSI/SDI-C1.0 as an acceptable design standard for composite concrete slabs on steel decks makes sense since SDI is the leading industry expert in this type of construction. In this document, the design professional is given the option of using synthetic fibers for the purpose of controlling cracking due to temperature and shrinkage volume changes. I believe taking away this option will do disservice to the construction industry because it would eliminate or hinder the use of a product that is effective, economic and safe to place. We have used synthetic fibers in our projects for quite some time now and we believe that when engineered properly, it will perform as well as other crack control systems.

Public Comment 7:

Thor Heimdahl, Target Corporation, requests Approval as Submitted.

Commenter's Reason: As a building owner, Target has used synthetic fiber in composite slabs on steel deck, and to this date, has found no reason to believe that the ability of the fibers to resist shrinkage and temperature is compromised when compared to, say, the use of welded wire mesh to achieve the same purpose. So, we want to have the option to decide for ourselves whether or not to use the synthetic fiber in lieu of welded wire reinforcing.

Public Comment 8:

Scott E. Saunders, Ericksen Roed and Associates, requests Approval as Submitted.

Commenter's Reason: We would like to keep the proposed change as submitted (AS) so we can design composite floor slabs per the ANSI/SDI code as it stands. For the design of composite metal decks the current IBC references standard ASCE 3-91 which is obsolete and out of print. An alternative is needed and ANSI/SDI code fills this gap. ANSI/SDI code does include provisions where steel or macro synthetic fibers can be used to satisfy requirements for minimum temperature and shrinkage reinforcement.

A representative for the Wire Reinforcing Institute (WRI) made allegations at the committee code hearings that fibers are not suitable to provide minimum temperature and shrinkage reinforcement, and that resultant crack widths of fiber reinforced concrete would be substantially wider than those with concrete reinforced with WWF. I reject these allegations and can confidently state that based on my experiences that fibers are not only a suitable alternative to WWF, but are in fact a superior alternative when performance and jobsite placement considerations are taken into account. Ericksen Roed & Associates has a staff of sixty-five people that only does structural engineering. We have completed several projects using W.R. Grace Strux 90/40 macro fibers in lieu of WWF and have observed less slab cracking than projects with WWF.

The WRI Representative also made allegations that fibers should not be used because they may not be suitably inspected to determine proper use. When placing concrete with fibers the fibers are quite visible so we know they're included in the mix. When observing post placed concrete, small areas that do not get as much of a hard trowel finish such as around columns and at slab edges, the surfaces show small amounts of protruding fibers. These observances, for inspection purposes, allow the inspector to know the fibers have been included in the concrete mix.

Public Comment 9:

James A. Lane, P.E., KPFF Consulting Engineers, Inc, requests Approval as Submitted.

Commenter's Reason: Fibers ensure every square inch of concrete has temperature and shrinkage control. To many times WWF ends up at the bottom of the deck/slab. We have been utilizing macro fibers now on multiple projects, both elevated slabs on deck and slabs on grade, with successful results. The usage of materials to control temperature and shrinkage control should be left to the engineer in charge to decide whether fiber or WWF should be utilized.

The elimination of not allowing fibers to be utilized places a restriction on the current industry and dampens the growth opportunities of future research for companies in the establishment of the new and ever evolving structural alternatives.

Public Comment 10:

Professor David Lange, University of Illinois, requests Approval as Submitted.

Commenter's Reason: Fiber reinforced concrete (FRC) is an important emerging material for a wide variety of civil engineering structures and pavements. Broadly speaking, fibers enhance concrete properties including: toughness, crack resistance, suppression of crack width, and post-peak load carrying capacity. FRC can be designed for many levels of performance by modifying the concrete mixture, the fiber type, and the fiber volume. FRC is now being integrated into appropriate design guides, standards, and codes, but we must continue to fully develop the potential of FRC as a civil engineering material. In my opinion, the literature and field experience make it abundantly clear that FRC offers important advantages for many applications. I believe that FRC can effectively be applied as a substitute for welded wire mesh for control of temperature and shrinkage cracking in composite steel form decks.

We are currently conducting tests sponsored by WR Grace and in collaboration with Prof. Salah Altoubat, University of Sharjah (UAE), that will allow us to evaluate synthetic macro fibers and welded wire mesh for controlling shrinkage cracking in composite metal decks. We are conducting laboratory restrained tests that demonstrate how fibers and welded wire mesh control crack openings, and we are conducting full scale tests that compare crack widths of concrete with fibers or welded wire mesh when cast over corrugated metal sheets used for stay-in-place forms. The results of this study are expected to be available in late summer 2008.

I request the opportunity to present our findings at the 2008 Final Action Hearings held September 17-23, 2008 in Minneapolis, Minnesota.

Public Comment 11:

Stephen Olsen, Olsen Engineers, requests Approval as Submitted.

Commenter's Reason: Prior to specifying fibers in lieu welded wire fabric (WWF) for shrinkage and temperature reinforcement it has been my experience that placement of the WWF has been difficult for the contractor to place and maintain as we've shown on the drawings – the upper 3rd of the slab. Stepping on the WWF tends to drive it to bottom without a substantial amount of chairs, and the process hooking and pulling the WWF up, after placement of the concrete, as some contactors offer is not acceptable. The fibers are much easier in terms "placement" compared to welded wire fabric.

The slabs on the three projects which I specified fibers, are exposed. In all cases there have been no owner complaints of visible cracks.

Public Comment 12:

Dr. Jean-Francois Trottier, P.Eng., Dalhousie University, representing Fiber Reinforced Concrete Association, requests Approval as Submitted.

Commenter's Reason: First, I would like to preface my comments by stating that I have over 20 years experience investigating the performance, in the laboratory and in the field, of various types of fibers (steel and synthetic) and conventional steel reinforcement. I am the Canada Research Chair in Innovative Materials and currently hold multi-million dollar research grants that specifically focus on developing new construction materials and establishing their suitability for the construction industry. Over the past 10 years, a significant research effort has been placed worldwide to evaluate and establish the credibility of the latest generation synthetic fibers (often referred to as "Structural synthetic or macro fibers"), and steel fibers as potential replacement to conventional steel reinforcement. Over that period, my research group alone has spent over $5 million dollars, mostly on large scale comparison type testing (steel reinforcement vs structural synthetic vs steel fibers) in support of this investigative effort. The resulting research has clearly determined that the new generation synthetic fibers and steel fiber are a suitable replacement to conventional WWF in composite and non composite slabs on steel decks. As a result, many steel deck projects in North America have been completed to date very successfully with fibers.

The IBC references ASCE 3-91 for design of composite metal deck document is out of date. The ANSI/SDI code which includes provisions for the use of steel or macro synthetic fibers in composite and non composite stabs on steel decks would be a suitable substitute. As mentioned before, steel and macro synthetic fibers have been shown to satisfy the requirements for minimum temperature and shrinkage reinforcement through extensive testing and SDI has practiced due diligence when determining appropriate fiber dosage levels to not only provide superior structural capacity as the specified WWF but also satisfy the other requirement of this application.

The proposed changes to section 2209.2.1 and 2209.2.2 are not only unfounded but based on allegations made at the committee code hearings " fibers (Steel and Macro-synthetics) are not suitable to provide minimum temperature and shrinkage reinforcement, because they are not capable of holding cracks closed as effectively as WWF". I would like to see any solid research evidence in support of this statement.

They were made by an uninformed party who has not researched the advances that have been made in fiber reinforced concrete technology and our understanding of its behavior. One of the largest American Concrete Institute Committees is currently working on several documents to support the use of fiber reinforcement in structural applications to replace considerably more steel reinforcement than used in this application. Many lab and full scale tests support this work. I therefore strongly reject these allegations and confidently state that macro-synthetic and steel fibers, as specified in the existing document, are a superior alternative to the level of WWF specified and provide the added benefit of improved performance, economics, and considerably improvement placement safety.

In addition to the above mentioned benefits the uniform distribution of commercially available macro-synthetic and steel fibers ensures complete reinforcement whereas accurate placement in WWF is very difficult if not impossible as seen by anyone who has cored a WWF reinforced slab and found the reinforcement at the bottom of the slab. When one considers the effectiveness of conventional reinforcement depends strongly on the distance between the surface and the reinforcement level, specifying WWF over fiber reinforcement in this application is a greater risk due to the high probability/certainty of WWF reinforcement misplacement. The widespread use of fiber reinforcement in slabs on grade, precast products and shotcrete applications where strict quality control and inspections are required confirms that fibers in the concrete are easily and have been successfully inspected like other concrete constituents in commercial applications.

Public Comment 13:

Ronald F. Zollo, Ph.D., P.E., University of Miami, requests Approval as Submitted.

Commenter's Reason: **What Is It You Are Asked To Approve?**

Fiber-reinforced concrete (FRC) is a material of construction that was first patented in the early 1960's. The original patents have now expired but R and D of materials and applications has been aggressive and ongoing and has resulted in commercialization of engineered design applications based on measurable performance requirements. Depending upon the fiber type and mixture specifications FRC performance may be limited to crack control or can extended to provide reliable and verifiable post cracking strength and ductility properties that are useful in engineered design applications. One such application is in composite deck for temperature and shrinkage crack control.

Where Will Objections To Approval Come From?

Applications that are presently served by the welded wire fabric (WWF) industry be the source of objections from that industry. In stark contrast to current widespread R&D and commercial applications of FRC, by comparison WWF is a technology that resembles the traditional concept of reinforced concrete but relies mostly on its historical use, and not on published research, as justification for its use. There has been an ongoing competition in the marketplace between WWF and FRC and market share, not proven performance, undermines meaningful debate. What is or should be at issue, in the context of what code bodies must or should consider in an approval process for industrial applications, is proven performance.

How is QA, QC and Performance Be Established for FRC?

There are least three ASTM testing standards serve the needs of the construction industry regarding quality assurance, quality control and research testing of FRC. For example among these is ASTM C1399, <u>Standard Test Method for Average Residual Strength of Fiber-reinforced Concrete</u>, which is a practical test method specifically designed to demonstrate how and to what extent a particular FRC formulation will develop post cracking flexural tensile strength, termed average residual strength (ARS), such that this parameter can be applied in rational engineering design applications. Therefore, to the extent that FRC can meet the performance specifications that are equivalent to current applications that utilize WWF, such as by providing for crack control and crack bridging stress transfer in temperature and shrinkage (T&S) reinforcing, FRC must be allowed as an equal alternate or demonstrably superior design alternative.

How Does Rational Engineering Design Apply?

The fact is that FRC is a portland cement based concrete material that is applied extensively and in increasing annual volume in the construction of residential, commercial-industrial, military and homeland security applications varying from cast in place slab on ground, structural floors and walls, hardened structures such as missile silos, security vaults, blast resistant structures, new construction and repair of land based and underwater structures, hydraulic structures such as locks, dams, dykes, channels, and spillways, and pavements and liners in tunnels, pavements for aircraft runways, bridge decks, subterranean structures such as manholes, burial vaults, slope stabilization, and numerous other applications. Moreover, many of these applications require QA and QC testing according to established standards conducted by independent testing laboratories to help establish performance parameters, the most common one of which is ARS post cracking strength, to help assure the integrity of the finished product or structure. This, and not tradition, history or market share, should be the basis of acceptance for any material system for such as T&S reinforcing.

What About FRC in Composite Decks?

For composite deck construction there should be no issue regarding acceptance of FRC. In this application FRC is not intended to act as a replacement for primary structural reinforcement. All other factors equal, neither WWF nor FRC has any effect on composite deck performance until cracking occurs. Thereafter, the resulting strength and stiffness of composite deck slabs will depend on the location of the WWF within the slab depth and that steel cover requirements will apply. These restrictive requirements do not apply to fiber reinforcement. For FRC there are no cover limitations and, from an engineering point of view, the reliability of using WWF for crack control and to establish aggregate interlock for shear transfer and diaphragm action cannot be assured in comparison to the ubiquitous and three dimensional performance of FRC. On this basis there is no real engineering advantage to using WWF in comparison to FRC and the decision to choose one over the other system is simply reduced to economics and constructability issues.

Public Comment 14:

W. Samuel Easterling, PhD, PE, Virginia Tech, requests Approval as Modified by this Public Comment.

Modify proposal as follows:

2209.2 Steel decks. The design and construction of cold-formed steel decks shall be in accordance with this section.

2209.2.1 Composite slabs on steel decks. Composite slabs of concrete and steel deck shall be permitted to be designed and constructed in accordance with ASCE 3 or ANSI/SDI-C1.0 <u>as modified in Section 2209.2.1.1.</u>

<u>**2209.2.1.1 ANSI/SDI-C1.0 Section 2.4B6a.** Replace Section 2.4B6a of ANSI/SDI-C1.0 with the following:</u>
<u>The load capacity of a composite floor deck slab is not dependent on the use of additional reinforcement to control the effects of temperature and shrinkage of the concrete above the steel deck.</u>

(Portions of proposal not shown remain unchanged)

Commenter's Reason: As written, Section 2.4B6a of SDI-C1.0 permits the use of welded wire reinforcing, reinforcing bars, steel fibers or synthetic fibers to control temperature and shrinkage cracking perpendicular to the span of the deck. The control of temperature and shrinkage cracks represents a serviceability design limit. Any cracking that occurs due to temperature and shrinkage of the concrete does not affect the maximum strength of a composite slab, therefore it is reasonable to not let this provision influence consideration of the strength based code provisions. That said, I do believe that designers benefit from code provisions, where appropriate, that address serviceability issues.

I support the provision as modified.

Section 2.4B6a – Text as it appears in SD1-C1.0:

a. Temperature and shrinkage reinforcement, consisting of welded wire fabric or reinforcing bars, shall have a minimum area of 0.00075 times the area of the concrete above the deck (per foot or meter of width), but shall not be less that [sic] the area provided by 6 x 6 – W1.4 x W1.4 welded wire fabric.

Fibers shall be permitted as a suitable alternative to the welded wire fabric specified for temperature and shrinkage reinforcement. Cold-drawn steel fibers meeting the criteria of ASTM A820, at a minimum addition rate of 25 lb/cu yd (14.8 kg/cu meter), or macro synthetic fibers "Coarse fibers" (per ASTM Subcommittee C09.42), made from virgin polyolefin, shall have an equivalent diameter between 0.4 mm (0.016 in.) and 1.25 mm (0.05 in.), having a minimum aspect ratio (length/equivalent diameter) of 50, at a minimum addition rate of 4 lb./cu yd (2.4 kg/m^3) are suitable to be used as minimum temperature and shrinkage reinforcement.

Public Comment 15:

Roy H. Rieterman, Wire Reinforcement Institute, requests Approval as Modified by this Public Comment.

Modify proposal as follows:

2209.2 Composite slabs on steel decks. Composite slabs of concrete and steel deck shall be designed and constructed in accordance with ASCE 3 <u>and ACI 318.</u>

(Portions of proposal not shown remain unchanged)

Commenter's Reason: Design of composite slabs on steel decks shall be in accordance with ASCE 3 and ACI 318. ACI 318 is the governing code for reinforced and non-reinforced concrete design.

Public Comment 16:

Thomas Sputo, PhD, PE, SE, Steel Deck Institute, requests Approval as Modified by this Public Comment.

Replace proposal as follows:

2209.2.1 Composite slabs on steel decks. Composite slabs of concrete and steel deck shall be permitted to be designed and constructed in accordance with ASCE 3 or ANSI/SDI-C1.0<u>, as modified in Section 2209.2.1.1.</u>

<u>**2209.2.1.1 ANSI/SDI-C1.0 Section 2.4B6a.** Replace Section 2.4B6a of ANSI/SDI-C1.0 with the following:</u>

<u>a. The load capacity of a composite floor deck slab shall not be dependent on the use of additional reinforcement to control the effects of temperature and shrinkage of the concrete above the steel deck.</u>

<u>**Steel Deck Institute**</u>
<u>C1.0-06</u> <u>Standard for Composite Steel Floor Deck</u>

Commenter's Reason: As written, Section 2.4B6a of SDI-C1.0 permits the use of welded wire reinforcing, reinforcing bars, steel fibers or synthetic fibers to control temperature and shrinkage cracking perpendicular to the span of the deck. At the ICC Code Development Hearings in Palm Springs, this proved to be more controversial than anticipated. The reality is that there really is no structural necessity to control cracking of a composite floor deck slab; that is, it is not needed to prevent structural failure. In fact, the original decision by the SDI Standards Committee to add temperature and shrinkage reinforcement to a composite floor deck slab was in response to a serviceability issue. However, upon reflection, it was decided that this should ultimately be at the discretion of the designer of the structure. It should not be mandated by the code, and its underlying standards, based on a belief that temperature and shrinkage reinforcement is necessary to maintain the integrity of a composite floor deck slab, as it could be in the case of a reinforced concrete slab made using removable forms. By deleting the requirements for temperature and shrinkage reinforcement, the IBC Structural Committee's reason for disapproval has been addressed.

Section 2.4B6a – Text as it appears in SD1-C1.0:

a. Temperature and shrinkage reinforcement, consisting of welded wire fabric or reinforcing bars, shall have a minimum area of 0.00075 times the area of the concrete above the deck (per foot or meter of width), but shall not be less that [sic] the area provided by 6 x 6 – W1.4 x W1.4 welded wire fabric.

Fibers shall be permitted as a suitable alternative to the welded wire fabric specified for temperature and shrinkage reinforcement. Cold-drawn steel fibers meeting the criteria of ASTM A820, at a minimum addition rate of 25 lb/cu yd (14.8 kg/cu meter), or macro synthetic fibers "Coarse fibers" (per ASTM Subcommittee C09.42), made from virgin polyolefin, shall have an equivalent diameter between 0.4 mm (0.016 in.) and 1.25 mm (0.05 in.), having a minimum aspect ratio (length/equivalent diameter) of 50, at a minimum addition rate of 4 lb./cu yd (2.4 kg/m^3) are suitable to be used as minimum temperature and shrinkage reinforcement.

Public Comment 17:

Thomas Sputo, PhD., PE, SE, Steel Deck Institute, requests Approval as Modified by this Public Comment.

Replace proposal as follows:

2209.2.2 Non-composite steel floor decks. Non-composite steel floor decks shall be permitted to be designed and constructed in accordance with ANSI/SDI-NC1.0, as modified in Section 2209.2.1.1.

2209.2.2.1 ANSI/SDI-NC1.0 Section 2.4B1. Replace Section 2.4B1 of ANSI/SDI-NC1.0 with the following:

1. General: The design of the concrete slabs shall be done in accordance with the ACI *Building Code Requirements for Reinforced Concrete.* The minimum concrete thickness above the top of the deck shall be 1 ½ inches (38 mm).

Steel Deck Institute
NC1.0-06 Standard for Non-Composite Steel Floor Deck

Commenter's Reason: This proposed modification to SDI NC1.0 is intended to remove any doubt that non-composite concrete slabs must be designed in accordance with the applicable reference standard -- ACI 318. By specifically requiring compliance with ACI 318 (which does not permit fibers or fibrous admixtures), and removing any mention of fibers or fibrous admixtures in SDI NC1.0, Section 2.4B1, the IBC Structural Committee's reason for Disapproval has been addressed.

Section 2.4B1 – Text as it appears in SD1-NC1.0:

1. **General:** The design of the concrete slabs shall be done in accordance with the ACI *Building Code Requirements for Reinforced Concrete*. The minimum concrete thickness above the top of the deck shall be 1-1/2 inches (38 mm). Randomly distributed fibers or fibrous admixtures shall not be substituted for welded wire fabric tensile reinforcement.

Public Comment 18:

Thomas Sputo, PhD., PE, SE, Steel Deck Institute, requests Approval as Modified by this Public Comment.

Replace proposal as follows:

2209.2.3 Steel roof deck. Steel roof decks shall be permitted to be designed and constructed in accordance with ANSI/SDI-RD1.0.

Steel Deck Institute
RD1.0-06 Standard for Steel Roof Deck

Commenter's Reason: Based on testimony from the floor, the IRC Building/Energy Committee disapproved the original proposal, citing confusion regarding the use of fibers as a substitute for steel reinforcement in both the composite steel floor deck and non-composite floor deck standards. However, the testimony from the floor regarding the substitution of fibers for steel reinforcement did not apply to the third SDI standard proposed for inclusion in Section 2209.2 – SDI-RD1.0, Standard for Steel Roof Deck. This comment simply recommends this document for adoption in the IBC.

Public Comment 19:

Todd R. Hawkinson, P.E, Wire Reinforcement Institute, requests Disapproval.

Commenter's Reason: The proposed code change was disapproved at the ICC Hearings and as noted from discussion at the ICC Hearings needs further consideration.

Final Action: AS AM AMPC____ D

S209-07/08
2210.3

Proposed Change as Submitted:

Proponent: Edwin Huston, National Council of Structural Engineers Association (NCSEA), representing NCSEA Code Advisory Committee – General Engineering Subcommittee

1. Revise as follows:

2210.3 Trusses. ~~The design, quality assurance, installation and testing of cold-formed steel trusses shall be in accordance with AISI-Truss, subject to the limitations therein.~~ Cold-formed steel trusses shall be designed in accordance with the provisions of this code and accepted engineering practice. Members are permitted to be joined by screws, pins, rivets, bolts, clinching, welding, or other approved connecting devices.

2. Add new text as follows:

2210.3.1 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. The 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 (including snow loads);
   - 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, snow, seismic, etc.); and
   - 5.7. Other lateral loads, including drag strut loads.
6. Maximum reaction force and direction, including maximum uplift reaction forces where applicable;
7. All truss joint connections, information and details;
8. Member sizes, properties and details;
9. Truss- to- truss connections and truss field assembly requirements.
10. Calculated span to deflection ratio and maximum vertical and horizontal deflection for live and total load as applicable;
11. Maximum axial tension and compression in the truss members; and
12. Required permanent individual truss member restraint locations and the method and details of restraint/bracing to be used in accordance with Section 2210.3.2.

2210.3.2 Permanent individual truss member restraint/bracing. Where permanent restraint/bracing of truss members is specified on the truss design drawings, it shall be accomplished by one of the following methods:

1. Permanent individual truss member restraint/bracing shall be installed using standard industry lateral restraint/bracing details in accordance with generally accepted engineering practice. Locations for lateral restraint shall be identified on the truss design drawing.
2. The trusses shall be designed so that the buckling of any individual truss member is resisted internally by the individual truss through suitable means (i.e., buckling reinforcement by T-reinforcement or L-reinforcement, proprietary reinforcement, etc.). The buckling reinforcement of individual members of the trusses shall be installed as shown on the truss design drawing or on supplemental truss member buckling reinforcement details provided by the truss designer.
3. A project specific permanent individual truss member restraint/bracing design shall be permitted to be specified by any registered design professional.

2210.3.3 Trusses spanning 60 feet or greater. The owner shall contract with a registered design professional for the design of the temporary installation restraint/bracing and the permanent individual truss member restraint/bracing for trusses with clear spans 60 feet (18 288mm) or greater.

2210.3.4 Truss designer. The individual or organization responsible for the design of the trusses.

2210.3.5 Truss design drawings. Where required by the registered design professional, the building official, or the statutes of the jurisdiction in which the project is to be constructed, each individual truss design drawing shall bear the seal and signature of the truss designer.

Exceptions:

1. Where a cover sheet and truss index sheet are combined into a single sheet and attached to the set of truss design drawings, the single cover/truss index sheet is the only document required to be signed and sealed by the truss designer.
2. When a cover sheet and a truss index sheet are separately provided and attached to the set of truss design drawings, the cover sheet and the truss index sheet are the only documents required to be signed and sealed by the truss designer.

2210.3.6 Truss placement diagram. The truss manufacturer shall provide a truss placement diagram that identifies the proposed location for each individually designated truss and references the corresponding truss design drawing. The truss placement diagram shall be provided as part of the truss submittal package, and with the shipment of trusses delivered to the job site. Truss placement diagrams that serve only as a guide for installation and do not deviate from the permit submittal drawings shall not be required to bear the seal or signature of the truss designer.

2210.3.7 Truss submittal package. The truss submittal package shall consist of each individual truss design drawing; the truss placement diagram; the permanent individual truss member restraint/bracing method and details; any other structural details germane to the trusses as applicable; and the cover/truss index sheet.

2210.3.8 Anchorage. The design for the transfer of loads and anchorage of each truss to the supporting structure is the responsibility of the registered design professional.

2210.3.9 Alterations to trusses. Truss members and components shall not be cut, notched, drilled, spliced or otherwise altered in any way without written concurrence and approval of a registered design professional. Alterations resulting in the addition of loads to any member (i.e. HVAC equipment, piping, additional roofing or insulation, etc.) shall not be permitted without verification that the truss is capable of supporting such additional loading.

2210.3.10 AISI specification. In addition to Sections 2210.3 through 2210.3.9, the design, manufacture, installation, testing and quality assurance of cold formed steel trusses shall be in accordance with AISI S214. Job-site inspections shall be in compliance with Section 109 as applicable.

2210.3.11 Truss quality assurance. Trusses not part of a manufacturing process in accordance with Section 2210.3.10 or in accordance with a standard listed in Chapter 35, which provides requirements for quality control done under the supervision of a third party quality control agency, shall be manufactured in compliance with Sections 1704.2 and 1704.3 as applicable.

Reason: This language is proposed to be added in order for cold formed steel trusses to have compatible criteria as the requirements for wood trusses specified in Section 2303.4. The current Section does not provide the necessary criteria and delineation of responsibilities.

Cost Impact: The code change proposal will not increase the cost of construction.

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**

Individual Consideration Agenda

This item is on the agenda for individual consideration because public comments were submitted.

Public Comment 1:

Edwin Huston, National Council of Structural Engineers Association (NCSEA), representing NCSEA Code Advisory Committee, General Engineering Subcommittee, requests Approval as Modified by this Public Comment.

Modify proposal as follows:

2210.3 Trusses. ~~Cold-formed steel trusses shall be designed in accordance with the provisions of this code and accepted engineering practice. Members are permitted to be joined by screws, pins, rivets, bolts, clinching, welding, or other approved connecting devices~~.

2210.3.1 Design. Cold-Formed Steel Trusses shall be designed in accordance with AISI S214, Sections 2210.3.1 through 2210.3.5 and accepted engineering practice.

~~**2210.3.1 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. The 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 (including snow loads);~~
   - ~~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, snow, seismic, etc.); and~~
   - ~~5.7. Other lateral loads, including drag strut loads.~~
6. ~~Maximum reaction force and direction, including maximum uplift reaction forces where applicable;~~
7. ~~All truss joint connections, information and details;~~
8. ~~Member sizes, properties and details;~~
9. ~~Truss- to- truss connections and truss field assembly requirements.~~
10. ~~Calculated span to deflection ratio and maximum vertical and horizontal deflection for live and total load as applicable;~~
11. ~~Maximum axial tension and compression in the truss members; and~~
12. ~~Required permanent individual truss member restraint locations and the method and details of restraint/bracing to be used in accordance with Section 2210.3.2.~~

2210.3.2 Truss Design Drawings. The Truss Design Drawings shall conform to the requirements of Section B2.3 of AISI S214 and shall be provided with the shipment of trusses delivered to the job site. The Truss Design Drawings shall include the details of permanent individual truss member restraint/bracing in accordance with section B6(a) or B6(c) of AISI S214 if these methods are utilized to provide restraint/bracing.

~~**2210.3.2 Permanent individual truss member restraint/bracing**. Where permanent restraint/bracing of truss members is specified on the truss design drawings, it shall be accomplished by one of the following methods:~~

1. ~~Permanent individual truss member restraint/bracing shall be installed using standard industry lateral restraint/bracing details in accordance with generally accepted engineering practice. Locations for lateral restraint shall be identified on the truss design drawing.~~
2. ~~The trusses shall be designed so that the buckling of any individual truss member is resisted internally by the individual truss through suitable means (i.e., buckling reinforcement by T-reinforcement or L-reinforcement, proprietary reinforcement, etc.). The buckling reinforcement of individual members of the trusses shall be installed as shown on the truss design drawing or on supplemental truss member buckling reinforcement details provided by the truss designer.~~
3. ~~A project specific permanent individual truss member restraint/bracing design shall be permitted to be specified by any registered design professional.~~

2210.3.3 Deferred Submittals. AISI Section B4.2 shall be deleted.

~~**2210.3.3**~~ **2210.3.4 Trusses spanning 60 feet or greater**. The owner shall contract with a registered design professional for the design of the temporary installation restraint/bracing and the permanent individual truss member restraint/bracing for trusses with clear spans 60 feet (18 288mm) or greater. Special inspection of trusses over 60 feet in length shall conform to Section 1704.

~~**2210.3.4 Truss designer**. The individual or organization responsible for the design of the trusses.~~

~~**2210.3.5 Truss design drawings**. Where required by the registered design professional, the building official, or the statutes of the jurisdiction in which the project is to be constructed, each individual truss design drawing shall bear the seal and signature of the truss designer.~~

~~**Exceptions:**~~

1. ~~Where a cover sheet and truss index sheet are combined into a single sheet and attached to the set of truss design drawings, the single cover/truss index sheet is the only document required to be signed and sealed by the truss designer.~~
2. ~~When a cover sheet and a truss index sheet are separately provided and attached to the set of truss design drawings, the cover sheet and the truss index sheet are the only documents required to be signed and sealed by the truss designer.~~

~~**2210.3.6 Truss placement diagram**. The truss manufacturer shall provide a truss placement diagram that identifies the proposed location for each individually designated truss and references the corresponding truss design drawing. The truss placement diagram shall be provided as part of the truss submittal package, and with the shipment of trusses delivered to the job site. Truss placement diagrams that serve only as a guide for installation and do not deviate from the permit submittal drawings shall not be required to bear the seal or signature of the truss designer.~~

~~**2210.3.7 Truss submittal package**. The truss submittal package shall consist of each individual truss design drawing; the truss placement diagram; the permanent individual truss member restraint/bracing method and details; any other structural details germane to the trusses as applicable; and the cover/truss index sheet.~~

~~**2210.3.8 Anchorage**. The design for the transfer of loads and anchorage of each truss to the supporting structure is the responsibility of the registered design professional.~~

~~**2210.3.9 Alterations to trusses.** Truss members and components shall not be cut, notched, drilled, spliced or otherwise altered in any way without written concurrence and approval of a registered design professional. Alterations resulting in the addition of loads to any member (i.e. HVAC equipment, piping, additional roofing or insulation, etc.) shall not be permitted without verification that the truss is capable of supporting such additional loading.~~

~~**2210.3.10 AISI specification**. In addition to Sections 2210.3 through 2210.3.9, the design, manufacture, installation, testing and quality assurance of cold formed steel trusses shall be in accordance with AISI S214. Job-site inspections shall be in compliance with Section 109 as applicable.~~

~~2210.3.11~~ **<u>2210.3.</u>5 Truss quality assurance**. Trusses not part of a manufacturing process ~~in accordance with Section 2210.3.10 or in accordance with a standard listed in Chapter 35, which~~ <u>that</u> provides requirements for quality control done under the supervision of a third party quality control agency, shall be manufactured in compliance with Sections 1704.2 and 1704.3 as applicable.

Commenter's Reason: Over the past year, NCSEA has been working diligently with both AISI and WTCA to address their concerns surrounding the truss design requirements in S209 as compared to those found in AISI S214-07, North American Standard for Cold-Formed Steel Framing – Truss Design. Unfortunately, the language that the IBC Structural Code Development Committee accepted as submitted in Proposal S209-07/08 does not recognize the latest work by the three organizations. The modifications presented in this public comment correct IBC Section 2210.3 to reflect the established consensus on this topic, where such consensus could be reached within this short timeframe.

In response to Proposal S209-07/08, AISI agreed to process a supplement to AISI S214-07 addressing both NCSEA's and WTCA's concerns. Supplement 2 to AISI S214-07 was issued June 9, 2008 and is available for download from the AISI website: www.aisi.org. The following topics, which were originally presented in Proposal S209-07/08, were picked up with some minor modification in AISI S214, Supplement 2 and, consequently, are recommended for deletion from IBC Section 2210.3:

- Truss design drawings (2210.3.1) – Other than pointing the user to S214, stating that the details need to be shipped to the jobsite, and pointing out that restraint/bracing details need to be provided, this topic is addressed in AISI S214, Supplement 2, Section B2.
- Permanent individual truss member restraint/bracing (2210.3.2) – The topic is addressed in AISI S214, Supplement 2, Section B6.
- Truss designer (2210.3.4) – The topic is addressed in AISI S214, Supplement 2, Section B2.
- Truss design drawings (2210.3.5) – The topic is addressed in AISI S214, Supplement 2, Section B2
- Truss placement diagram (2210.3.6) – The topic is addressed in AISI S214, Supplement 2, Section B3.
- Truss submittal package (2210.3.7) – The topic is addressed in AISI S214, Supplement 2, Section B3.
- Anchorage (2210.3.8) – The topic is addressed in AISI S214, Supplement 2, Section B4.
- Alterations to trusses (2210.3.9) – The topic is addressed in AISI S214, Supplement 2, Section B5.
- AISI Specification (2210.3.10) – The reference to AISI S214 has been relocated to IBC Section 2210.3. It is considered that a pointer to IBC Section 109 is unnecessary

This Public Comment deletes Section B4.2, because when the Contract Drawings are submitted for permit, the Registered Design Professional may not know what items will be designated as deferred submittals. In addition, it is inappropriate for AISI Section B4.2 for cold-formed trusses to list requirements for deferred submittals on other items or materials.
During the development of Supplement 2, two topics were determined to be outside the jurisdiction of the AISI Committee on Framing Standards, the committee responsible for AISI S214. Consequently, they have been retained for inclusion in IBC Section 2210.3. They are:

- Design of Trusses spanning 60 feet or greater (2210.3.3) – AISI S214 covers the topic of permanent not temporary installation restraint/bracing. However, S209 put this under design. In this Public Comment it has been retained as 2210.3.3.
- Truss quality assurance (2210.3.11) – AISI S214, Chapter E provides requirements for Quality Criteria for Steel Trusses that are part of a manufacturing process. This section addresses trusses that fall outside of those processes. In this Public Comment it has been moved to 2210.3.4.

Please note, the modifications presented here coordinate with the changes accepted by the IBC Structural Code Development Committee in Proposal S208-07/08. Also, to fully integrate AISI S214-07, Supplement 2 into the IBC, a public comment has been submitted on Proposal S238-07/08 adopting the supplement in IBC Chapter 35.

Public Comment 2:

Bonnie Manley, American Iron and Steel Institute, requests Approval as Modified by this Public Comment.

Modify proposal as follows:

2210.3 Trusses <u>Truss design</u>. Cold-formed steel trusses shall be designed in accordance with the provisions of this code and accepted engineering practice. Members are permitted to be joined by screws, pins, rivets, bolts, clinching, welding, or other approved connecting devices. <u>Trusses shall be designed in accordance with AISI S214 and Sections 2210.3.1 through 2210.3.3.</u>

~~**2210.3.1 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. The 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 (including snow loads);~~
    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, snow, seismic, etc.); and~~
    5.7. ~~Other lateral loads, including drag strut loads.~~
6. ~~Maximum reaction force and direction, including maximum uplift reaction forces where applicable;~~
7. ~~All truss joint connections, information and details;~~
8. ~~Member sizes, properties and details;~~
9. ~~Truss- to- truss connections and truss field assembly requirements.~~
10. Calculated span to deflection ratio and maximum vertical and horizontal deflection for live and total load as applicable;
11. ~~Maximum axial tension and compression in the truss members; and~~
12. ~~Required permanent individual truss member restraint locations and the method and details of restraint/bracing to be used in accordance with Section 2210.3.2.~~

~~**2210.3.2 Permanent individual truss member restraint/bracing**. Where permanent restraint/bracing of truss members is specified on the truss design drawings, it shall be accomplished by one of the following methods:~~

1. ~~Permanent individual truss member restraint/bracing shall be installed using standard industry lateral restraint/bracing details in accordance with generally accepted engineering practice. Locations for lateral restraint shall be identified on the truss design drawing.~~
2. ~~The trusses shall be designed so that the buckling of any individual truss member is resisted internally by the individual truss through suitable means (i.e., buckling reinforcement by T-reinforcement or L-reinforcement, proprietary reinforcement, etc.). The buckling reinforcement of individual members of the trusses shall be installed as shown on the truss design drawing or on supplemental truss member buckling reinforcement details provided by the truss designer.~~
3. ~~A project specific permanent individual truss member restraint/bracing design shall be permitted to be specified by any registered design professional.~~

~~2210.3.3~~ <u>2210.3.1 Design of</u> ~~T~~ <u>t</u>russes spanning 60 feet or greater. The owner shall contract with a registered design professional for the design of the temporary installation restraint/bracing and the permanent individual truss member restraint/bracing for trusses with clear spans 60 feet (18 288mm) or greater. <u>Special inspection of trusses over 60 feet in length shall conform to Section 1704.</u>

~~**2210.3.4 Truss designer.** The individual or organization responsible for the design of the trusses.~~

~~**2210.3.5 Truss design drawings**. Where required by the registered design professional, the building official, or the statutes of the jurisdiction in which the project is to be constructed, each individual truss design drawing shall bear the seal and signature of the truss designer.~~

~~Exceptions:~~

1. ~~Where a cover sheet and truss index sheet are combined into a single sheet and attached to the set of truss design drawings, the single cover/truss index sheet is the only document required to be signed and sealed by the truss designer.~~
2. ~~When a cover sheet and a truss index sheet are separately provided and attached to the set of truss design drawings, the cover sheet and the truss index sheet are the only documents required to be signed and sealed by the truss designer.~~

~~**2210.3.6 Truss placement diagram**. The truss manufacturer shall provide a truss placement diagram that identifies the proposed location for each individually designated truss and references the corresponding truss design drawing. The truss placement diagram shall be provided as part of the truss submittal package, and with the shipment of trusses delivered to the job site. Truss placement diagrams that serve only as a guide for installation and do not deviate from the permit submittal drawings shall not be required to bear the seal or signature of the truss designer.~~

~~**2210.3.7 Truss submittal package**. The truss submittal package shall consist of each individual truss design drawing; the truss placement diagram; the permanent individual truss member restraint/bracing method and details; any other structural details germane to the trusses as applicable; and the cover/truss index sheet.~~

~~**2210.3.8 Anchorage**. The design for the transfer of loads and anchorage of each truss to the supporting structure is the responsibility of the registered design professional.~~

~~**2210.3.9 Alterations to trusses**. Truss members and components shall not be cut, notched, drilled, spliced or otherwise altered in any way without written concurrence and approval of a registered design professional. Alterations resulting in the addition of loads to any member (i.e. HVAC equipment, piping, additional roofing or insulation, etc.) shall not be permitted without verification that the truss is capable of supporting such additional loading.~~

~~2210.3.10~~ <u>2210.3.2</u> ~~AISI specification.~~<u>Job-site inspections</u>. ~~In addition to Sections 2210.3 through 2210.3.9, the design, manufacture, installation, testing and quality assurance of cold formed steel trusses shall be in accordance with AISI S214.~~ Job-site inspections shall be in compliance with Section 109 as applicable.

~~2210.3.11~~ **<u>2210.3.3</u>Truss quality assurance**. Trusses not part of a manufacturing process ~~in accordance with Section 2210.3.10 or in accordance with a standard listed in Chapter 35, which~~ <u>that</u> provides requirements for quality control done under the supervision of a third party quality control agency, shall be manufactured in compliance with Sections 1704.2 and 1704.3 as applicable.

Commenter's Reason: Over the past year, AISI has been working with both WTCA and NCSEA to address their concerns surrounding the truss design responsibility requirements found in AISI S214-07, North American Standard for Cold-Formed Steel Framing – Truss Design. Unfortunately, the language that the IBC Structural Code Development Committee accepted as submitted in Proposal S209-07/08 does not recognize the latest work by the three organizations. The modifications presented in this public comment correct IBC Section 2210.3 to reflect the established consensus on this topic.

In response to Proposal S209-07/08, AISI agreed to process a supplement to AISI S214-07 addressing both NCSEA's and WTCA's concerns. Supplement 2 to AISI S214-07 was issued in June 2008 and is available for download from the AISI website: www.steel.org. (Click on "Construction" link and then click on "Codes and Standards" link.) Supplement 2 completely replaces Supplement 1 to AISI S214-07.

The following topics, which were originally presented in Proposal S209-07/08, were picked up with some minor modification in AISI S214, Supplement 2 and, consequently, are recommended for deletion from IBC Section 2210.3:

- Truss design drawings (2210.3.1) – The topic is addressed in AISI S214, Supplement 2, Section B2.3.
- Permanent individual truss member restraint/bracing (2210.3.2) – The topic is addressed in AISI S214, Supplement 2, Section B6.
- Truss designer (2210.3.4) – The topic is addressed in AISI S214, Supplement 2, Section B2.1.
- Truss design drawings (2210.3.5) – The topic is addressed in AISI S214, Supplement 2, Section B2.4.
- Truss placement diagram (2210.3.6) – The topic is addressed in AISI S214, Supplement 2, Section B3.3.
- Truss submittal package (2210.3.7) – The topic is addressed in AISI S214, Supplement 2, Section B3.4.
- Anchorage (2210.3.8) – The topic is addressed in AISI S214, Supplement 2, Section B4.4.
- Alterations to trusses (2210.3.9) – The topic is addressed in AISI S214, Supplement 2, Section B5.6.
- AISI Specification (2210.3.10) – The reference to AISI S214 has been relocated to IBC Section 2210.3.

During the development of Supplement 2, three topics were determined to be outside the jurisdiction of the AISI Committee on Framing Standards, the committee responsible for AISI S214. Consequently, they have been retained for inclusion in IBC Section 2210.3. They include the following:

- Design of Trusses spanning 60 feet or greater (2210.3.3) – AISI S214 covers the topic of permanent, not temporary, installation restraint/bracing.
- Job-site inspections (2210.3.10) – This is a traditional building code topic; and,
- Truss quality assurance (2210.3.11) – AISI S214, Chapter E provides requirements for Quality Criteria for Steel Trusses that are part of a manufacturing process. This section addresses trusses that fall outside of those processes.

Of these three retained topics, only the section on "Design of trusses spanning 60 feet or greater" has been modified slightly to include a reference to Section 1704, which coordinates with changes accepted in Proposal S115-07/08.

Please note, the modifications presented here coordinate with the changes accepted by the IBC Structural Code Development Committee in Proposal S208-07/08. Also, to fully integrate AISI S214-07, Supplement 2 into the IBC, a public comment has been submitted on Proposal S238-07/08 adopting the supplement in IBC Chapter 35.

Public Comment 3:

Larry Wainright, WTCA, representing The Structural Building Components Industry, requests Approval as Modified by this Public Comment.

Modify proposal as follows:

2210.3 Trusses. ~~Cold-formed steel trusses shall be designed in accordance with the provisions of this code and accepted engineering practice. Members are permitted to be joined by screws, pins, rivets, bolts, clinching, welding, or other approved connecting devices.~~

<u>**2210.3.1 Design.** Cold-formed steel trusses shall be designed in accordance with AISI S214, Section 2210.3.1 through Section 2210.3.5.</u>

~~**2210.3.1 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. The 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 (including snow loads);~~
    - ~~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, snow, seismic, etc.); and~~
    - ~~5.7. Other lateral loads, including drag strut loads.~~
6. ~~Maximum reaction force and direction, including maximum uplift reaction forces where applicable;~~
7. ~~All truss joint connections, information and details;~~
8. ~~Member sizes, properties and details;~~
9. ~~Truss- to- truss connections and truss field assembly requirements.~~
10. ~~Calculated span to deflection ratio and maximum vertical and horizontal deflection for live and total load as applicable;~~
11. ~~Maximum axial tension and compression in the truss members; and~~
12. ~~Required permanent individual truss member restraint locations and the method and details of restraint/bracing to be used in accordance with Section 2210.3.2.~~

~~**2210.3.1**~~ **2210.3.2 Truss Design Drawings**. The Truss Design Drawings shall conform to the requirements of Section B2.3 of AISI S214 and shall be provided with the shipment of trusses delivered to the job site. The Truss Design Drawings shall include the locations of required individual truss member restraint. Where permanent individual truss member restraint/bracing is required it shall conform to the requirements of Section B6 of AISI S214.

~~2210.3.2 Permanent individual truss member restraint/bracing. Where permanent restraint/bracing of truss members is specified on the truss design drawings, it shall be accomplished by one of the following methods:~~

1. ~~Permanent individual truss member restraint/bracing shall be installed using standard industry lateral restraint/bracing details in accordance with generally accepted engineering practice. Locations for lateral restraint shall be identified on the truss design drawing.~~
2. ~~The trusses shall be designed so that the buckling of any individual truss member is resisted internally by the individual truss through suitable means (i.e., buckling reinforcement by T-reinforcement or L-reinforcement, proprietary reinforcement, etc.). The buckling reinforcement of individual members of the trusses shall be installed as shown on the truss design drawing or on supplemental truss member buckling reinforcement details provided by the truss designer.~~
3. ~~A project specific permanent individual truss member restraint/bracing design shall be permitted to be specified by any registered design professional.~~

2210.3.3 Deferred submittals. Section B4.2 of AISI S214 shall be deleted.

~~**2210.3.3**~~ **2210.3.4 Trusses spanning 60 feet (18 m) or greater**. Special inspection of trusses shall conform to Section 1704.3.4. ~~The owner shall contract with a registered design professional for the design of the temporary installation restraint/bracing and the permanent individual truss member restraint/bracing for trusses with clear spans 60 feet (18 288mm) or greater~~.

~~**2210.3.4 Truss designer**. The individual or organization responsible for the design of the trusses.~~

~~**2210.3.5 Truss design drawings**. Where required by the registered design professional, the building official, or the statutes of the jurisdiction in which the project is to be constructed, each individual truss design drawing shall bear the seal and signature of the truss designer.~~

~~Exceptions:~~

1. ~~Where a cover sheet and truss index sheet are combined into a single sheet and attached to the set of truss design drawings, the single cover/truss index sheet is the only document required to be signed and sealed by the truss designer.~~
2. ~~When a cover sheet and a truss index sheet are separately provided and attached to the set of truss design drawings, the cover sheet and the truss index sheet are the only documents required to be signed and sealed by the truss designer.~~

~~**2210.3.6 Truss placement diagram**. The truss manufacturer shall provide a truss placement diagram that identifies the proposed location for each individually designated truss and references the corresponding truss design drawing. The truss placement diagram shall be provided as part of the truss submittal package, and with the shipment of trusses delivered to the job site. Truss placement diagrams that serve only as a guide for installation and do not deviate from the permit submittal drawings shall not be required to bear the seal or signature of the truss designer.~~

~~**2210.3.7 Truss submittal package.** The truss submittal package shall consist of each individual truss design drawing; the truss placement diagram; the permanent individual truss member restraint/bracing method and details; any other structural details germane to the trusses as applicable; and the cover/truss index sheet.~~

~~**2210.3.8 Anchorage**. The design for the transfer of loads and anchorage of each truss to the supporting structure is the responsibility of the registered design professional.~~

~~**2210.3.9 Alterations to trusses**. Truss members and components shall not be cut, notched, drilled, spliced or otherwise altered in any way without written concurrence and approval of a registered design professional. Alterations resulting in the addition of loads to any member (i.e. HVAC equipment, piping, additional roofing or insulation, etc.) shall not be permitted without verification that the truss is capable of supporting such additional loading.~~

~~**2210.3.10 AISI specification**. In addition to Sections 2210.3 through 2210.3.9, the design, manufacture, installation, testing and quality assurance of cold formed steel trusses shall be in accordance with AISI S214. Job site inspections shall be in compliance with Section 109 as applicable.~~

~~2210.3.11~~ **2210.3.5 Truss quality assurance**. Trusses not part of a manufacturing process ~~in accordance with Section 2210.3.10 or in accordance with a standard listed in Chapter 35, which~~ that provides requirements for quality control done under the supervision of a third party quality control agency, shall be manufactured in compliance with Sections 1704.2 and 1704.3 as applicable.

Commenter's Reason: Over the past year, WTCA has been working diligently with both AISI and NCSEA to address their concerns surrounding the truss design requirements in S209 as compared to those found in AISI S214-07, North American Standard for Cold-Formed Steel Framing – Truss Design. While we agree technically on the provisions here, there were a few sections that needed to be editorially corrected so that the language is consistent within AISI S214 and these provisions. This change is for editorial purposes to clean up the inconsistent language.

Final Action: AS AM AMPC____ D