IBC - Structural

2016 GROUP B PUBLIC COMMENT AGENDA

OCTOBER 19 - OCTOBER 25, 2016
KANSAS CITY CONVENTION CENTER
KANSAS CITY, MO
Committee Action: Approved as Modified

Proposed Change as Submitted

Proponent: Dennis Richardson, American Wood Council, representing American Wood Council (drichardson@awc.org)

2015 International Building Code

Revise as follows:

1401.1 Scope. The provisions of this chapter shall establish the minimum requirements for exterior walls; exterior wall coverings; exterior wall openings; exterior windows and doors; and architectural trim, balconies and similar projections; and bay and oriel windows.

1501.1 Scope. The provisions of this chapter shall govern the design, materials, construction and quality of roof assemblies, and rooftop structures, and balconies where the structural framing is protected by an impervious moisture barrier.

Reason: Provisions regarding ventilation for balconies that are protected by an impervious barrier yet are located outside of the building envelope are being added to Chapter 15 (new Section 1503.7) under a separate proposal. Since a balcony outside of the building envelope that has weather protection and supports loads most closely resembles a roof (see definition of roof assembly in IBC Section 202), it is felt chapter 15 is the most appropriate place for this provision. This code change revises the scoping statement of Chapter 15 to reflect this and also corrects the scoping statement in Chapter 14 Section 1401 that was not modified when Group A code change FS15-15 removed Balconies, similar projections and Bay and oriel windows from Chapter 14.

Cost Impact: Will not increase the cost of construction

This code change merely clarifies the scoping of chapters and references needing correction from a previous code change and does not change any provision of the code affecting cost.

Public Hearing Results

Modification: Approved as Modified

1501.1 Scope. The provisions of this chapter shall govern the design, materials, construction and quality of roof assemblies, and rooftop structures, and balconies where the structural framing is protected by an impervious moisture barrier.

Committee Reason: Since all balcony provisions were previously moved from Chapter 14 to Chapter 7, the revision to the scope of Chapter 14 is a good catch. The modification does away with the proposed change to the scope of Chapter 15, retaining only the current wording.

Assembly Action: None

Individual Consideration Agenda

Proponent: Scott Campbell, representing Portland Cement Association (scampbell@cement.org) requests Disapprove.

Commenter's Reason: Both the original proposal and the modified proposal are incorrect. There are no balcony provisions in Chapter 15, and only fire protection provisions for balconies are present in Chapter 7. Chapter 14 has multiple provisions for balconies, and as such balconies should remain in the scope of the chapter. Also, all balconies are addressed in Chapter 14 but this change would limit code provision for balconies where structural framing is protected by an impervious moisture barrier.
Committee Action: Disapproved
Assembly Action: None

2015 International Building Code
Add new text as follows:

**1511.7** Attic and rafter ventilation. For roof replacement over attics that require ventilation, intake and exhaust vents shall be provided in accordance with Section 1203 and the vent product manufacturer’s approved installation instructions.

Delete without substitution:

**1503.5** Attic and rafter ventilation. Intake and exhaust vents shall be provided in accordance with Section 1203.2 and the vent product manufacturer’s installation instructions.

2015 International Existing Building Code
Add new text as follows:

**706.7** Attic and rafter ventilation. For roof replacement over attics that require ventilation, intake and exhaust vents shall be provided in accordance with IBC Section 1203 and the vent product manufacturer’s approved installation instructions.

Reason: IRC and IEBC: This proposal clarifies the intent of the code and removes ambiguity. Ventilation of an attic space should be provided for roof replacement projects in addition to new construction. IBC: Section 1503.5 can be interpreted to apply to roofing projects types—new construction, replacement and re-cover, and does not differentiate between vented and unvented attics. Adding the proposed new language to the Reroofing section of the code and making it specific to roof replacement over vented attics clarifies the intent of the code.

Cost Impact: Will not increase the cost of construction
The proposal adds no new requirements.

Public Hearing Results

Part I

Committee Action: Disapproved
Committee Reason: Disapproval is consistent with the action taken on Part II of this code change. The terminology such as "ventilation openings" versus "attic and rafter ventilation" need to be made more consistent to avoid confusion.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent : James Kirby, Kellen, representing Asphalt Roofing Manufacturers Association (jameskirby47@icloud.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

**1511.7** Attic and rafter ventilation. For roof replacement over attics that require ventilation, intake and exhaust vents shall be provided in accordance with Section 1203 and the vent product manufacturer’s approved installation instructions.

**1503.5** Attic and rafter ventilation. Intake and exhaust vents shall be provided in accordance with Section 1203.2 and the vent product manufacturer’s installation instructions.

2015 International Existing Building Code
706.7 Attic and rafter ventilation. **Roof Replacement**

For roof replacement over attics that require ventilation, intake and exhaust vents enclosed rafter assemblies shall be provided in accordance with IBC meet the requirements of Section 1203 and R806 of the vent product manufacturer's approved installation instructions. *International Residential Code or Section 1206 of the International Building Code.*

**Commenter's Reason:** The Structural and IRC Building committees disapproved the original proposal because it wasn't clear enough about roof re-covers and existing roof vents. The Public Comment As-Modified clears up any confusion by moving the new code language for the IBC from the Reroofing section to the Weatherization section. This Public Comment keeps the existing language in section 1503.5 as-is. And the IECC language is appropriately revised to reference the requirements of the ventilation sections in the IBC and IRC.
S5-16 Part II
IRC: R908.7 (New).

**Proposed Change as Submitted**

**Proponent**: Mike Fischer, Kellen, representing Asphalt Roofing Manufacturers Association
(mfischer@kellencompany.com)

2015 International Residential Code

Add new text as follows:

**R908.7 Attic and rafter ventilation.** For roof replacement over attics that require ventilation, intake and exhaust vents shall be provided in accordance with Section R806 and the vent product manufacturer’s approved installation instructions.

**Reason:** IRC and IEBC: This proposal clarifies the intent of the code and removes ambiguity. Ventilation of an attic space should be provided for roof replacement projects in addition to new construction.

IBC: Section 1503.5 can be interpreted to apply to roofing projects types—new construction, replacement and re-cover, and does not differentiate between vented and unvented attics. Adding the proposed new language to the Reroofing section of the code and making it specific to roof replacement over vented attics clarifies the intent of the code.

**Cost Impact:** Will not increase the cost of construction

The proposal adds no new requirements.

---

**Public Hearing Results**

**Part II**

**Committee Action:** Disapproved

**Committee Reason:** The proposal provides no exception for roof recover and no recognition of existing roof vents.

**Assembly Action:** None
Proposed Change as Submitted

Proponent: Dennis Richardson, American Wood Council, representing American Wood Council (drichardson@awc.org)

2015 International Building Code

Add new text as follows:

1503.7 Ventilation required beneath balcony or elevated walking surfaces. Enclosed framing in exterior balconies and elevated walking surfaces that are exposed to rain, snow, or drainage from irrigation, where the structural framing is protected by an impervious moisture barrier, shall be provided with openings that provide a net free cross ventilation area not less than 1/150 of the area of each separate space. Where framing supports such surfaces over 30 inches (762 mm) above grade, the ventilation openings shall be designed to allow inspection of framing material.

Reason: This change clarifies the intent of the code when a balcony or elevated walking surface serves as a weather resistant barrier and the joist spaces below are enclosed, cross ventilation is required as for enclosed rafter spaces of roofs. When the ventilation is provided for elevated walking surfaces, the ventilation openings must be designed to accommodate routine inspection of the framing material for decay or corrosion.

Cost Impact: Will increase the cost of construction

Some vent openings may need to be modified to accommodate inspection of framing material. Many vent covers that are easily removable and re-installed with hand tools already comply with the intent of this requirement.

Public Hearing Results

Committee Reason: The committee understands there is a problem that needs to be addressed, but believes the proposed requirement should only apply to wood and possibly light-gage steel. The fire-rating issues need to be correlated, probably in one big change, so that they allow these openings if they are small so that the inspections can be made and ventilation can be provided. As written this would be creating a conflict in the code. The committee would like to see more specificity on the inspection portals, giving some guidance to building officials. There is a concern that this is not the right location for this provision since most people would not think of walking surfaces as part of roofing. In addition it is not completely clear whether the problem that is being addressed is code-related versus something that was a construction defect.

Individual Consideration Agenda

Public Comment 1:

Proponent: Dennis Richardson, representing American Wood Council (drichardson@awc.org) requests Approve as Modified by this Public Comment.

Modify as Follows:

1503.7 2304.12.2.6 Ventilation required beneath balcony or elevated walking surfaces. Enclosed framing in exterior balconies and elevated walking surfaces that are exposed to rain, snow, or drainage from irrigation, where the structural framing is protected by an impervious moisture barrier, shall be provided with openings that provide a net free cross ventilation area not less than 1/150 of the area of each separate space. Where framing supports such surfaces over 30 inches (762 mm) above grade, the ventilation openings shall be designed to allow inspection of framing material.

Commenter's Reason: Section 1203.3 of the IBC is generally applied by many to require ventilation in the instance where wood supports a balcony and is enclosed. A key word is enclosed. Whenever the wood framing supporting such structures is enclosed it is more difficult for water in the assembly to dry out regardless of the source of the water (even if remaining from rain during the construction period). Even though section 1203.3 is generally applied by many, there is no specific reference to this application. It is critical to provide ventilation to these areas when enclosed and the wood supports an elevated balcony exposed to the weather.
The committee suggested this change needed to be located where it is clear it applies to wood hence the change to chapter 23.

The committee also suggested this needs to be correlated with fire-rating issues and this code change proposal could create an inconsistency. That is incorrect as Section 1406.3 of the 2015 IBC makes it clear how fire-rating issues can be resolved with the current code by extending sprinkler protection to these areas (1406.3 will be relocated in the 2018 IBC):

Section 1406.3, Exception 3: "Balconies and similar projections on buildings of Type III, IV and V construction shall be permitted to be of Type V construction, and shall not be required to have a fire resistance rating where sprinkler protection is extended to these areas."

Finally the original code change went a step further to introduce the concept of providing ventilation openings that allow the inspection for decay. This is a concept that has been introduced and is being tested by one jurisdiction where there was a balcony failure. The concept is valid but still being perfected so it has been removed from this proposal and may need to be addressed with a future code change when it is ready for prime time.

Information on this and other code change proposals by American Wood Council may be found at the following web address: www.woodcode.org (http://www.woodcode.org).
Proposed Change as Submitted

2015 International Building Code
Add new text as follows:

1504.3.1.1 Nonballasted low slope roofs. Nonballasted low slope (roof slope < 2:12) roof systems with built-up, modified bitumen, fully adhered or mechanically attached single ply shall be installed in accordance with manufacturers instructions or ANSI/SPRI WD-1.

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
ANSI/SPRI WD-1 Wind Design Standard Practice for Roofing Assemblies
Approval date 11/17/2014

Reason: ANSI/SPRI WD-1 Wind Design Standard Practice for Roofing Assemblies provides general building design considerations as well as a methodology for selecting an appropriate roofing system assembly to meet the rooftop design wind uplift pressures that are calculated in accordance with the current version of the International Building Code (IBC). This Standard Practice is appropriate for non-ballasted Built-Up, Modified Bitumen, and Single-Ply roofing system assemblies installed over any type of roof deck. It provides a rationale analysis method for determining the perimeter and corner attachment requirements for the roofing assembly. It is a companion document to ASCE7. ASCE7 is used to calculated the wind loads that will be imposed on the low slope roof, and the WD-1 standard provides the methodology to install a roof to resist those loads.

Cost Impact: Will not increase the cost of construction
This proposal does not require the use of new materials or installation practices and will not increase the cost of construction.

Analysis: A review of the standard(s) proposed for inclusion in the code, ANSI/SPRI WD-1, with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 1, 2016.

Committee Reason: The committee felt that the proposed reference standard is merely a design guide and does not provide prescriptive requirements and details. It would only be a reference document. The proposed wording could even lead to invalidating or ignoring the manufacturer's instructions.

Assembly Action: None

Public Hearing Results

Individual Consideration Agenda

Public Comment 1:

Proponent: Mike Ennis, representing SPRI, Inc. (m.ennis@mac.com) requests Approve as Modified by this Public Comment.

Modify as Follows:
2015 International Building Code

1504.3.1.1 Nonballasted low slope roofs. Nonballasted low slope (roof slope < 2:12) roof systems with built-up, modified bitumen, fully adhered or mechanically attached single ply shall be installed in accordance with manufacturers instructions or ANSI/SPRI WD-1.

Commenter's Reason: SPRI believes that ANSI/SPRI WD-1, Wind Design Standard Practice would be a good addition to the IBC. This Public Comment modifies the proposal and addresses specific comments/concerns that were raised at the Committee Action Hearings.
Comment: The proposed reference standard is merely a design guide and does not provide specific requirements and details. It would only be a reference document.

ANSI/SPRI WD-1 provides general building design considerations as well as a methodology for selecting an appropriate roofing system assembly to meet the rooftop design wind uplift pressures that are calculated in accordance with the current version of the International Building Code (IBC). As noted in the comment it does not provide specific construction details however it does contain very useful information including a rationale analysis method that can be used to determine the proper way to secure the roof membrane systems in the corner and perimeter areas where the wind loads are the greatest. The information is useful for design professionals, contractors and manufacturers.

Comment: The proposed wording could lead to ignoring or even invalidating manufacturers instructions.

This standard was developed by manufacturers to assist industry professionals responsible for designing low slope roof systems to resist design wind loads. It is to be used in conjunction with manufacturers instructions as pointed out in the Introduction section of the standard, so it not be used to ignore or invalidate manufacturers instructions. To provide additional clarification the code change proposal has been revised to state that manufacturers instructions and ANSI/SPRI WD-1 shall be used instead of manufacturers instructions or ANSI/SPRI WD-1.

S12-16
Committee Action: Disapproved

Assembly Action: None

S13-16
IBC: 1504.3.3 (New).

Proposed Change as Submitted

Proponent: Mike Ennis, SPRI, representing SPRI, Inc. (m.ennis@mac.com)

2015 International Building Code

Add new text as follows:

1504.3.3 Roof gardens and landscaped roofs, Roof gardens and landscaped roofs shall comply with Section 1507.16 and shall be installed in accordance with manufacturers instructions or ANSI/SPRI RP-14.

Reference standards type: This reference standard is new to the ICC Code Books

Add new standard(s) as follows:

BSR/SPRI RP-14 Wind Design Standard for Vegetative Roofing Systems

Reason: Section 1507.16 requires that roof gardens and landscaped roofs comply with the requirements of Chapter 15. Section 1504.1 describes requirements for the wind resistance of roofs, however no guidance is provided for designing garden and landscaped roofs for wind resistance. This proposal requests the addition of Section 1504.3.3, under Section 1504.3 Wind resistance of nonballasted roofs, that will provide requirements for garden and landscaped roofs. This new section would require that the garden and landscaped roof be installed per the requirements of ANSI/SPRI RP-14 Wind Design Standard for Vegetative Roofing Systems. This standard was developed through the ANSI standards development process and provides requirements for adhered roof systems that meet the requirements of 1504.3.1. Requirements are provided for the installation of the vegetative roof materials for various design wind speeds, building heights, exposure conditions, parapet heights and special conditions that may exist on the roof. The standard was developed in cooperation with Green Roofs for Healthy Cities and contains important information developed by the National Roofing Contractors Association regarding potential blow-off of growth media. Specifically the standard requires that any areas of exposed growth media in excess of 4-inches have a system to prevent growth media blow-off.

Cost Impact: Will increase the cost of construction
Because the ANSI/SPRI RP-14 standard requires that areas with exposed growth media in excess of 4-inches have a system to prevent growth media blow-off the cost of construction could be increased for some vegetative roof systems.

Analysis: A review of the standard(s) proposed for inclusion in the code, ANSI/SPRI RP-14, with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 1, 2016.

S13-16 : 1504.3.3 (NEW)-ENNIS10874

Public Hearing Results

Committee Reason: The proposed referenced standard is written as a design guide, rather than providing prescriptive requirements. As written, this change would allow the referenced standard to be used as a way around the manufacturers instructions. Also the proposal is referring to roof gardens and landscaped roofs, rather than vegetative roofs.

Assembly Action: None

Individual Consideration Agenda

Proponent: Mike Ennis, representing SPRI, Inc. (m.ennis@mac.com) requests Approve as Submitted.

Commenter's Reason: Section 1507.16 currently requires that roof gardens and landscaped roofs comply with the requirements of Chapter 15, however there are no requirements provided that would allow the code official to verify compliance with the wind resistance requirements of this chapter. ANSI/SPRI RP-14, Wind Design Standard for Vegetative Roofing Systems provides these requirements. This Public Comment requests that the code change proposal be accepted as submitted and addresses the following comments/questions raised at the Committee Action Hearings.
Comment: The proposed reference standard is merely a design guide and does not provide prescriptive requirements and details.
The standard does provide prescriptive requirements. It provides requirements for the field, perimeter and corner areas of the roof for three different designs. It then provides a series of Tables that determines where these systems can be used based on the building height, design wind speed, exposure category and parapet height. Further the standard provides specific requirements for building specific issues such as large openings in walls, rooftop projects, positive building pressures and others.

Comment: The change would allow for the referenced standard to be used as a way around manufacturers instructions
This standard has been developed by manufacturers as a consensus standard for installing vegetative roof systems that will resist design wind speeds. It is not a way around manufacturers instructions. Currently there are no requirements in the code for wind resistance of vegetative roof systems. The allowance for the use of manufacturers instructions was included as an option after the last round of code change hearings. During that cycle a concern was raised by the Committee that this standard did not address all possible vegetative roof systems. This is true due to the fact that there are a wide variety of options available. So, the option of using manufacturers instructions was added to address this issue.

Comment: The proposal is referring to roof gardens and landscaped roofs, rather than vegetative roofs. Roof gardens and landscaped roofs are the terms used in the IBC. The standard is using the term vegetative roofs that has been adopted by ASTM and is used in standards dealing with these systems.

S13-16
Proposed Change as Submitted

Proponent: Wanda Edwards, representing RCI, Inc. (wedwards@rci-online.org)

2015 International Building Code

Revise as follows:

1504.5 Edge securement for low-slope roofs. Low-slope
In hurricane prone regions as defined in Section 202, low-slope built-up, modified bitumen and single-ply roof system metal edge securement, except gutters, shall be designed and installed for wind loads in accordance with Chapter 16 and tested for resistance in accordance with Test Methods RE-1, RE-2 and RE-3 of ANSI/SPRI ES-1, except $V_{ult}$ wind speed shall be determined from Figure 1609.3(1), 1609.3(2) or 1609.3(3) as applicable.

Reason: ES-1 was proposed for inclusion in the code based upon research and damage assessments after a hurricane. ES-1 contains provisions for design and testing. While it is appropriate to require to use of ES-1 in hurricane-prone areas and higher wind zones, there is no evidence that would suggest that there are substantial edge metal securement failures in other areas of the country. Requiring the use of ES-1 adds unnecessary costs to projects. This proposal would require ES-1 in hurricane-prone areas only, and will lower project costs for projects outside the hurricane-prone areas. The amount of the savings will depend on the design and size of a building.

Cost Impact: Will not increase the cost of construction

ES-1 contains special design and testing requirements. Not requiring testing in areas not prone to hurricanes will save project costs. The amount of the saving will depend on the design and the size of the building.

Public Hearing Results

Committee Reason: The committee felt there was not sufficient justification to lower the threshold on where the edge securement requirements apply. Rather than limiting to hurricane prone regions, there is more interest in establishing the threshold at 115 mph.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Jason Wilen AIA CDT RRO, National Roofing Contractors Association (NRCA), representing National Roofing Contractors Association (NRCA) (jwilen@nrca.net) requests Approve as Modified by this Public Comment.

Replace Proposal as Follows:

2015 International Building Code

1504.5 Edge securement for low-slope roofs. Low-slope built-up, modified bitumen and single-ply roof system metal edge securement, except gutters, shall be designed and installed for wind loads in accordance with Chapter 16 and tested for resistance in accordance with Test Methods RE-1, RE-2 and RE-3 of ANSI/SPRI ES-1, except $V_{ult}$ wind speed shall be determined from Figure 1609.3(1), 1609.3(2) or 1609.3(3) as applicable.

Exception: The following need not comply with the requirements of this section:

1. A building with a mean roof height of 30 feet (9144 mm) or less.
2. A building located in an area where the applicable basic design wind speed, $V$, is 115 mph (51.4 m/s) or less.

Commenter's Reason: In response to concerns raised by The Structural Committee, we have modified the proposed modifications to section 1504.5 to more precisely limit when buildings are subjected to the requirements of this section. We have created an exception to exempt buildings outside of all areas with design wind speeds less than those for hurricane-prone regions. Additionally we have exempted buildings with a mean roof height of 30 feet or less because these buildings experience...
relatively low roof area perimeter wind pressures.
S16-16
IBC: 1504.5.1 (New).

Proposed Change as Submitted

Proponent: Mike Ennis, representing SPRI, Inc. (m.ennis@mac.com)

2015 International Building Code

Add new text as follows:

**1504.5.1 Gutter securement for roofs**, Roof gutters shall be designed and installed for wind loads in accordance with Chapter 16 and tested for resistance in accordance with ANSI/SPRI GT-1.

Reference standards type: This reference standard is new to the ICC Code Books

Add new standard(s) as follows:

BSR/SPRI GT-1 Test Standard for Gutter Systems

Reason: Currently the IBC requires that low-slope built-up, modified bitumen, and single-ply roof system metal edge securement be tested to resist wind and static loads, but specifically excludes gutters that are used to secure these roof systems in many cases. Studies of the aftermath of hurricanes revealed that many gutter systems did not resist the loads that occur during high wind events. Examples of these observations are shown below. SPRI developed the gutter test standard to address this issue. The wind resistance tests included in this standard measure the resistance of the gutter system to wind forces acting outwardly (away from the building) and to wind forces acting upwardly tending to lift the gutter off of the building. The standard also measures the resistance of the gutter system to static forces of water, snow and ice acting downward. Following are examples of gutter failures during high wind events observed during investigations conducted by the Roofing Industry Committee on Weather Issues (RICOWI).

Figure 1

Figure 1 is a photo taken of the gutter/cleat attachment after Hurricane Ike, and is a good example of the damage progression. This building, located in Anahuac, TX experienced wind speeds of 110 mph. The inspection team determined that an overhanging gutter and fractured nailer provided the starting point for peel-back of this multi-ply membrane. The roof membrane peeled away from the insulation layer over most of the roof as shown in Figure 2.
Figure 3 is a photo of a building located in Dickinson, TX after Hurricane Ike. This building experienced wind speeds of 100 mph.

In this case the inspection team determined that a cornering wind caused detachment of the gutter and metal edge, allowing wind to infiltrate and pressurize the roof membrane which led to roll-back of the metal edge and roof membrane, exposing the underlying substrate.
Figure 4

Figure 4 is of a building located in Lumberton, MS. This photo was taken after Hurricane Katrina. Estimated wind speed at this location was 110 to 120 mph.

The inspection team noted that approximately two-thirds of the roof membrane was blown off of the roof. Initial failure appears to have occurred at the south roof edge where approximately 25 feet of gutter and edge nailer separated from the structure. A vented 3 ft deep soffit may have contributed to the damage by pressurizing the space between the deck and the roof assembly. However, the roof assembly may have been pressurized by failure of the south roof edge.

**Cost Impact:** Will increase the cost of construction

A cost comparison was done between a gutter system that would and would not resist design wind loads. There was no difference in the cost of the two systems, so the cost difference may be limited to the cost of testing the gutter system. This cost is estimated to be $2,500 to $3,000.

**Analysis:** A review of the standard(s) proposed for inclusion in the code, ANSI/SPRI GT-1, with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 1, 2016.

---

**Committee Action:** Disapproved

**Committee Reason:** Disapproval is consistent with the committee's action on S24-16. The proposal lacks prescriptive requirements for field fabrication of gutters. There are concerns with the language with the draft of the proposed referenced standard - it should require that they "resist" or "withstand" the wind loads. Also, gutters are already required to resist these loads, even without this change.

**Assembly Action:** None
Public Comment 1:

Proponent: Mike Ennis, SPRI, Inc., representing SPRI, Inc. (m.ennis@mac.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

1504.5.1 Gutter securement for low-slope roofs. Roof gutters, low-slope built-up, modified bitumen, and single ply roof external gutter securement, shall be designed and installed for wind loads in accordance with Chapter 16 and tested for resistance in accordance with Test Methods G-1 and G-2 of ANSI/SPRI GT-1.

Commenter's Reason: ANSI/SPRI GT-1 was approved as an ANSI standard on May 26, 2016 with no objections and 100% approval from the canvass group. This standard should be added to the IBC to fill a gap that currently exists in Section 1504.5 Edge securement for low-slope roofs. This section provides requirements for testing the edge securement for low slope roofs (the most vulnerable area of the roof when exposed to wind) but excepts gutter because prior to the development of ANSI/SPRI GT-1 no test standard existed to evaluate gutter securement. Gutters are also used to secure the edge of low-slope roof systems. The securement of the edge system, including gutters, have failed in hurricane and non-hurricane prone zones. It is important to note that the requirement is to test the gutter to resist the design wind loads for the area where it will be used. This proposal limits the test requirements to low-slope roofs, consistent with the current Section 1504.5 and addresses the following concerns raised during the Code Action Hearings:

Comment: Gutters are already required to resist these loads without this change.

The Plumbing Code does require gutters to resist water loads. There are no requirements in the code to verify that gutters resist wind loads. This proposal eliminates the test requirement for water loads (Test G-3) and only requires tests G-1 and G-2 that evaluate the ability of the gutter to resist wind load forces. These requirements are best addressed in the IBC.

Comment: The proposal lacks prescriptive requirements for field fabrication of gutters.

The design of field fabricated gutters can be tested just as factory fabricated gutters. Once the design is tested it can be used in the field whenever desired and the test report can be provided to verify that the gutter is designed to resist the calculated wind loads for the application.
Proposed Change as Submitted

Proponent: Mike Ennis, representing SPRI Inc. (m.ennis@mac.com)

2015 International Building Code

Revise as follows:

1504.7 Impact resistance. Roof coverings installed on low-slope roofs (roof slope < 2:12) in accordance with Section 1507 shall resist impact damage based on the results of tests conducted in accordance with ASTM D 3746, ASTM D 4272, CGSB 37-GP-52M or the "Resistance to Foot Traffic Test" tests conducted in Section 5.5 of FM 4470 or tests conducted in accordance with procedures adapted from UL2218.

Reason: FM 4470 should not be referenced since it is not a consensus standard. UL 2218 should be added because it is a consensus standard that can be used to evaluate the impact resistance of roof coverings. Although the scope of UL2218 is applicable to steep slope roofs (a limitation in the scope of the standard), the UL2218 testing method when adapted is appropriate to use as a testing method for materials applied to low slope roofs as well.

Cost Impact: Will not increase the cost of construction

This proposal deletes reference to a non-consensus standard and replaces it with a consensus standard. It will not impact the cost of construction.

Analysis: A review of the standard(s) proposed for inclusion in the code, UL 2218, with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 1, 2016.

Public Hearing Results

Committee Action: Disapproved

Committee Reason: The committee felt the proposed language could be applied in many ways, thus the intent of this proposal is unclear. How are the procedures "adapted" from the proposed referenced standard? Perhaps a public comment is in order that could clarify that. Adaptation could be anything and the scope of the referenced standard is critical. It is also not clear why the FM 4470 referenced standard is removed.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Mike Ennis, SPRI, Inc., representing SPRI, Inc. (m.ennis@mac.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

1504.7 Impact resistance. Roof coverings installed on low-slope roofs (roof slope < 2:12) in accordance with Section 1507 shall resist impact damage based on the results of tests conducted in accordance with ASTM D 3746, ASTM D 4272, CGSB 37-GP-52M, the "Resistance to Foot Traffic Test" tests conducted in Section 5.5 of FM 4470 or tests conducted in accordance with procedures adapted from UL2218.

Commenter's Reason: This Public comment is being submitted to revise the code change proposal to address questions raised at the Committee Action Hearings as follows: Comment: The committee felt the proposed language could be applied in many ways, thus the intent of this proposal is unclear. How are the procedures "adapted" from the proposed referenced standard? Perhaps a public comment is in order that could clarify that. Adaptation could be anything and the scope of the referenced standard is critical.

Response: The "procedures adapted from" concept has precedence. The terms were used when ASTM D3161 was solely for...
asphalt shingles, but the test method could easily be used for other material type. In this case the “procedures adapted from” was to allow the use of a standard for a different orientation. Several low slope roofing systems have been tested and certified using this impact test method, see UL’s Certification Directory - Roofing Systems (TGFU).

Comment: It is also not clear why the FM 4470 referenced standard is removed.

Response: The reference to FM4470 has been added back into the proposal.
Proposed Change as Submitted

Proponent: Mike Fischer, Kellen, representing Asphalt Roofing Manufacturers Association, representing Asphalt Roofing Manufacturers Association (mfischer@kellencompany.com)

2015 International Residential Code

TABLE R905.11.2
MODIFIED BITUMEN ROOFING MATERIAL STANDARDS

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>STANDARD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acrylic coating</td>
<td>ASTM D 6083</td>
</tr>
<tr>
<td>Asphalt adhesive</td>
<td>ASTM D 3747</td>
</tr>
<tr>
<td>Asphalt cement</td>
<td>ASTM D 3019</td>
</tr>
<tr>
<td>Asphalt coating</td>
<td>ASTM D 1227; D 2824</td>
</tr>
<tr>
<td>Asphalt primer</td>
<td>ASTM D 41</td>
</tr>
<tr>
<td>Modified bitumen roof membrane</td>
<td>ASTM D 6162; D 6163; D 6164; D 6222; D 6223; D 6298; CGSB 37-GP-56M</td>
</tr>
</tbody>
</table>

Revise as follows:

R905.12.2 Material standards. Thermoset single-ply roof coverings shall comply with ASTM D 6377, or ASTM D 5019 or CGSB 37-GP-52M.

R905.13.2 Material standards. Thermoplastic single-ply roof coverings shall comply with ASTM D 4434, ASTM D 6754, or ASTM D 6878 or CGSB CAN/CGSB 37.54.

Reason: The proposal removes withdrawn Canadian standards.

Cost Impact: Will not increase the cost of construction
The referenced standards have been withdrawn and are invalid.

Public Hearing Results

Part II
Committee Action: Approved as Submitted
Committee Reason: This code change removes a referenced standard that has been withdrawn.
Assembly Action: None

Individual Consideration Agenda

Proponent: Ed Berkel, representing ICC Code Correlation Committee (ccc@icc.org) requests Disapprove.

Commenter’s Reason: The Code Correlation Committee requests disapproval for this code change proposal to allow the membership to review and consider action for the changes proposed to the IRC. When compiling these code change proposals, ICC staff failed to separate the items belonging to the IRC and therefore the IRC-Building Committee did not have the opportunity to consider these items separately, as is required for all code change proposals to the IRC. The request for disapproval is simply to place this item on the Public Comment Hearing Agenda for individual consideration. Note that the IBC-Structural Committee recommendation for both parts was Approval As Submitted.
Note that the CCC has no technical opinion regarding this issue. If no one comes forward with the opinion that this item should be heard during the Public Comment Hearings by the time of the start of the IRC-Building portion of the hearings, CCC will withdraw this public comment, thus relegating the item to the Consent Agenda.
The Code Correlation Committee is a standing committee of the International Code Council whose objectives, procedures and organization are set forth in Council Policy CP#44-13. The objective of the Code Correlation Committee is to maintain technical and editorial consistency among the International Codes and to assist staff in the evaluation and processing of code change proposals and comments that are exclusively editorial.

**Analysis:** The Code Correlation Committee submitted this public comment at the request of ICC Staff because a mistake was initially made in processing this code change proposal. When compiling these code change proposals, ICC staff failed to separate the items belonging to the IRC and therefore the IRC-Building Committee did not have the opportunity to consider these items separately, as is required for all code change proposals to the IRC. *Note that the CCC intends to withdraw this public comment, if no one comes forward requesting that this item be considered individually during the Public Comment Hearings.* If anyone desires that this proposal be considered individually during the public comment hearings, please notify ICC Staff, Manager of Codes, Dave Bowman: dbowman@icc safe.org before the start of the IRC-Building portion of the Public Comment Hearings Oct. 19 - 26.
NOTE: PART I DID NOT RECEIVE A PUBLIC COMMENT AND IS REPRODUCED FOR INFORMATIONAL PURPOSES ONLY

S18-16 Part I


Proposed Change as Submitted

Proponent: Mike Fischer, Kellen, representing Asphalt Roofing Manufacturers Association, representing Asphalt Roofing Manufacturers Association (mfischer@kellencompany.com)

2015 International Building Code

Revise as follows:

1504.7 Impact resistance. Roof coverings installed on low-slope roofs (roof slope < 2:12) in accordance with Section 1507 shall resist impact damage based on the results of tests conducted in accordance with ASTM D 3746, ASTM D 4272, CGSB 37-GP-52M or the “Resistance to Foot Traffic Test” in Section 5.5 of FM 4470.


1507.12.2 Material standards. Thermoset single-ply roof coverings shall comply with ASTM D 4637 or ASTM D 5019 or CGSB 37-GP-52M.

1507.13.2 Material standards. Thermoplastic single-ply roof coverings shall comply with ASTM D 4434, ASTM D 6754, or ASTM D 6878 or CGSB CAN/CGSB 37-54.

Reason: The proposal removes withdrawn Canadian standards.

Cost Impact: Will not increase the cost of construction

The referenced standards have been withdrawn and are invalid.

S18-16 Part I:

1504.7-

FISCHER16842

Public Hearing Results

Part I

Committee Action: Approved as Submitted

Committee Reason: This code change removes a referenced standard that has been withdrawn.

Assembly Action: None
Proponent: Mike Ennis, representing SPRI Inc. (m.ennis@mac.com)

2015 International Building Code
Revise as follows:

1504.4 Ballasted Aggregate surfaced low-slope roof systems. Ballasted low-slope (roof slope < 2:12) single-ply roof system coverings installed in accordance with Sections 1507.12 and 1507.13 shall be designed in accordance with Section 1504.8 and ANSI/SPRI RP-4. Aggregate surfaced built-up roofs and aggregate surfaced sprayed polyurethane foam roofing shall be designed in accordance with Section 1504.8.

1504.8 Aggregate

Loose aggregate used as surfacing for roof coverings and aggregate, gravel, or stone used as ballast shall not be used on the roof of a building located in a hurricane-prone wind-borne debris region as defined in Section 202, or on any other building with a mean roof not meeting the aggregate size and building parapet height exceeding that permitted by requirements of Table 1504.8 based on the exposure category and basic wind speed at the site.

### Table 1504.8

<table>
<thead>
<tr>
<th>NOMINAL DESIGN WIND SPEED, $v_{asd}$ (mph)</th>
<th>MAXIMUM ALLOWABLE MEAN ROOF HEIGHT PERMITTED FOR BUILDINGS WITH AGGREGATE ON THE ROOF IN AREAS OUTSIDE A HURRICANE-PRONE REGION Minimum Required Parapet Height (inches)$^b$ For Aggregate Surfaced Roof Coverings $^c,d$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Exposure category</td>
</tr>
<tr>
<td></td>
<td>B</td>
</tr>
<tr>
<td>85</td>
<td>170</td>
</tr>
<tr>
<td>90</td>
<td>140</td>
</tr>
<tr>
<td>95</td>
<td>120</td>
</tr>
<tr>
<td>100</td>
<td>65</td>
</tr>
<tr>
<td>106</td>
<td>40</td>
</tr>
<tr>
<td>110</td>
<td>30</td>
</tr>
<tr>
<td>115</td>
<td>20</td>
</tr>
<tr>
<td>120</td>
<td>15</td>
</tr>
<tr>
<td>Greater than 120</td>
<td>NP</td>
</tr>
</tbody>
</table>

#### WIND EXPOSURE AND NOMINAL DESIGN WIND SPEED $v_{asd}$ (MPH)$^e,f$

<table>
<thead>
<tr>
<th>ASTM Gradation</th>
<th>Mean Roof Height $^g$ (ft)</th>
<th>Exposure B</th>
<th>Exposure C</th>
<th>Exposure D</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM 15</td>
<td>15</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>D1863 20</td>
<td>20</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>No. 7 30</td>
<td>30</td>
<td>13</td>
<td>27</td>
<td>41</td>
</tr>
<tr>
<td>or No. 67 40</td>
<td>40</td>
<td>15</td>
<td>27</td>
<td>37</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>17</td>
<td>27</td>
<td>37</td>
</tr>
<tr>
<td>ASTM 60</td>
<td>60</td>
<td>18</td>
<td>27</td>
<td>37</td>
</tr>
<tr>
<td>D7655 80</td>
<td>80</td>
<td>21</td>
<td>32</td>
<td>42</td>
</tr>
<tr>
<td>No. 4 100</td>
<td>100</td>
<td>23</td>
<td>34</td>
<td>44</td>
</tr>
<tr>
<td></td>
<td>125</td>
<td>25</td>
<td>35</td>
<td>45</td>
</tr>
<tr>
<td></td>
<td>150</td>
<td>27</td>
<td>37</td>
<td>45</td>
</tr>
<tr>
<td>ASTM 15</td>
<td>15</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>D1863 20</td>
<td>20</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>No. 6 30</td>
<td>30</td>
<td>13</td>
<td>25</td>
<td>38</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>13</td>
<td>25</td>
<td>38</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>12</td>
<td>27</td>
<td>38</td>
</tr>
<tr>
<td></td>
<td>60</td>
<td>13</td>
<td>27</td>
<td>38</td>
</tr>
<tr>
<td></td>
<td>80</td>
<td>16</td>
<td>30</td>
<td>43</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>18</td>
<td>30</td>
<td>43</td>
</tr>
</tbody>
</table>
For SI: 1" = 25.4 mm; 1 foot = 304.8 mm; 1 mile per hour = 0.447 m/s.

a. Mean roof height as defined in ASCE 7.

b. For intermediate values of $V_{asd}$, the height associated with the next higher value of $V_{asd}$ shall be used, or direct interpolation is permitted.

c. NP = gravel and stone not permitted for any roof height.

d. $V_{asd}$ shall be determined in accordance with Section 1609.3.1.

a. Interpolation between wind speeds and mean roof heights as defined in ASCE 7 shall be permitted.

b. Aggregate surfaced roofs shall not be permitted for $V_{asd}$ wind speeds greater than 120 mph, or where the building height exceeds 150 feet.

c. Wind exposure shall be determined in accordance with Chapter 16.

d. $V_{asd}$ shall be determined in accordance with Section 1609.3.1.

e. Where the minimum required parapet height is indicated to be 0 inches (0mm), a gravel stop shall be provided that extends a minimum 2 inches from the roof surface, but not less than the height of the aggregate.

Reason: Requirements for the use of aggregate surfaced roofs were revised in the IBC in 2006 and 2009. These revised requirements are not based on the K-W design method (Kind Wardlaw 1976), the wind tunnel studies underlying the K-W design method (Kind 1977), or a quantitative analysis of observed good and bad roofing system performances in real wind events. Instead, current building code requirements are based on variation in surface pressure with building height which is known to be an inappropriate predictor of aggregate blow-off or scour due to pressure equalization effects (Smith, 1997).

Furthermore, these recent requirements do not address critical parameters such as aggregate size and parapet height which govern performance. This code change proposal replaces the current Table 1504.8 with one based on the K-W design method and new research by the Asphalt Roofing Manufacturers Association (ARMA) (Crandell Fischer RCI 2010). Results demonstrate that the use of aggregate-surfaced roofing systems is a viable option in high wind areas with appropriate aggregate sizing and parapet design. The Kind-Wardlaw design method has been simplified, improved, and calibrated to a number of field observations to refine its application to low-slope, built-up roof (BUR) and sprayed polyurethane foam (SPF) roof systems (Crandell Smith Hugo Conference 2010).

The proposed Table addresses the critical parameters of aggregate size and parapet height.

Two types of roof coverings: ballasted single ply roofs and those with aggregate surfaces, such as Builtup roofs (BUR) and certain spray polyurethane roof systems are covered by this Table. Over 6 billion square feet of ballasted single ply roofing applications have been installed over the last two decades. The vast majority of these systems have performed very well with respect to their resistance to wind pressure loads. However some damage has been observed due to aggregate blowing off non-code compliant roofs during high wind events. The proposed Table is based on over 200 wind tunnel tests in addition to over 40 years of field experience and observations from hurricane investigation teams. The proposed Table, and the remaining portions of Section 1504.8 provide restrictions on the use of ballasted single ply roof systems that will allow for the responsible use of aggregate surfacing that is a cost effective method to keep the roof system in place and to improve the energy performance of the building.

Bibliography: REFERENCES:

Cost Impact: Will not increase the cost of construction

This proposal will provide additional design options for aggregate surfaced roofs and will not increase the cost of construction.
**Public Hearing Results**

**Committee Action:** Disapproved

**Committee Reason:** While the committee felt this proposal is headed in the right direction, the amount of conflicting testimony indicates that work is needed on these requirements, a revised version should be submitted in the public comment phase.

Questions were raised on how the research results have been turned into code provisions. As formatted with options for wind speed, exposure and roof heights the table is complex and a more simplified, straightforward table that is not so exhaustive would be preferable even if it is more conservative. Due to the difficulty reading the column with ASTM gradation, it could be preferable to split this into two tables. There is also a concern over whether the reference to a specific product type is appropriate.

**Assembly Motion:** As Submitted

**Online Vote Results:** Failed

Support: 33.46% (91) Oppose: 66.54% (181)

**Assembly Action:** None

**Individual Consideration Agenda**

**Public Comment 1:**

Proponent: Jay Crandell, P.E., ARES Consulting, representing Single-Ply Roofing Industry (jcrandell@aresconsulting.biz) requests Approve as Modified by this Public Comment.

Modify as Follows:

**2015 International Building Code**

**1504.4 Aggregate surfaced low-slope roof systems.** Ballasted low-slope (roof slope < 2:12) single-ply roof system coverings installed in accordance with Sections 1507.12 and 1507.13 shall be designed in accordance with Section 1504.8 and ANSI/SPRI RP-4. Aggregate surfaced built-up roofs and Other aggregate surfaced sprayed polyurethane foam roofing roofs shall be designed in accordance with Section 1504.8.

**Commenter's Reason:** The IBC structural committee indicated that this proposal was "heading in the right direction" and recommended that a public comment be developed to address the indicated concerns. The concerns are being addressed in two PCs. This first PC is editorial and removes reference to a specific roof material type in Section 1504.4. Now, aggregate surfaced roofs are generically referenced as appropriate. A second (separate) PC addresses the committee's recommendation to simplify and improve readability of the table (which was partly a font size or CDP access table formatting issue). It also provides additional information regarding the development of the S19 proposal as requested by the committee.

**Public Comment 2:**

Proponent: Jay Crandell, P.E., ARES Consulting, representing Single-Ply Roofing Industry (jcrandell@aresconsulting.biz) requests Approve as Modified by this Public Comment.

Modify as Follows:

**2015 International Building Code**

<table>
<thead>
<tr>
<th>Aggregate Size</th>
<th>Mean Roof Height</th>
<th>B</th>
<th>Exposure A</th>
<th>Exposure B</th>
<th>Exposure C</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM D1863 (No.7 or No.67)</td>
<td>15</td>
<td>0</td>
<td>0</td>
<td>15</td>
<td>20</td>
</tr>
<tr>
<td>or ASTM D7655 (No.4)</td>
<td>20</td>
<td>0</td>
<td>12</td>
<td>17</td>
<td>23</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>13</td>
<td>15</td>
<td>21</td>
<td>27</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>17</td>
<td>20</td>
<td>26</td>
<td>32</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>23</td>
<td>26</td>
<td>33</td>
<td>40</td>
</tr>
</tbody>
</table>
For SI: 1 inch = 25.4 mm; 1 foot = 305 mm; 1 mile per hour = 0.447 m/s

a. Interpolation between wind speeds and mean roof heights as defined in ASCE 7 shall be permitted.

b. Aggregate surfaced roofs shall not be permitted for \( V_{\text{ASD}} \) wind speeds greater than 120 mph, or where the building height exceeds 150 feet.

c. Where the minimum required parapet height is indicated to be 0 inches (0 mm), a gravel stop shall be provided that extends a minimum of 2 inches from the roof surface, but not less than the height of the aggregate.

d. Wind exposure shall be determined in accordance with Chapter 16.

e. \( V_{\text{ASD}} \) shall be determined in accordance with Section 1609.3.1.

f. For exposure D, add 8 inches (203 mm) to the parapet height required for Exposure C.

**Commenter’s Reason:** The IBC structural committee indicated that this proposal was “heading in the right direction” and recommended that a public comment be developed to address the indicated concerns. The concerns are being addressed in two PCs. The first PC (separate) removes reference to a specific roof material type in the text of Section 1504.4. This second PC addresses the committee’s recommendation to simplify and improve readability of the table (which was partly a font size or CDP access table formatting issue). These revisions are technically consistent with the original proposal and the referenced research.

The committee also mentioned that questions were raised with regard to how the provisions were developed from the referenced research. The methodology (and design procedure) is clearly documented in the referenced research in an understandable, repeatable, and scientific manner (see original proposal for referenced research reports and papers). The procedure used is consistent with the findings of many wind tunnel studies and uses the same principles as applied in the ANSI/SPRI RP-4 standard currently referenced in the code. It is also consistent with the treatment of aggregate blow-off when incorporated in wind risk models. Furthermore, the analytical procedure was evaluated by comparison to numerous documented field studies of successful and failed loose aggregate surfaced roofs systems in various high wind events to confirm its ability to reliably predict performance as a means to design roofs (or develop prescriptive provisions as proposed) to prevent roof aggregate blow-off. Thus, a robust combination of current engineering practice, wind tunnel data, and field research was used to support development of the requirements as proposed for Table 1504.8.

However, this proposal does not merely provide a more academic solution. It is necessary to correct deficiencies in the current code provisions. For example, the current Table 1504.8 allows buildings up to 170’ tall or buildings in areas with design wind speeds up to 120 mph with NO PARAPET which creates a general safety hazard (e.g., falling debris from the roof) and unacceptable wind damage vulnerability (i.e., aggregate blow-off risk). This proposal corrects this safety and building performance issue based on correct scientific principles and sound engineering practices. For example, buildings with loose aggregate surfaced roofs in a 120 mph wind zone would be required to have parapets with heights ranging from 20 inches (minimum for 15’ mean roof height and wind exposure B with the large aggregate size specified) to 72 inches (maximum for 150’ mean roof height and wind exposure D with the smaller aggregate size permitted). Similar improvements are made for lesser wind speed conditions as shown in proposed Table 1405.8. This is a significant improvement over 0 inches (no parapet) as currently allowed in the code.

If implemented, this proposal will serve to prevent many past observations of roof aggregate blow-off from being repeated. Simply put, this proposal is implementing lessons learned in a rational, scientific manner based on real-world and wind tunnel laboratory data to prevent history from repeating itself in an unfavorable manner. Any argument against this proposal as being inadequate is an argument to leave the code in a far worse condition from a building safety and performance standpoint.

**Proponent:** Scott Campbell, representing Portland Cement Association (scampbell@cement.org) requests Approve as Submitted.

**Commenter’s Reason:** This proposal uses the latest research to improve the design for aggregate surfaced low slope roofs. In addition, the design moves towards considering parapet height, and not just roof height, which is in line with what factors...
actually affect the performance of aggregate roof systems. The proposed change would increase public safety and property protection.
Proposed Change as Submitted

Proponent: Mike Fischer, Kellen, representing Asphalt Roofing Manufacturers Association (mfischer@kellencompany.com)

2015 International Building Code
Revise as follows:

1504.8 Aggregate

Loose-laid aggregate used as surfacing for roof coverings and aggregate, gravel or stone used as ballast shall not be used on the roof of a building located in a hurricane-prone region as defined in Section 202, or on any other building with a mean roof height exceeding that permitted by Table 1504.8 based on the exposure category and basic wind speed at the site.

Exception. Aggregate shall be permitted on roofs located outside of the windborne debris region using approved parapet design to control aggregate blow-off, when the parapet systems have been designed by a registered design professional.

Reason: The use of aggregate on roofs has been the subject of debate for the past decades as post-storm evaluations of building performance has led to significant restrictions to the use of roofing aggregate, despite research that provides recommendations on the use of parapets to prevent roof aggregate blow-off under design conditions. The proposal provides an option for the use of aggregate when the roof system has an engineered parapet control system. It further limits the current aggregate restrictions to loose-laid aggregate; there are methods in use for the embedment of aggregate into the roofing material such as asphalt built-up roof systems approved for use under the Florida Building Code.

Cost Impact: Will not increase the cost of construction
The proposal provides greater product availability due to increased flexibility.

Public Hearing Results

Committee Action: Disapproved
Committee Reason: The wording of the proposed exception for aggregate on roofs has problems with enforceability. There is a question on what is meant by "controlling" aggregate blow-off and no prescriptive requirements are provided to make this clear. Another concern was raised over the use of the term "parapet systems".

Assembly Motion: As Modified
Online Vote Results: Failed
Support: 20.3% (55) Oppose: 79.7% (216)
Assembly Action: None

Online Floor Modification:

1504.8 Aggregate. Loose-laid aggregate used as surfacing for roof coverings and aggregate, gravel or stone used as ballast shall not be used on the roof of a building located in a hurricane-prone region as defined in Section 202, or on any other building with a mean roof height exceeding that permitted by Table 1504.8 based on the exposure category and basic wind speed at the site.

Exception. Aggregate shall be permitted on roofs located outside of the windborne debris region using approved parapet design to control aggregate blow-off, when the parapet systems have been designed by a registered design professional.

Individual Consideration Agenda

Proponent: Mike Fischer, Kellen, representing Asphalt Roofing Manufacturers Association (mfischer@kellencompany.com) requests Approve as Submitted.

Commenter’s Reason: Roofing aggregate has been appropriately used for many decades, and the current code provisions are flawed- the code permits aggregate to be used without parapet control while overly restricting this product in hurricane-prone regions. The proposal provides guidance to the code official on a means to accept engineered design. The alternate means and methods provisions in the IBC lack detail on specific applications; this proposal will solve that need.
The proposed code change is consistent with findings and recommendations from the National Institute of Standards and Technology (NIST) technical investigation of the 2011 tornado in Joplin Missouri (NIST 2014, Finding 19, and Recommendation 10). This change proposal is also consistent with FEMA recommendations, developed from observations of building performance in tornadoes. FEMA recommends that aggregate roof surfacing not be specified for critical facilities in tornado-prone regions (FEMA 2012). The NIST and FEMA recommendations are intended to reduce the potential number of missiles generated by the tornado, and hence reduce the potential for building damage and injury to people.

Glazing damage to Risk Category III and IV type-buildings by roof aggregate blow off, including 'self-inflicted' damage, has been well documented in severe windstorms, both tornadoes (e.g., NIST 2014, and FEMA 2007, 2010, and 2012) and hurricanes (e.g., NIST 2006 and FEMA 2005). These buildings often experience little to no structural damage, but suffer catastrophic damage to the building interior and contents that can also result in injuries and fatalities. Such was the case at St. John's Regional Medical Center (SJRMC) in Joplin, Missouri. The main buildings at SJRMC consisted of two mid-rise hospital towers and several three and four story clinic and medical office buildings. Following the May 22, 2011 Joplin tornado, despite the fact that there was no significant structural damage to any of these buildings, 14 people died due to injuries sustained while inside the buildings, or succumbed later to their injuries, 12 of which were caused by "multiple blunt-force trauma to the body" according to the death certificates (NIST 2014, p. 261). Although there was debris from many sources, blown off roof aggregate from SJRMC buildings contributed significantly to damage to the building envelopes, allowing wind and rain and debris inside of buildings (see Figure 1a and b). "The damage to these buildings included the breakage of almost all vertical glass; damage to the roof systems, including the loss of aggregate roof ballast, which became wind-borne debris that further damaged the facility and the surrounding areas" (NIST 2014, p. 317).

Although none of the main buildings at SJRMC suffered any significant structural damage, the damage to the interiors was so great that the entire Medical Center was ultimately demolished and rebuilt at a different location. Many lessons learned from the tornado were incorporated in the design of the replacement facility, including NOT using roof aggregate, as reported below by Sickles (2014).

"A blanket of rock, with some pieces the size of a golf ball, was used to weigh down the roof on the old hospital, which was built in 1965. Those actually turned into projectiles during the tornado," Felton said of the gravel. "They were being shot right through the patient room windows." [Ryan Felton, project director with McCarthy Building Companies, the firm constructing the new facility].

There will be no rocks on Mercy's new roof, but a protective layer of lightweight concrete is being incorporated into the roofing scheme.

Figure 2a and b shows another example of roof aggregate damaging the building it is supposed to be protecting. FEMA (2012) documented that aggregate from a one story section of a building at Ringgold High School was the likely source of damage to windows in a taller part of the same building during a 2011 tornado. Similarly, a hospital in Greensburg Kansas suffered glazing damage from aggregate from the ballasted single-ply membrane roofs (FEMA 2007) as shown in Figure 3. Pieces of the large aggregate (1 ½ inches in diameter, nominal) were found inside the building following the 2007 tornado.

It should be noted that the vast majority of aggregate blow-offs have occurred during hurricanes and tornadoes. The 2006 edition of IBC prohibited the use of aggregate in hurricane-prone regions. The 2006 edition also added Table 1504.4 (1504.8 in the 2015 edition), which is applicable to small aggregate used on built-up and sprayed polyurethane roofs outside of hurricane-prone regions.
prone regions. Although improvements to the Table have been proposed (Crandall and Smith, 2009), it is believed that except for tornadoes, the potential for aggregate blow-off outside of hurricane-prone regions is generally small. Because the probability of a site specific tornado strike is very low, this proposal is limited to Risk Category III and IV buildings.

Although tornadoes generate many types of debris, an aggregate surfaced roof has a tremendous number of potential missiles. For example, a ballasted 20,000 square foot roof would have about 1.6 million loose aggregates. A similarly sized built-up roof would have about 4.5 to 9 million loose aggregates, depending upon gradation (based on aggregate samples collected from a number of roofs reported by FEMA (2006, p. 5-63)). Additionally, the aggregate problem can be easily mitigated, whereas other debris sources are much more difficult to mitigate.

Note – The code change references a Figure in ICC 500. ICC 500 is a standard already referenced in the IBC for design and construction of storm shelters. Figure 304.2(1) of ICC 500 (Figure 4 below) provides a map of tornado wind speeds. The 250 mph wind speed region on that map, covering parts of the midwest and the southeast US generally known as “Tornado Alley” and “Dixie Alley” respectively, represent the most tornado-prone areas of the US.

Figure 1a - Glazing failures in hospital tower at SJRMC following the Joplin tornado (above); interior damage at SJRMC. Note the extensive amount of roof aggregate inside the building (below).
Source NIST

Figure 1b - Glazing failures in hospital tower at SJRMC following the Joplin tornado; interior damage at SJRMC. Note the extensive amount of roof aggregate inside the building.
Copyright 2011 Malcolm Carter. Used with permission.
Figure 2a - Aggregate scoured from the roof of the first story section of this high school building in Georgia during a tornado (above) likely broke the windows on the adjacent taller section of the same building (below).
Source: FEMA Mitigation Assessment Team

Figure 2b - Aggregate scoured from the roof of the first story section of this high school building in Georgia during a tornado likely broke the windows on the adjacent taller section of the same building.
Source: FEMA Mitigation Assessment Team
Figure 3 - Glazing damage at a hospital in Greensburg Kansas following a tornado in 2007. The craters shown in the right center pane and at the vehicle windshield were caused by the large aggregate blown from the ballasted single-ply membranes (FEMA 2012).

Figure 4 - Shelter Design Wind Speeds for Tornadoes - Source ICC 500-2014, International Code Council. The ICC Building Code Action Committee (BCAC) is a co-proponent of this proposal. BCAC was established by the ICC Board of Directors to pursue opportunities to improve and enhance assigned International Codes or portions thereof. In 2014 and 2015 the BCAC has held 5 open meetings. In addition, there were numerous Working Group meetings and conference calls for the current code development cycle, which included members of the committee as well as any interested party to discuss and debate the proposed changes. Related documentation and reports are posted on the BCAC website at: BCAC (http://www.iccsafe.org/codes-tech-support/codes/code-development-process/building-code-action-committee-bcac/)
Bibliography: References:


Cost Impact: Will increase the cost of construction
A variety of other types of roof systems are available for use on roofs of Category III and IV buildings located in the 250 mph area on Figure 304.2(1) of ICC 500. These alternative systems may or may not cost more than an aggregate surfaced or a ballasted roof system.

Committee Action: Disapproved
Committee Reason: The committee believes it does not make sense to restrict aggregate on roofs, using wind speed criteria that has not been used in the design of the building.

Assembly Action: None

Public Comment 1:
Proponent: Edwin Huston, representing National Council of Structural Engineers' Associations (NCSEA) (huston@smithhustoninc.com) requests Approve as Modified by this Public Comment.
Modify as Follows:

2015 International Building Code

1504.9 Surfacing and ballast materials in tornado-prone regions. Aggregate shall not be used as surfacing for roof coverings and aggregate, gravel or stone shall not be used as ballast on the roof of a Risk Category III or IV building located in areas where the wind speed is 250 MPH in accordance with Figure 304.2(1) of ICC 500.

Commenter's Reason: There was discussion among the Structural Committee during the 2018 IBC Code Development Hearings about the possibility of aggregate ballast, blown off of adjacent buildings of any Risk Category damaging a Risk Category III or IV building. While aggregate has been blown off of a building, and damaged the same building, it is also the case that aggregate ballast has been lifted from buildings, become airborne and damaged buildings downstream.

The proponent took the following photographs of glazing damage in New Orleans after Hurricane Katrina. The glazing damage in both of these buildings was caused by airborne gravel from adjacent, or nearby buildings.

New Orleans Hyatt Hotel: Gravel was removed from the hotel rooms and the windows were sealed with the white panels. The hotel finally reopened a year or so ago.

New Orleans Shopping Center Office Tower: Some unbroken windows had several "bullseye" fractures caused by gravel impacts that had not yet fractured the glazing. There were small piles of gravel at the base of the building which had struck the building, but had not yet broken windows. The proponent believes that after a sufficient number of "bullseye" damaging impacts, the glazing finally gives way.

The proponent freely admits that this is hurricane and not tornado damage, but the photographs from S22-16 clearly show glazing damage from tornados.

The Structural Engineers Association of Kansas and Missouri prepared a reconnaissance report after the May 22, 2011 Joplin, Missouri Tornado.
One of their recommendations was:

9) Codes should prohibit the use of ballasted roofs with rock or crushed stone in all construction for tornado prone areas.

During high wind tornado events, loose roof ballast (gravel) is proven to be ineffective at preventing roof blow-off. This is noted during both hurricanes and tornadoes. Roof ballast often becomes airborne debris which typically destroys glazing systems and other brittle exterior finishes. This debris can injure innocent people. As noted above, during the inspection of the roofs at St. John's Medical Center, the ballast of the roof was shifted into piles, was blown into the glass, broke the facade and landed in many of the rooms on the west side of the building, ineffectively holding the roof membrane in place, while potentially overloading other structural portions of the roof. Many hurricane prone regions of the country have enforced codes restricting the use of them, based on the same determinations as mentioned above.

While S22 didn't go that far, it is certainly a step in the right direction.

Bibliography: Structural Engineers Association of Kansas and Missouri Report "Investigations and Recommendations based on the May 22, 2011 Joplin, Missouri Tornado"

Public Comment 2:

Proponent: Edward Kulik, representing Building Code Action Committee (bcac@iccsafe.org); Marc Levitan (marc.levitan@nist.gov) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

1504.9 Surfacing and ballast materials in tornado-prone regions. Aggregate shall not be used as surfacing for roof coverings and aggregate, gravel or stone shall not be used as ballast on the roof of a Risk Category III or IV building located in areas where the region having the greatest wind speed is 250 MPH in accordance with Figure 304.2(1) of ICC 500.

Commenter's Reason: The Committee Reason for disapproval states:

"The committee believes it does not make sense to restrict aggregate on roofs, using wind speed criteria that has not been used in the design of the building."

The committee is incorrect in its understanding of the intent of this proposal.

The 250 mph wind speed zone on the ICC 500 map referenced in the code change proposal (see Figure 4) is ONLY used to geographically locate the most severe tornado-prone region of the US. We could have alternately used a map of tornado prone parts of the US developed using National Oceanic and Atmospheric Administration (NOAA) tornado climate data, and based on tornado occurrence rates instead of estimated maximum tornado wind speeds. For convenience, we instead chose to use a tornado hazard map that is already referenced in the IBC and delineates a similar region.

The intent of this code change is to reduce the windborne debris hazard, particularly to glazed openings, in tornado prone parts of the country, which is the same rationale behind the existing prohibition for aggregate in hurricane prone regions (IBC 2015 Section 1504.8). As described in more detail later in the reason statement, even a modest size aggregate surfaced or ballasted roof contains millions of potential windborne missiles, which have been documented to significantly contribute to building damage and injuries.

This proposal will have a small impact on the cost of construction. A variety of other types of roof systems are available for use on the roofs of Risk Category III and IV buildings in the tornado-prone region. These alternative systems may or may not cost more than aggregate-surfaced or ballasted roof systems.

The Applied Economics Office in the Engineering Laboratory at the National Institute of Standards and Technology conducted a study to determine the impacts of this proposal. This code change would only affect a comparatively small number of future roofs on Risk Category III and IV buildings in the tornado-prone region, which consists of all of Iowa, Missouri, Arkansas, Illinois, Indiana, Ohio, and parts of the surrounding states (see Figure 4), which could otherwise have built-up or ballasted EPDM roofs. This is estimated to be just 0.1% of roof construction in the US, based on analysis of information from the National Roofing Contractors Association Annual Market Survey (NRCA 2016), EPDM Roofing Association (EPDMRA 2016), and building stock data (FEMA, 2000) from Hazus (assuming the distribution of building occupancy types for new construction follows the distribution for the existing building stock).

For comparison, the existing prohibition on aggregate-surfaced and ballasted roofs of all Risk Category buildings in hurricane prone regions (Section 1508 of IBC 2015) impacts about 4.6 times more buildings than the number of buildings potentially...
impacted by this code change proposal.
Within the defined tornado prone region, roof construction on an estimated 0.2% of all buildings would potentially be affected.

References


S23-16
IBC: , 1504.9 (New), 202 (New).

Proposed Change as Submitted

Proponent : Jonathan Roberts, representing UL LLC (jonathan.roberts@ul.com)

2015 International Building Code
Add new definition as follows:

SECTION 202 DEFINITIONS

MODERATE HAIL EXPOSURE. One or more hail days with hail diameters greater than 1.5 in (38 mm) in a 20 year period.

SECTION 202 DEFINITIONS

SEVERE HAIL EXPOSURE. One or more hail days with hail diameters greater than 2.0 in (50 mm) in a 20 year period.

Add new text as follows:

1504.9 Roof coverings subject to hail exposure. Roof coverings installed in regions with moderate hail exposure shall be listed and labeled as Class 3. Roof coverings installed in regions with severe hail exposure shall be listed and labeled as Class 4. Roof coverings on a slope greater than 2:12 shall be tested in accordance with UL2218. Roof coverings on a slope of 2:12 or less shall be tested in accordance with procedures adapted from UL 2218.

Reference standards type:
Add new standard(s) as follows:

UL 2218-2010 Impact Resistance of Prepared Roof Covering Materials, with revisions through May 1, 2012

Reason: This new code section would require roof covering materials to have increased impact resistance in areas where there is moderate or severe hail exposure. Requiring this added level of resiliency in roof coverings reduces the frequency of replacement or repair due to hail damage and will help to reduce the negative potential impacts on the built environment and the building owners overall. Several companies currently have listings for products for both steep slope and low slope roof applications. Although the scope of UL 2218 is applicable to steep slope roofs, a limitation in the scope of the standard, the UL 2218 testing method when adapted is appropriate to use as testing method for materials applied to for low slope roofs as well.

Cost Impact: Will increase the cost of construction
Initial cost would increase. In many cases, replacement costs can be avoided, netting a decrease in life cycle cost.

Analysis: A review of the standard(s) proposed for inclusion in the code, UL 2218, with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 1, 2016.

Committee Action: Disapproved
Committee Reason: This code change does not provide definition or direction on how the hail exposure is determined. The proposed wording is questionable and there is no instruction to designer on how to apply it.

Assembly Action: None

Public Comment 1:
Proponent : Scott Campbell, representing Portland Cement Association (scampbell@cement.org) requests Approve as Modified by this Public Comment.

Modify as Follows:
SECTION 202 DEFINITIONS

MODERATE HAIL EXPOSURE. One or more two hail days with hail diameters greater than 1.5 or equal to 1.0 in (38.25 mm) in a 20 10 year period as identified in Figure 1504.9.

SEVERE HAIL EXPOSURE. One
Three or more hail days with hail diameters greater than 2.0 or equal to 1.0 in (50.25 mm) in a 20 10 year period as identified in Figure 1504.9.

1504.9 Roof coverings subject to hail exposure. Roof coverings installed in regions with moderate hail exposure shall be listed and labeled as Class 3. Roof coverings installed in regions with severe hail exposure shall be listed and labeled as Class 4. Roof coverings on a slope greater than 2:12 shall be tested in accordance with UL2218. Roof coverings on a slope of 2:12 or less shall be tested in accordance with procedures adapted from UL 2218.

Add new figure as follows:

FIGURE 1504.9
AVERAGE NUMBER OF HAIL DAYS OF 1 INCH (25 MM) OR LARGER OVER A TEN YEAR PERIOD

Commenter's Reason: Design of roofs for hail where a moderate or severe risk exists provides for a minimum level of property protection. The committee cites difficulty in predicting hail exposure as the reason to disapprove the proposal, but data on hail risk currently exists.
Bibliography: https://disastersafety.org/hail/reduce-hail-damage-to-homes/
Committee Action: Approved as Submitted

Assembly Action: None

**R905.1.1**

**Underlayment.** Underlayment for asphalt shingles, clay and concrete tile, metal roof shingles, mineral-surfaced roll roofing, slate and slate-type shingles, wood shingles, wood shakes and, metal roof panels and photovoltaic shingles shall conform to the applicable standards listed in this chapter. Underlayment materials required to comply with ASTM D 226, D 1970, D 4869 and D 6757 shall bear a label indicating compliance to the standard designation and, if applicable, type classification indicated in Table R905.1.1(1). Underlayment shall be applied in accordance with Table R905.1.1(2). Underlayment shall be attached in accordance with Table R905.1.1(3).

**Exceptions:**

1. As an alternative, self-adhering polymer-modified bitumen underlayment complying with ASTM D 1970 installed in accordance with both the underlayment manufacturer's and roof covering manufacturer's instructions for the deck material, roof ventilation configuration and climate exposure for the roof covering to be installed, shall be permitted.
2. As an alternative, a minimum 4-inch-wide (102 mm) strip of self-adhering polymer-modified bitumen membrane complying with ASTM D 1970, installed in accordance with the manufacturer's instructions for the deck material, shall be applied over all joints in the roof decking. An approved underlayment for the applicable roof covering for maximum ultimate design wind speeds, $V_{ul}$, less than 140 miles per hour shall be applied over the entire roof over the 4-inch-wide (102 mm) membrane strips.

**Reason:** Roofing underlayments are a critical component and provide protection to the roof deck and other components during installation as well as replacements and after storm events. The proposal adds a requirement that self-adhering underlayments bear a label to demonstrate compliance to the code. The labeling requirement for underlayments was part of a comprehensive proposal for the IRC by IBHS in the past cycle; it is hoped that a similar proposal will be approved for the IBC this year. The proposal also adds photovoltaic shingles to the list of roof covering materials requiring labeled underlayment materials in the IRC.

**Cost Impact:** Will increase the cost of construction

Will require products bear a label, which will add product approval costs. The IRC already contains an underlayment labeling requirement, so the cost impact is expected to be minimal.

**Public Hearing Results**

Part II

**Committee Action:** Approved as Submitted

**Committee Reason:** This code change levels the playing field for underlayment by clarifying the requirement for labeling.

**Assembly Action:** None

**Individual Consideration Agenda**

**Proponent:** Ed Berkel, representing ICC Code Correlation Committee (ccc@iccsafe.org) requests Disapprove.

**Commenter's Reason:** The Code Correlation Committee requests disapproval for this code change proposal to allow the membership to review and consider action for the changes proposed to the IRC. When compiling these code change proposals, ICC staff failed to separate the items belonging to the IRC and therefore the IRC-Building Committee did not have the opportunity to consider these items separately, as is required for all code change proposals to the IRC. The request for disapproval is simply to place this item on the Public Comment Hearing Agenda for individual consideration. Note that the IBC-Structural Committee recommendation for both parts was Approval As Submitted.

Note that the CCC has no technical opinion regarding this issue. If no one comes forward with the opinion that this item should
be heard during the Public Comment Hearings by the time of the start of the IRC-Building portion of the hearings, CCC will withdraw this public comment, thus relegating the item to the Consent Agenda.

The Code Correlation Committee is a standing committee of the International Code Council whose objectives, procedures and organization are set forth in Council Policy CP#44-13. The objective of the Code Correlation Committee is to maintain technical and editorial consistency among the International Codes and to assist staff in the evaluation and processing of code change proposals and comments that are exclusively editorial.

**Analysis:** The Code Correlation Committee submitted this public comment at the request of ICC Staff because a mistake was initially made in processing this code change proposal. When compiling these code change proposals, ICC staff failed to separate the items belonging to the IRC and therefore the IRC-Building Committee did not have the opportunity to consider these items separately, as is required for all code change proposals to the IRC. *Note that the CCC intends to withdraw this public comment, if no one comes forward requesting that this item be considered individually during the Public Comment Hearings.* If anyone desires that this proposal be considered individually during the public comment hearings, please notify ICC Staff, Manager of Codes, Dave Bowman: dbowman@icc.org before the start of the IRC-Building portion of the Public Comment Hearings Oct. 19 - 26.
Proposed Change as Submitted

Proponent: Mike Fischer, Kellen, representing Asphalt Roofing Manufacturers Association
(mfischer@kellencompany.com)

2015 International Building Code
Revise as follows:

1507.2.4 Self-adhering polymer modified bitumen sheet. Self-adhering polymer modified bitumen sheet shall comply and bear a label indicating compliance with ASTM D 1970.

1507.2.8.1 High wind attachment. Underlayment applied in areas subject to high winds [$V_{asd}$ greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's instructions. Fasteners are to be applied along the overlap not more than 36 inches (914 mm) on center.

Underlayment installed where $V_{asd}$, in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II, ASTM D 4869 Type IV, or ASTM D 6757. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with Section 1507.2.8 except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gage [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of $\frac{3}{4}$ inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with a label indicating compliance with ASTM D 1970 shall be permitted.

1507.2.9.2 Valleys. Valley linings shall be installed in accordance with the manufacturer's instructions before applying shingles. Valley linings of the following types shall be permitted:

1. For open valleys (valley lining exposed) lined with metal, the valley lining shall be at least 24 inches (610 mm) wide and of any of the corrosion-resistant metals in Table 1507.2.9.2.
2. For open valleys, valley lining of two plies of mineral-surfaced roll roofing complying with ASTM D 3909 or ASTM D 6380 shall be permitted. The bottom layer shall be 18 inches (457 mm) and the top layer a minimum of 36 inches (914 mm) wide.
3. For closed valleys (valleys covered with shingles), valley lining of one ply of smooth roll roofing complying with ASTM D 6380, and at least 36 inches (914 mm) wide or types as described in Item 1 or 2 above shall be permitted. Self-adhering polymer modified bitumen underlayment complying with a label indicating compliance with ASTM D 1970 shall be permitted in lieu of the lining material.

Underlayment applied in areas subject to high wind [$V_{asd}$ greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's installation instructions. Fasteners are to be applied along the overlap not more than 36 inches (914 mm) on center.

Underlayment installed where $V_{asd}$, in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with Sections 1507.3.3.1 and 1507.3.3.2 except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gage [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of $\frac{3}{4}$ inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment bearing a label indicating compliance with ASTM D 1970 shall be permitted.

1507.3.9 Flashing. At the juncture of the roof vertical surfaces, flashing and counterflashing shall be provided in accordance with the manufacturer's installation instructions, and where of metal, shall not be less than 0.019-inch (0.48 mm) (No. 26 galvanized sheet gage) corrosion-resistant metal. The valley flashing shall extend at least 11 inches (279 mm) from the centerline each way and have a splash diverter rib not less than 1 inch (25 mm) high at the flow line formed as part of the flashing. Sections of flashing shall have an end lap of not less than 4 inches (102 mm). For roof slopes of three units vertical in 12 units horizontal (25-percent slope) and over, the valley flashing shall have a 36-inch-wide (914 mm) underlayment of either one layer of Type I underlayment running the full length of the valley, or a self-adhering polymer-
modified bitumen sheet complying bearing a label indicating compliance with ASTM D 1970, in addition to other required underlayment. In areas where the average daily temperature in January is 25°F (-4°C) or less or where there is a possibility of ice forming along the eaves causing a backup of water, the metal valley flashing underlayment shall be solid cemented to the roofing underlayment for slopes under seven units vertical in 12 units horizontal (58-percent slope) or self-adhering polymer-modified bitumen sheet shall be installed.

1507.4.5 Underlayment and high wind. Underlayment applied in areas subject to high winds $V_{asd}$ greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's installation instructions. Fasteners are to be applied along the overlap not more than 36 inches (914 mm) on center.

Underlayment installed where $V_{asd}$ in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II, or ASTM D 4869 Type IV, or ASTM D 1970. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gage [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of $\frac{3}{4}$ inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying bearing a label indicating compliance with ASTM D 1970 shall be permitted.

1507.5.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds $V_{asd}$ greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where $V_{asd}$ in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch spacing (152 mm) at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gage [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of $\frac{3}{4}$ inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying bearing a label indicating compliance with ASTM D 1970 shall be permitted.

1507.5.7 Flashing. Roof valley flashing shall be of corrosion-resistant metal of the same material as the roof covering or shall comply with the standards in Table 1507.4.3(1). The valley flashing shall extend at least 8 inches (203 mm) from the centerline each way and shall have a splash diverter rib not less than $\frac{3}{4}$ inch (19.1 mm) high at the flow line formed as part of the flashing. Sections of flashing shall have an end lap of not less than 4 inches (102 mm). In areas where the average daily temperature in January is 25°F (-4°C) or less or where there is a possibility of ice forming along the eaves causing a backup of water, the metal valley flashing shall have a 36-inch-wide (914 mm) underlayment directly under it consisting of either one layer of underlayment running the full length of the valley or a self-adhering polymer-modified bitumen sheet complying bearing a label indicating compliance with ASTM D 1970, in addition to underlayment required for metal roof shingles. The metal valley flashing underlayment shall be solidly cemented to the roofing underlayment for roof slopes under seven units vertical in 12 units horizontal (58-percent slope) or self-adhering polymer-modified bitumen sheet shall be installed.

1507.6.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds $V_{asd}$ greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's installation instructions. Fasteners are to be applied along the overlap not more than 36 inches (914 mm) on center.

Underlayment installed where $V_{asd}$ in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gage [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of $\frac{3}{4}$ inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying bearing a label indicating compliance with ASTM D 1970 shall be permitted.
1507.7.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [\(V_{\text{asd}}\) greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's installation instructions. Fasteners are to be applied along the overlap not more than 36 inches (914 mm) on center.

Underlayment installed where \(V_{\text{asd}}\), in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226, Type II or ASTM D 4869, Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm).

Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gage [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of \(3/4\) inch (19.1 mm) into the roof sheathing.

**Exception:** As an alternative, adhered underlayment complying bearing a label indicating compliance with ASTM D 1970 shall be permitted.

1507.8.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [\(V_{\text{asd}}\) greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's installation instructions. Fasteners are to be applied along the overlap not more than 36 inches (914 mm) on center.

Underlayment installed where \(V_{\text{asd}}\), in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226, Type II or ASTM D 4869, Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm).

Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gage [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of \(3/4\) inch (19.1 mm) into the roof sheathing.

**Exception:** As an alternative, adhered underlayment complying bearing a label indicating compliance with ASTM D 1970 shall be permitted.

1507.8.8 Flashing. At the juncture of the roof and vertical surfaces, flashing and counterflashing shall be provided in accordance with the manufacturer's installation instructions, and where of metal, shall be not less than 0.019-inch (0.48 mm) (No. 26 galvanized sheet gage) corrosion-resistant metal. The valley flashing shall extend at least 11 inches (279 mm) from the centerline each way and have a splash diverter rib not less than 1 inch (25 mm) high at the flow line formed as accordance with the manufacturer's installation instructions. Fasteners are to be applied along the overlap not more than 36 inches (914 mm) on center.

Underlayment installed where \(V_{\text{asd}}\), in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226, Type II or ASTM D 4869, Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm).

Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gage [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of \(3/4\) inch (19.1 mm) into the roof sheathing.

**Exception:** As an alternative, adhered underlayment complying bearing a label indicating compliance with ASTM D 1970 shall be permitted.

1507.9.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [\(V_{\text{asd}}\) greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's installation instructions. Fasteners are to be applied along the overlap not more than 36 inches (914 mm) on center.

Underlayment installed where \(V_{\text{asd}}\), in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226, Type II or ASTM D 4869, Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm).

Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gage [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of \(3/4\) inch (19.1 mm) into the roof sheathing.

**Exception:** As an alternative, adhered underlayment complying bearing a label indicating compliance with ASTM D 1970 shall be permitted.

1507.9.9 Flashing. At the juncture of the roof and vertical surfaces, flashing and counterflashing shall be provided in accordance with the manufacturer's installation instructions, and where of metal, shall be not less than 0.019-inch (0.48 mm) (No. 26 galvanized sheet gage) corrosion-resistant metal. The valley flashing shall extend at least 11 inches (279 mm)
from the centerline each way and have a splash diverter rib not less than 1 inch (25 mm) high at the flow line formed as part of the flashing. Sections of flashing shall have an end lap of not less than 4 inches (102 mm). For roof slopes of three units vertical in 12 units horizontal (25-percent slope) and over, the valley flashing shall have a 36-inch-wide (914 mm) underlayment of either one layer of Type I underlayment running the full length of the valley or a self-adhering polymer-modified bitumen sheet complying bearing a label indicating compliance with ASTM D 1970, in addition to other required underlayment. In areas where the average daily temperature in January is 25°F (-4°C) or less or where there is a possibility of ice forming along the eaves causing a backup of water, the metal valley flashing underlayment shall be solidly cemented to the roofing underlayment for slopes under seven units vertical in 12 units horizontal (58-percent slope) or self-adhering polymer-modified bitumen sheet shall be installed.

1507.17.4.1 High wind attachment. Underlayment applied in areas subject to high winds \[ V_{ASD} \] greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's instructions. Fasteners shall be applied along the overlap at not more than 36 inches (914 mm) on center. Underlayment installed where \( V_{ASD} \) is not less than 120 mph (54 m/s) shall comply with ASTM D 226, Type II, ASTM D 4869, Type IV or ASTM D 6757. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with Section 1507.2.8 except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of not less than 32-gage \[ 0.0134 \text{ inch (0.34 mm)} \] sheet metal. The cap nail shank shall be a minimum of 12 gage \[ 0.105 \text{ inch (2.67 mm)} \] with a length to penetrate through the roof sheathing or a minimum of \( 3/4 \) inch (19.1 mm) into the roof sheathing.

**Exception:** As an alternative, adhered underlayment complying bearing a label indicating compliance with ASTM D 1970 shall be permitted.

**Reason:** Roofing underlayment are a critical component and provide protection to the roof deck and other components during installation as well as replacements and after storm events. The proposal adds a requirement that self-adhering underlayment bear a label to demonstrate compliance to the code. The labeling requirement for underlayment was part of a comprehensive proposal for the IRC by IBHS in the past cycle; it is hoped that a similar proposal will be approved for the IBC this year. The proposal also adds photovoltaic shingles to the list of roof covering materials requiring labeled underlayment materials in the IRC.

**Cost Impact:** Will increase the cost of construction
Will require products bear a label, which will add product approval costs. The IRC already contains an underlayment labeling requirement, so the cost impact is expected to be minimal.

---

**Public Hearing Results**

**Part I**

**Committee Action:** Approved as Submitted

**Committee Reason:** This code change levels the playing field for underlayment by clarifying the requirement for labeling.

**Assembly Action:** None
Proposed Change as Submitted

Proponent: Mike Fischer, Kellen, representing Asphalt Roofing Manufacturers Association (mfischer@kellencompany.com)

2015 International Residential Code

Revise as follows:

R905.2.6 Attachment. Asphalt shingles shall have the minimum number of fasteners required by the manufacturer’s approved installation instructions, but not less than four fasteners per strip shingle or two fasteners per individual shingle. Where the roof slope exceeds 21 units vertical in 12 units horizontal (21:12, 175-percent slope), shingles shall be installed as required by in accordance with the manufacturer's approved installation instructions.

Reason: The proposal removes ASTM D 225 (withdrawn by ASTM) and makes editorial changes for the use of manufacturer's installation instructions. Instructions are required as part of labeling requirements; it is important that the instructions that are part of the labeling for ASTM D 7158 and D 3161 are in use in the field. The proposal also refers the user of the code to the manufacturers instructions for ultra-steep slope applications.

Cost Impact: Will not increase the cost of construction
The proposal is editorial and adds no new requirements.

2016 ICC PUBLIC COMMENT AGENDA
Page 2185
Proposed Change as Submitted

Proponent: Mike Fischer, Kellen, representing Asphalt Roofing Manufacturers Association
(mfischer@kellencompany.com)

2015 International Building Code
Revise as follows:

1507.2.5 Asphalt shingles. Asphalt shingles shall comply with ASTM D 225 or ASTM D 3462.

1507.2.7 Attachment. Asphalt shingles shall have the minimum number of fasteners required by the manufacturer's approved installation instructions, but not less than four fasteners per strip shingle or two fasteners per individual shingle. Where the roof slope exceeds 21 units vertical in 12 units horizontal (21:12), shingles shall be installed as required by the manufacturer's approved installation instructions.

Reason: The proposal removes ASTM D 225 (withdrawn by ASTM) and makes editorial changes for the use of manufacturer's installation instructions. Instructions are required as part of labeling requirements; it is important that the instructions that are part of the labeling for ASTM D 7158 and D 3161 are in use in the field. The proposal also refers the user of the code to the manufacturer's instructions for ultra-steep slope applications.

Cost Impact: Will not increase the cost of construction
The proposal is editorial and adds no new requirements.

Public Hearing Results

Part I
Committee Action: Disapproved
Committee Reason: Consistency with the committee's disapproval of S26-16.
Assembly Action: None
Proposed Change as Submitted

**Proponent**: Mike Fischer, Kellen, representing Asphalt Roofing Manufacturers Association (mfischer@kellencompany.com)

**2015 International Residential Code**

**Revise as follows:**

R905.2.8 Flashing. Flashing for asphalt shingles shall comply with this section and the asphalt shingle manufacturer's approved installation instructions.

**Reason**: The proposal makes an editorial change to recognize the use of approved installation requirements, which may be digital and not printed. Installation instructions are part of the product labeling for the wind standards, it is important that the manufacturer's instructions for flashing also be followed to ensure installation consistent with the tested products and assemblies.

**Cost Impact**: Will not increase the cost of construction
The proposal is editorial.

---

**Public Hearing Results**

**Part II**

**Committee Action**: Approved as Submitted

**Committee Reason**: The committee approved this proposal based on the proponents published reason statement and previous action on S29-16, Part II.

**Assembly Action**: None

---

**Individual Consideration Agenda**

**Proponent**: Rebecca Baker, representing Jefferson County, CO / Colorado Chapter of the International Code Council (bbaker@co.jefferson.co.us) requests Disapprove.

**Commenter's Reason**: This proposal adds compliance with the asphalt shingle 'manufacturer's approved installation instructions' for flashing, and S29-16 adds compliance with the 'manufacturer's approved installation instructions' for the number of fasteners and for the installation on roofs exceeding 21:12. However, there is no general or other specific requirements for installation in accordance with the manufacturer's instructions in R905.2. The hit and miss of the will add confusion. This was disapproved for the IBC and for consistency, this needs to be disapproved.

**Proponent**: Theresa Weston, representing DuPont Building Innovations (theresa.a.weston@dupont.com) requests Disapprove.

**Commenter's Reason**: This proposal would require "manufacturer's approved installation instructions". As approved is defined in the IBC as "acceptable to the building official.", this could then require manufacturer's to submit their installation instructions to individual building code officials for approval. This would add significant time and cost to the building process and would create a large burden on both manufacturers and building code officials.

---

S34-16 Part II
S34-16 Part I

IBC: 1507.2.9.

**Proposed Change as Submitted**

**Proponent**: Mike Fischer, Kellen, representing Asphalt Roofing Manufacturers Association
(mfischer@kellencompany.com)

**2015 International Building Code**

Revise as follows:

1507.2.9 Flashings. Flashing for asphalt shingles shall comply with this section. Flashing shall be applied in accordance with this section and the asphalt shingle manufacturer’s printed approved installation instructions.

**Reason**: The proposal makes an editorial change to recognize the use of approved installation requirements, which may be digital and not printed. Installation instructions are part of the product labeling for the wind standards, it is important that the manufacturer’s instructions for flashing also be followed to ensure installation consistent with the tested products and assemblies.

**Cost Impact**: Will not increase the cost of construction

The proposal is editorial.

---

**Public Hearing Results**

Part I

**Committee Action**: Disapproved

**Committee Reason**: Consistency with actions on S26-16 and S29-16, Part I. The proponent will be working on defining terminology in the public comment phase.

**Assembly Action**: None
Proposed Change as Submitted

Proponent: David Roodvoets, DLR Consultants, representing Cedar Shake & Shingle Bureau (davelee@ix.netcom.com)

2015 International Building Code

Revise as follows:

1507.8 Wood shingles. The installation of wood shingles shall comply with the provisions of this section and Table 1507.8 and the wood shingle manufacturer’s instructions.

### TABLE 1507.8

WOOD SHINGLE AND SHAKE INSTALLATION

<table>
<thead>
<tr>
<th>ROOF ITEM</th>
<th>WOOD SHINGLES</th>
<th>WOOD SHAKES</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Roof slope</td>
<td>Wood shingles shall be installed on slopes of not less than three units vertical in 12 units horizontal (3:12).</td>
<td>Wood shingles shall be installed on slopes of not less than four units vertical in 12 units horizontal (4:12).</td>
</tr>
<tr>
<td>2. Deck requirement</td>
<td>Shingles shall be applied to roofs with solid or spaced sheathing. Where spaced sheathing is used, sheathing boards shall not less than 1” × 4” nominal dimensions and shall be spaced on centers equal to the weather exposure to coincide with the placement of fasteners.</td>
<td>Shakes shall be applied to roofs with solid or spaced sheathing. Where spaced sheathing is used, sheathing boards shall not less than 1” × 4” nominal dimensions and shall be spaced on centers equal to the weather exposure to coincide with the placement of fasteners. When 1” × 4” spaced sheathing is installed at 10 inches, boards must be installed between the sheathing boards.</td>
</tr>
<tr>
<td>Temperate climate</td>
<td>Solid sheathing is required.</td>
<td>Solid sheathing is required.</td>
</tr>
<tr>
<td>In areas where the average daily temperature in January is 25°F or less or where there is a possibility of ice forming along the eaves causing a backup of water.</td>
<td>Underlayment shall comply with ASTM D 226, Type I or ASTM D 4869 Type IV.</td>
<td>Underlayment shall comply with ASTM D 226, Type I or ASTM D 4869 Type IV.</td>
</tr>
<tr>
<td>4. Underlayment</td>
<td>Underlayment shall comply with ASTM D 226, Type I or ASTM D 4869 Type IV.</td>
<td>Underlayment shall comply with ASTM D 226, Type I or ASTM D 4869 Type IV.</td>
</tr>
<tr>
<td>Temperate climate</td>
<td>An ice barrier that consists of at least two layers of underlayment cemented together or of a self-adhering polymer-modified bitumen sheet shall extend from the eave’s edge to a point at least 24 inches inside the exterior wall line of the building.</td>
<td>An ice barrier that consists of at least two layers of underlayment cemented together or of a self-adhering polymer-modified bitumen sheet shall extend from the lowest edges of all roof surfaces eave’s edge to a point at least 24 inches inside the exterior wall line of the building.</td>
</tr>
<tr>
<td>In areas where there is a possibility of ice forming along the eaves causing a backup of water.</td>
<td>Fasteners for wood shingles shall be hot-dipped galvanized or Type 304 (Type 316 for coastal areas) stainless steel. Type 304 or 316 or hot-dipped galvanized weight of ASTM A 153 Class D (1 oz. ft²) with a minimum penetration of 0.75 inch into the sheathing. For sheathing less than 0.60.75 inch thick, the fasteners shall extend through the sheathing.</td>
<td>Fasteners for wood shakes shall be hot-dipped galvanized or Type 304 (Type 316 for coastal areas) stainless steel. Type 304 or 316 or hot-dipped galvanized weight of ASTM A 153 Class D (1 oz. ft²) with a minimum penetration of 0.75 inch into the sheathing. For sheathing less than 0.60.75 inch thick, the fasteners shall extend through the sheathing.</td>
</tr>
<tr>
<td>5. Application</td>
<td>Two per shingle.</td>
<td>Two per shake.</td>
</tr>
</tbody>
</table>
Weather exposures shall not exceed those set forth in Table 1507.8.7.

Weather exposures shall not exceed those set forth in Table 1507.9.8.

<table>
<thead>
<tr>
<th>Exposure</th>
<th>Method</th>
<th>Flushing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shingles shall be laid with a side lap of not less than 1.5 inches between joints in courses, and no two joints in any three adjacent courses shall be in direct alignment. Spacing between shingles shall be 0.25 to 0.375 inch.</td>
<td>In accordance with Section 1507.8.8.</td>
</tr>
<tr>
<td></td>
<td>Shakes shall be laid with a side lap of not less than 1.5 inches between joints in adjacent courses. Spacing between shakes shall not be less than 0.375 inch or more than 0.625 inch for shakes and taper sawn shakes of naturally durable wood and shall be 0.25 to 0.375 inch for preservative treated taper sawn shakes.</td>
<td>In accordance with Section 1507.9.9.</td>
</tr>
</tbody>
</table>

For SI: 1 inch = 25.4 mm, °C = [(°F) - 32]/1.8.

1507.8.3 Underlayment. Underlayment shall comply with ASTM D 226, Type I or ASTM D 4869 Type IV and shall be applied in accordance with the wood shingle manufacturer's instructions.

1507.8.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds \( V_{\text{asd}} \) greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1 shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's installation instructions. Fasteners are to be applied along the overlap not more than 36 inches (914 mm) on center.

Underlayment installed where \( V_{\text{asd}} \), in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226, Type II or ASTM D 4869, Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the wood shingle manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gage [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of \( \frac{3}{4} \) inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted where shingles are installed over a combination of securely fastened vertical spacers and horizontal nailers.

1507.8.6 Attachment. Fasteners for wood shingles shall be corrosion resistant stainless steel Type 304 or 316 or hot dipped galvanized weight of ASTM A 153 Class D (1 oz. ft²). Alternatively, two 16 gage stainless steel Type 304 or 316 staples with a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of \( \frac{3}{4} \) inch (19.1 mm) into the roof sheathing.

1507.9 Wood shakes. The installation of wood shakes shall comply with the provisions of this section, the wood shake manufacturer’s instructions and Table 1507.8.

1507.9.3 Underlayment. Underlayment shall comply with ASTM D 226, Type I or ASTM D 4869 Type IV.

1507.9.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds \( V_{\text{asd}} \) greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1 shall be applied with corrosion-resistant fasteners in accordance with the manufacturer’s installation instructions. Fasteners are to be applied along the overlap not more than 36 inches (914 mm) on center.

Underlayment installed where \( V_{\text{asd}} \), in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226, Type II or ASTM D 4869, Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the wood shake manufacturer’s installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gage [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of \( \frac{3}{4} \) inch (19.1 mm) into the roof sheathing.

Exception: As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted where shakes are installed over a combination of securely fastened vertical spacers and horizontal nailers.

1507.9.5 Interlayment. Interlayment shall comply with ASTM D 226, Type I or ASTM D 4869 Type IV and shall be applied in accordance with the wood shake manufacturer's instructions.
1507.9.7 Attachment. Fasteners for wood shakes shall be corrosion-resistant stainless steel Type 304 or 316 or hot-dipped galvanized weight of ASTM 153 Class D (1 oz ft²). Alternatively, two 16 gage stainless steel Type 304 or 316 staples with crown widths 7/16 inch (11.1 mm) minimum, 3/4 inch (19.1 mm) maximum, shall be installed. Fasteners installed within 15 miles (24 km) of salt water coastal areas shall be stainless steel Type 316. Fasteners for pressure impregnated fire-retardant treated shakes or pressure impregnated-preservative-treated shakes of naturally durable wood in accordance with AWPA U1, shall be stainless steel Type 316 with a minimum penetration of 3/4 inch (19.1 mm) into the sheathing. For sheathing less than 3/4 inch (19.1 mm) in thickness, the fasteners shall extend through the sheathing. Each shake shall be attached with a minimum of two fasteners.

1507.9.8 Application. Wood shakes shall be laid with a side lap not less than 1 1/2 inches (38 mm) between joints in adjacent courses. Spacing between shakes in the same course shall be 3/8 to 5/8 inch (9.5 to 15.9 mm) for shakes and tapersawn shakes of naturally durable wood and shall be 1/4 to 3/8 inch (6.4 to 9.5 mm) for preservative tapersawn shakes. Weather exposure for wood shakes shall not exceed those set in Table 1507.9.8.

Reason: Changes to Section 1507.8, There are several additions requiring that the shingles or underlayment be installed in accordance with the shingle or shake manufacturer's instructions. This is to clarify that the shingle or shake manufacturer's instructions and the written code should prevail if there are issues with the required product's installation. There are other clarifications that make the language in the IBC the same as in the IRC.

They will have no cost impact.

In 1507.8.3.1: The specific underlayment is better defined than in the current code. The heavier weight felts are the standard practice defined in the manufacturer's installation manual and provide better moisture resistance and durability.

In the Exception's: The additional requirement for spaced sheathing is based on field experience where moisture from dew and rain accumulates on the surface of the ASTM D 1970 type products and then is driven into the shingle when the roof cycles from cold to hot. Wood when wet on one side and dry on the opposite side will result in the shingle or shake cupping. Moisture barrier products, such as those complying with ASTM D1970 and similar products retain moisture on their surface, which can be driven into the wood creating cupping. This phenomenon has resulted in premature failure of the shingles and corrosion of the fasteners. Spacing the shingles off the non permeable underlayment creates a place for air movement, usually the chimney effect, which drives the moisture out of the system. The roof deck is always cooler than when the shingles are installed directly on the deck. It also provides additional cooling savings, and the reduced possibility of ice dams.

Image: Fasteners -Asphalt Non permeable underlayment
This photo shows premature failure of the fasteners and the wood shakes by the constant moisture cycling when non permeable underlayment is used. On some non permeable underlayments moisture accumulates and then is moved in and out of the wood resulting in curling and warping of the shingle or shake. Spacing the shingle or shake away from the non permeable underlayment allows air movement beneath the shingle or shake. Shingles and shakes have been applied directly over traditional asphalt based roofing felts without observing this phenomenon.

Table 1507.8 captures the changes noted above.

1507.8.6 and 1507.9.7 Attachment: There are many known cases of wood shakes and shingles falling or blowing off roofs due to the use of inferior fasteners. Specifying "corrosion resistant" is no longer sufficient; the type of fastener to be used is determined by various environmental factors and product types. Specific requirements will improve roof system integrity and lifespan. The code currently requires more corrosion resistant fasteners in several applications. Hot dipped galvanized remains as the base requirement, but for locations near salt water and whenever treated shingles are used stainless steel Type 316 is the only fastener material found that resist these seriously corrosive environments. Corrosion of fasteners has been found relatively far inland, especially Hawaii. The 15 mile requirement reduces the possibility of fastener corrosion. The requirement is supported by the Stainless Steel Institute's recommendations. This change was accepted in the IRC 2015 code. Image (2) (3) (4) These images show the corrosion that is seen frequently in coastal areas and interior areas and when fire retardant or preservative treated shingles or shakes are installed.
1507.9.8  There is no need to have different dimensional requirements for tapersawn shakes.

Bibliography: Maze Nails web site.
Committee Action: Disapproved

Assembly Action: None

Cost Impact: Will increase the cost of construction
Section 1507.8
There will be no cost impact for requiring the installer to follow cedar shingle or shake manufacturer's instructions. Other clarifications have no cost impact.

Sections 1507.8.3.1
The cost impact of changing from an ASTM D 226 Type I underlayment to an ASTM D 4869 Type IV underlayment would be about $218 for a 25 square roof or less than $0.09 per square foot. This is based on Home Depot prices of $16.98 for a 432 square foot roll of Type I felt and a price of $26.69 for a 216 square foot roll of ASTM D 4869 Type IV felt.

The cost impact of adding the spacers and nailers is likely to be about $1000 for a 25 square roof, or $0.40 per square foot. This is based on 1 x 2 spacers at 1.07 per 8 foot and 1 x 4 nailers at $2.06 per 8 foot, nails and labor at $250, prices at Home Depot 12/26/2014.

Sections 1507.8.6 and 1507.9.7
The cost impact from changing from "typical corrosion resistant" nails to hot dipped galvanized fasteners meeting ASTM A 153 Class D is about $20 for a 25 square roof. Improving to a stainless steel Type 316 will cost about $416. This is based on standard corrosion resistant nails at $138/50 pounds, hot dipped galvanized nails at $147 for 50 pounds and stainless steel Type 316 at $286 for 25 pounds. Prices are from Maze Nails 12/26/14.

1507.8.6 and 1507.9.7 There should be no cost impact incurred for the change from 1/2 inch to 3/4 inch.

1507.9.8
There is no cost impact for this change.

Committee Reason: The committee believes there needs to be a definition of coastal as it defines the areas affected by the proposed provisions. As written, 15 miles inland is very far and it will impact areas where there are no problems. Recognizing that corrosion is an issue, it is recommended that the proposal be reworked in the form of a public comment. It will need to reduce the area that would be covered by these provisions.

Assembly Action: None

Public Comment 1:

Proponent: David Roodvoets, Cedar Shake & Shingle Bureau, representing Cedar Shake & Shingle Bureau (davelee@ix.netcom.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

1507.8.6 Attachment. Fasteners for wood shingles shall be stainless steel Type 304 or 316 or hot dipped galvanized weight of ASTM A 153 Class D (1 oz. ft²). Alternatively, two 16 gage stainless steel Type 304 or 316 staples with crown widths 7/16 inch (11.1 mm) minimum 3/4 inch (19.1 mm) maximum, shall be used. Fasteners installed within 15-10 miles (24 km) of salt water coastal areas shall be stainless steel Type 316. Fasteners for pressure impregnated fire-retardant treated shingles or pressure impregnated preservative treated shingles of naturally durable wood in accordance with AWPA U1, shall be stainless steel Type 316 with a minimum penetration of 3/4 inch (19.1 mm) into the sheathing. For sheathing less than 3/4 inch (19.1 mm) in thickness, the fasteners shall extend through the sheathing. Each shingle shall be attached with a
minimum of two fasteners.

1507.9.7 Attachment. Fasteners for wood shakes shall be stainless steel Type 304 or 316 or hot dipped galvanized weight of ASTM 153 Class D (1 oz ft²). Alternatively, two 16 gauge stainless steel Type 304 or 316 staples with crown widths 7/16 inch (11.1 mm) minimum, 3/4 inch (19.1 mm) maximum, shall be installed. Fasteners installed within 15-10 miles (24 km 16 km) of salt water coastal areas, sea coasts shall be stainless steel Type 316. Fasteners for pressure impregnated fire-retardant treated shakes or pressure impregnated-preservative-treated shakes of naturally durable wood in accordance with AWPA U1, shall be stainless steel Type 316 with a minimum penetration of 3/4 inch (19.1 mm) into the sheathing. For sheathing less than 3/4 inch (19.1 mm) in thickness, the fasteners shall extend through the sheathing. Each shake shall be attached with a minimum of two fasteners.

Commenter's Reason:
In response to the Structural committee comments the distance from sea coast requiring Stainless Steel Type 316 has been reduced to 10 miles in accordance with the Speciality Steel Industry recommendations and Cedar Shake and Shingle Bureau experience.

From Speciality Steel Handbook: Sea Spray and dry salt particles carried by wind may cause pitting and rusting of stainless steel, unless a sufficiently corrosion resistant grade is chosen. The distance airborne salt is carried can vary significantly with local wind patterns. Generally, locations within five to ten miles (9 to 18 km) of salt water are considered at risk for chloride-related corrosion, but local weather patterns and the performance of metal near the site should be evaluated prior to material selection. In some locations, marine salt accumulation are only a factor within the first 0.9 miles or 1.5 km from the shore. In other areas salt deposits have been measured 27 miles (50 km) or more inland.

Considering the above 10 miles appears to be a reasonable compromise.

# Proposed Change as Submitted

**Proponent**: Mike Fischer, Kellen, representing Asphalt Roofing Manufacturers Association (mfischer@kellencompany.com)

**2015 International Residential Code**

**Revise as follows:**

**R905.9 Built-up roofs.** The installation of built-up roofs shall comply with the provisions of this section and the manufacturer's approved installation instructions.

**Reason:** The proposal adds a reference to manufacturers instructions to add clarity.

**Cost Impact:** Will not increase the cost of construction

The proposal does not add additional requirements to the code.

## Public Hearing Results

### Part II

<table>
<thead>
<tr>
<th>Committee Action:</th>
<th>Approved as Submitted</th>
</tr>
</thead>
<tbody>
<tr>
<td>Committee Reason:</td>
<td>The committee approved this proposal based on the proponents published reason statement and prior action on S29-16, Part II.</td>
</tr>
<tr>
<td>Assembly Action:</td>
<td>None</td>
</tr>
</tbody>
</table>

### Individual Consideration Agenda

**Proponent**: Rebecca Baker, representing Jefferson County, CO / Colorado Chapter of the International Code Council (bbaker@co.jefferson.co.us) requests Disapprove.

**Commenter's Reason:** The added language is redundant. Section R903.1 states that all roof assemblies shall be designed and installed in accordance with the approved manufacturer's instructions. The first sentence in Section R905.1 requires that all roof coverings shall be applied in accordance with the manufacturer's installation instructions. Manufacturer's instructions are also in Section R905.9.3 which requires that built-up roofs be installed in accordance with the manufacturer's instructions. There is no need for another mention of manufacturer's instructions. This was disapproved for the IBC and to maintain consistency, this also needs to be disapproved.

---

S41-16 Part II

---

2016 ICC PUBLIC COMMENT AGENDA
Committee Action: Disapproved
Assembly Action: None

S41-16 Part I
IBC: 1507.10.

Proposed Change as Submitted

Proponent: Mike Fischer, Kellen, representing Asphalt Roofing Manufacturers Association
(mfischer@kellencompany.com)

2015 International Building Code

Revise as follows:

1507.10 Built-up roofs. The installation of built-up roofs shall comply with the provisions of this section and the manufacturer’s approved installation instructions.

Reason: The proposal adds a reference to manufacturers instructions to add clarity.

Cost Impact: Will not increase the cost of construction

The proposal does not add additional requirements to the code.

Public Hearing Results

Part I

Committee Action: Disapproved

Committee Reason: The proponent recognizes the need to further work on the proposal through the public comment phase, particularly on use of “approved”. Disapproval is consistent with prior actions on this series of proposals.

Assembly Action: None
Proposed Change as Submitted

Proponent: Mike Fischer, Kellen, representing Asphalt Roofing Manufacturers Association (mfischer@kellencompany.com)

2015 International Residential Code

Revise as follows:

R905.11 Modified bitumen roofing. The installation of modified bitumen roofing shall comply with the provisions of this section and the manufacturer's approved installation instructions.

Reason: The proposal is editorial, adds a reference to manufacturer's instructions.

Cost Impact: Will not increase the cost of construction

the proposal adds no additional requirements.

S42-16 Part II:
R905.11-
FISCHER13607

Public Hearing Results

Part II

Committee Action: Approved as Submitted

Committee Reason: The committee approved this proposal based on the proponents published reason statement and prior action on S29-16, Part II.

Assembly Action: None

Individual Consideration Agenda

Proponent: Rebecca Baker, representing Jefferson County, Colorado (bbaker@co.jefferson.co.us) requests Disapprove.

Commenter's Reason: The added language is redundant. Section R903.1 states that all roof assemblies shall be designed and installed in accordance with the approved manufacturer's instructions. The first sentence in Section R905.1 requires that all roof coverings shall be applied in accordance with the manufacturer's installation instructions. Manufacturer's instructions are also in Section R905.11.3 which require modified bitumen roofs to be installed in accordance with the manufacturer's instructions. This was disapproved for the IBC and to maintain consistency, this needs to be disapproved.
Proposed Change as Submitted

Proponent: Mike Fischer, Kellen, representing Asphalt Roofing Manufacturers Association
(mfischer@kellencompany.com)

2015 International Building Code

Revise as follows:

1507.11 Modified bitumen roofing. The installation of modified bitumen roofing shall comply with the provisions of this section and the manufacturer's approved installation instructions.

Reason: The proposal is editorial, adds a reference to manufacturer's instructions.

Cost Impact: Will not increase the cost of construction
the proposal adds no additional requirements.

Public Hearing Results

Part I

Committee Action: Disapproved

Committee Reason: Same as S42-16, Part I.

Assembly Action: None
Proposed Change as Submitted

Proponent: James Kirby, representing Roof Coating Manufacturers Association, representing Center for Environmental Innovation in Roofing (jkirby@kellencompany.com)

2015 International Building Code

Add new text as follows:

1507.15.3 Application Liquid-applied roofing shall be installed in accordance with this chapter and the manufacturer's approved installation instructions.

Reason: This proposal adds text to the IBC that is already in the IRC. The 2018 IBC and IRC should have the same language regarding application of liquid-applied roofing.

Cost Impact: Will not increase the cost of construction

The proposal adds clarity and does not change code requirements.

Public Hearing Results

Committee Action: Disapproved

Committee Reason: Consistency with actions on other proposals that dealt with manufacturers installation instructions. There is some question whether the proposed text is necessary and the reason really does not indicate why.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: James Kirby, Kellen, representing Roof Coating Manufacturers Association (jameskirby47@icloud.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

1507.15.3 Application Liquid-applied roofing shall be installed in accordance with this chapter and the manufacturer's approved installation instructions.

Commenter's Reason: In April, the Structural Committee disapproved this code change because of confusion with the word "approved". This public comment removes the controversial word and brings a needed section about Application to the Liquid-applied roofing section in Chapter 15 of the IBC. This proposal adds text to the IBC that is already in the IRC. The 2018 IBC and IRC should have the same language regarding application of liquid-applied roofing.
Proposed Change as Submitted

Proponent: Jennifer Goupil, American Society of Civil Engineers, representing SELF (jgoupil@asce.org)

2015 International Building Code

Revise as follows:

1602.1 NOTATIONS.

\( D \) = Dead load.

\( D_i \) = Weight of ice in accordance with Chapter 10 of ASCE 7.

\( E \) = Combined effect of horizontal and vertical earthquake induced forces as defined in Section 12.4.2 2.3.6 of ASCE 7.

\( F \) = Load due to fluids with well-defined pressures and maximum heights.

\( F_a \) = Flood load in accordance with Chapter 5 of ASCE 7.

\( H \) = Load due to lateral earth pressures, ground water pressure or pressure of bulk materials.

\( L \) = Roof live load greater than 20 psf (0.96 kN/m\(^2\)) and floor live load.

\( L_r \) = Roof live load of 20 psf (0.96 kN/m\(^2\)) or less.

\( R \) = Rain load.

\( S \) = Snow load.

\( T \) = Self-straining load forces and effects, Cummulative effect of self-straining load forces and effects.

\( V_{asd} \) = Nominal design wind speed (3-second gust), miles per hour (mph) (km/hr) where applicable.

\( V_{ult} \) = Ultimate design wind speeds (3-second gust), miles per hour (mph) (km/hr) determined from Figure 1609.3(1), 1609.3(2), 1609.3(3) or ASCE 7.

\( W \) = Load due to wind pressure.

\( W_i \) = Wind-on-ice in accordance with Chapter 10 of ASCE 7.

Reason: This change proposes to coordinate the Notation in Chapter 16 of the IBC with the 2016 edition of the referenced loading standard Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE/SEI 7-16).

Cost Impact: Will not increase the cost of construction

The proposed changes will not increase the cost of construction. This proposal coordinates the IBC with the referenced loading standard ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE 7 will be updated from the 2010 edition to the 2016 edition as an Administrative Update to the 2018 I-Codes.
As of the submission date of this code change proposal, the ASCE 7 Standards Committee has completed the committee balloting on technical changes. The document designated ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures is expected to be completed, published and available for purchase prior to the ICC Public Comment Hearings for Group B in October of 2016. Any person interested in obtaining a public comment copy of ASCE 7-16 may do so by contacting James Neckel at ASCE (jneckel “at” asce.org).

**Public Hearing Results**

Committee Action: Approved as Submitted

Committee Reason: This proposal coordinates IBC notation listed in Chapter 16 with the latest edition of the referenced load standard, ASCE 7, updated in ADM94-16.

Assembly Action: None

**Individual Consideration Agenda**

Proponent: Ed Berkel, ICC Code Correlation Committee, representing ICC Code Correlation Committee (ccc@icc.org) requests Disapprove.

Commenter's Reason: The Code Correlation Committee requests Disapproval of this code change proposal in order to bring a correlation issue to the attention of the full membership at the Public Comment Hearings and to allow the membership to coordinate action on this code change proposal with action taken on Code Change Proposal ADM94-16.

ADM94-16 is the administrative update to referenced standards in the I-Codes. One of these standards, ASCE 7, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, was proposed for update to ASCE7-16. However, a successful assembly motion requests that the referenced ASCE7 remain at ASCE7-10 as it presently is referenced in the 2015 I-Codes. This code change proposal coordinates with and relies upon reference to ASCE7-16.

The Code Correlation Committee is a standing committee of the International Code Council whose objectives, procedures and organization are set forth in Council Policy CP#44-13. The objective of the Code Correlation Committee is to maintain technical and editorial consistency among the International Codes and to assist staff in the evaluation and processing of code change proposals and comments that are exclusively editorial.
S55-16

Proposed Change as Submitted

Proponent: Edwin Huston, representing National Council of Structural Engineers' Associations (NCSEA)
(huston@smithhustoninc.com)

2015 International Building Code
Revise as follows:

1602.1 Definitions. The following terms are defined in Chapter 2:

L_f = Fire wall horizontal live load.

1605.2 Load combinations using strength design or load and resistance factor design. Where strength design or load and resistance factor design is used, buildings and other structures, and portions thereof, shall be designed to resist the most critical effects resulting from the following combinations of factored loads:

1.4(D + F)  
(Equation 16-1)

1.2(D + F) + 1.6(L + H) + 0.5(L_f or S or R)  
(Equation 16-2)

1.2(D + F) + 1.6(L_f or S or R) + 1.6H + (f_1L or 0.5 W)  
(Equation 16-3)

1.2(D + F) + 1.0W + f_1L + 1.6H + 0.5(L_f or S or R)  
(Equation 16-4)

1.2(D + F) + 1.0E + f_1L + 1.6H + f_2S  
(Equation 16-5)

0.9D + 1.0W + 1.6H  
(Equation 16-6)

0.9(D + F) + 1.0E + 1.6H  
(Equation 16-7)

0.9D + 1.6L_f + 1.6H  
(Equation 16-8)

where:

f_1 = 1 for places of public assembly live loads in excess of 100 pounds per square foot (4.79 kN/m^2), and parking garages; and 0.5 for other live loads.

f_2 = 0.7 for roof configurations (such as saw tooth) that do not shed snow off the structure, and 0.2 for other roof configurations.

Exceptions:
1. Where other factored load combinations are specifically required by other provisions of this code, such combinations shall take precedence.
2. Where the effect of H resists the primary variable load effect, a load factor of 0.9 shall be included with H where H is permanent and H shall be set to zero for all other conditions.

1605.3 Load combinations using allowable stress design.

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

D + F  
(Equation 16-8)

D + H + F + L  
(Equation 16-9)

D + H + F + (L_f or S or R)  
(Equation 16-10)

D + H + F + 0.75(L) + 0.75(L_f or S or R)  
(Equation 16-11)

D + H + F + (0.6 W or 0.7 E)  
(Equation 16-12)

D + H + F + 0.75(0.6 W) + 0.75L + 0.75(L_f or S or R)  
(Equation 16-13)

D + H + F + 0.75(0.7 E) + 0.75L + 0.75S  
(Equation 16-14)

0.6D + 0.6W + H  
(Equation 16-15)

0.6(D + F) + 0.7E + H  
(Equation 16-16)

0.6D + L_f + H  
(Equation 16-18)
Exceptions:
1. Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m²) or less and roof live loads of 30 psf (1.44 kN/m²) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m²), 20 percent shall be combined with seismic loads.
3. Where the effect of $H$ resists the primary variable load effect, a load factor of 0.6 shall be included with $H$ where $H$ is permanent and $H$ shall be set to zero for all other conditions.
4. In Equation 16-15, the wind load, $W$, is permitted to be reduced in accordance with Exception 2 of Section 2.4.1 of ASCE 7.
5. In Equation 16-16, $0.6D$ is permitted to be increased to $0.9D$ for the design of special reinforced masonry shear walls complying with Chapter 21.

Add new text as follows:

1607.14.2 Fire walls. In order to meet the structural stability requirements of section 706.2 where the structure on either side of the wall has collapsed, fire walls and their supports shall be designed to withstand a minimum horizontal load, $L_f$, of 5 psf (0.240 kN/m²).

Reason: This code change clarifies the minimum lateral loading that fire walls are required to resist to meet the structural stability requirements of section 706.2 where the structure on either side of the wall has collapsed and can no longer provide support. This is the same lateral load that is required for fire walls designed per NFPA 221 as allowed by 706.2. Currently, however, there is no horizontal fire wall load criteria for those who are not using the "deemed to comply" NFPA option. A new definition $L_f$ and additional load combinations are added to clarify how to combine horizontal fire loads with other loads. Unlike other live loads, it is important that horizontal loads for cantilevered fire walls be combined with the reduced dead loads of equations 16-9 and 16-20. Like the other load combinations, these combinations need to be maintained in IBC and not referenced to another standard.

Cost Impact: Will not increase the cost of construction
The cost of construction will not increase by clarifying the lateral load on firewalls, since it is already standard practice to use this loading per NFPA 221.
This clarification will decrease the cost of design, as it provides structural engineers a clearer understanding of code intent.

Public Hearing Results

Committee Action: Approved as Modified

Modification:

2015 International Building Code

1602.1 Definitions. The following terms are defined in Chapter 2:

$L_f$ = Fire wall horizontal live load.

1605.2 Load combinations using strength design or load and resistance factor design. Where strength design or load and resistance factor design is used, buildings and other structures, and portions thereof, shall be designed to resist the most critical effects resulting from the following combinations of factored loads:

1. $1.4(D + F)$ (Equation 16-1)
2. $1.2(D + F) + 1.6(L + H) + 0.5(L_f or S or R)$ (Equation 16-2)
3. $1.2(D + F) + 1.6(L_f or S or R) + 1.6H + (f_1L or 0.5W)$ (Equation 16-3)
4. $1.2(D + F) + 1.0W + f_1L + 1.6H + 0.5(L_f or S or R)$ (Equation 16-4)
5. $1.2(D + F) + 1.0E + f_1L + 1.6H + f_2S$ (Equation 16-5)
where:

- \( f_1 = 1 \) for places of public assembly live loads in excess of 100 pounds per square foot (4.79 kN/m²), and parking garages; and 0.5 for other live loads.
- \( f_2 = 0.7 \) for roof configurations (such as saw tooth) that do not shed snow off the structure, and 0.2 for other roof configurations.

**Exceptions:**
1. Where other factored load combinations are specifically required by other provisions of this code, such combinations shall take precedence.
2. Where the effect of \( H \) resists the primary variable load effect, a load factor of 0.9 shall be included with \( H \) where \( H \) is permanent and \( H \) shall be set to zero for all other conditions.

### 1605.3.1 Basic load combinations.

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

- \( D + F \) (Equation 16-8)
- \( D + H + F + L \) (Equation 16-9)
- \( D + H + F + (L_f \text{ or } S \text{ or } R) \) (Equation 16-10)
- \( D + H + F + 0.75(L) + 0.75(L_f \text{ or } S \text{ or } R) \) (Equation 16-11)
- \( D + H + F + (0.6W \text{ or } 0.7E) \) (Equation 16-12)
- \( D + H + F + 0.75(0.6W) + 0.75L + 0.75(L_f \text{ or } S \text{ or } R) \) (Equation 16-13)
- \( D + H + F + 0.75(0.7E) + 0.75L + 0.75S \) (Equation 16-14)
- \( 0.6D + 0.8W + H \) (Equation 16-15)
- \( 0.6(D + F) + 0.7E + H \) (Equation 16-16)
- \( 0.6D + LF + H \) (Equation 16-18)

**Exceptions:**
1. Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m²) or less and roof live loads of 30 psf (1.44 kN/m²) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m²), 20 percent shall be combined with seismic loads.
3. Where the effect of \( H \) resists the primary variable load effect, a load factor of 0.6 shall be included with \( H \) where \( H \) is permanent and \( H \) shall be set to zero for all other conditions.
4. In Equation 16-15, the wind load, \( W \), is permitted to be reduced in accordance with Exception 2 of Section 2.4.1 of ASCE 7.
5. In Equation 16-16, 0.6 \( D \) is permitted to be increased to 0.9 \( D \) for the design of special reinforced masonry shear walls complying with Chapter 21.

### 1607.14.2 Fire walls.

In order to meet the structural stability requirements of section 706.2 where the structure on either side of the wall has collapsed, fire walls and their supports shall be designed to withstand a minimum horizontal allowable stress load of 5 psf (0.240 kN/m²).

**Committee Reason:** This proposal will give designers and plan checkers guidance on how to comply with requirements for fire walls. The modification removes all portions of the original proposal except for new section 1607.14.2 where a clarification is made to indicate the load is an allowable stress design load.
Proponent: Ali Fattah, City of San Diego Development Services Department, representing City of San Diego Development Services Department (afattah@sandiego.gov) requests Approve as Modified by this Public Comment.

Further Modify as Follows:

2015 International Building Code

706.2 Structural stability. Fire walls

Fire walls shall be designed and constructed to allow collapse of the structure on either side without collapse of the wall under fire conditions. Fire walls designed and constructed in accordance with Section 706, or NFPA 221, as well as Section 1607.14.2 shall be deemed to comply with this section.

1607.14.2 Fire walls. In order to meet the structural stability requirements of section 706.2 where the structure on either side of the wall has collapsed, fire walls and their supports shall be designed to withstand a minimum horizontal, out-of-plane allowable stress load of 5 psf (0.240 kN/m²) or Strength Design load of 8 psf.

Commenter's Reason: This public comment is editorial and provides a pointer to the load requirements from the Fire Wall section. It makes clear that the load requirement applies whether the IBC or NFPA 221 are the basis for structural stability compliance. We were proponents of S101 and an associated public comment and believe that the code change approved as modified by the Structural Committee would not be complete without this pointer. S101 was submitted from the vantage point of loading on the wall during fire conditions and S55 was submitted from the vantage point of post fire wall stability during the reconstruction and partial occupancy phase of the building. Both scenarios are addressed in S55.

Further editorial clarifications were added based on a floor modification to S101 that address strength design loads since NFPA 221 includes proposed changes to show both. The committee intent is for this load not to be concurrent with other loads so the load combinations were deleted.

We opted to modify S55 rather than submit S101 as modified in an unsuccessful floor modification in light of approval of S55. The committee reason in S101 suggests the pointer from 706.2.
S61-16

IBC: 1603.1.5.

Proposed Change as Submitted

Proponent: James Bela, representing Oregon Earthquake Awareness (saskrake@gmail.com)

2015 International Building Code
Revise as follows:

1603.1.5 (Dsaf) Earthquake design data. The following information related to seismic loads shall be shown, regardless of
whether seismic loads govern the design of the lateral force-resisting system of the structure:

1. Risk category.
   1. Occupancy and Use categories.
   2. Seismic importance safety factor, I_{E}.

2. Mapped spectral response acceleration parameters, S_{x} and S_{y}.

3. Mapped lateral design force (base shear) coefficient

4. Site class.

5. Design spectral response acceleration parameters, S_{Dx} and S_{Dy}.

6. Design lateral force (base shear) coefficient.

7. Basic seismic force-resisting system(s).

8. Seismic category.

9. Basic design lateral force (base shear) coefficient.

10. Analysis procedure used.

Reason: Change in language/terminology to clarify the code.

Bibliography:
Cost Breakdown of Nonstructural Building Elements
Earthquake Engineering Rese3arch Center, U.C. Berkeley, 96 p.

Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake.
Eduardo Miranda, Gilberto Mosqueda, Rodrigo Retamales, and Gokhan Pekcan (2012) Performance of Nonstructural
doi:http://dx.doi.org/10.1193/1.4000032
http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032

Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Permalink: http://dx.doi.org/10.1061/(ASCE)CF.1943-5509.0000630
Read More: http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630

Homeowner’s Guide to Earthquake Safety

Retrofitting Questions and Answers
Earthquake Safety, Inc., 2015 (web based)
http://www.earthquakesafety.com/earthquake-retrofitting-faq.html

Cost and Seismic Design
https://www.google.com/#q=cost+and+seismic+design+christopher+arnold+pdf

Earthquake Architecture website
http://www.ittk.ac.in/nicee/wcee/article/14_05-06-0185.PDF

Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee
NIST GCR 14-917-26 2013, 249 p. NEHRP Consultants Joint Venture A partnership of the Applied Technology Council and the
Consortium of Universities for Research in Earthquake Engineering.
**Cost Impact:** Will not increase the cost of construction
NA, as is simply a change in language/terminology

**Public Hearing Results**

**Committee Action:** Disapproved

**Committee Reason:** The proposed terminology changes under earthquake design data would create a conflict with the referenced load standard, ASCE 7.

**Assembly Action:** None

**Individual Consideration Agenda**

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com) requests Approve as Submitted.

Commenter’s Reason: “If your definition is wrong, you’ll look for the wrong thing.”

“Anyone who conducts an argument by appealing to authority is not using his intelligence; he is just using his memory.”

- Carol Cleland

“Just because it comes from a consensus standard doesn’t mean it isn’t without problems.”

- Leonardo DaVinci

“Just because it comes from a consensus standard doesn’t mean it isn’t without problems.”

- Jay Crandell

The Committee Reason for Disapproval is not an argument, but simply a statement of the **Logical Fallacy - Authority:** “You said that because an authority [ASCE 7] thinks something, it must therefore be true.” The proposed terminology change comes from the *same level of understanding* as that underlying ASCE 7, and it simply is applying the empirical evidence of our recent experiences with global earthquakes - for a better, more logical and more consistent approach to earthquake engineering and seismic design.

This change in language/terminology helps to clarify the code.
"All sciences are vain and full of errors that are not born of Experience, the mother of all Knowledge."

- Leonardo DaVinci

This recently published article Reality Check: Seismic Hazard Models You Can Trust provides the comprehensive understanding (based on empirical evidence) of why these and other associated proposed changes are needed to provide a more reasonable and more practical approach to seismic safety for the general public that is exposed to major earthquake risk – namely because the “current probabilistic methods to quantify earthquake hazards have serious problems.”

https://eos.org/opinions/reality-check-seismic-hazard-models-you-can-trust

Bibliography: Response Assessment [Cost Breakdown] of Nonstructural Building Elements

Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake
doi: http://dx.doi.org/10.1193/1.4000032
http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032

Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Permalink: http://dx.doi.org/10.1061/(ASCE)CF.1943-5509.0000630
http://www.ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630

Homeowner's Guide to Earthquake Safety
Retrofitting Questions and Answers
Earthquake Safety, Inc., 2015 (web based)
http://www.earthquakesafety.com/sitemap.html

Cost and Seismic Design
https://www.google.com/#q=cpst+and+seismic+design+christopher+arnold+pdf

Seismic Analysis and Design

THE EARTHQUAKE ARCHITECTURE WEBSITE
http://www.iitk.ac.in/nicee/wcee/article/14_05-06-0185.PDF

Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee

S61-16
**Proposed Change as Submitted**

**Proponent:** Jennifer Goupil, AMERICAN SOCIETY OF CIVIL ENGINEERS, representing SELF (jgoupil@asce.org)

**2015 International Building Code**

Revise as follows:

**1604.1 General.** Building, structures and parts thereof shall be designed and constructed in accordance with strength design, load and resistance factor design, allowable stress design, empirical design or conventional construction methods, as permitted by the applicable material chapters and referenced standards.

**1604.3 Serviceability.** Structural systems and members thereof shall be designed to have adequate stiffness to limit deflections and lateral drift as indicated in Table 1604.3. See Section 12.12.1 of ASCE 7 for drift limits applicable to earthquake loading shall be in accordance with ASCE 7 Chapters 12,13,15 or 16, as applicable.

### TABLE 1604.3 DEFLECTION LIMITS

<table>
<thead>
<tr>
<th>CONSTRUCTION</th>
<th>L or L&lt;sub&gt;f&lt;/sub&gt;</th>
<th>S or W&lt;sup&gt;f&lt;/sup&gt;</th>
<th>D + L&lt;sup&gt;d,g&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof members:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Supporting plaster or stucco ceiling</td>
<td>L/360</td>
<td>L/360</td>
<td>L/240</td>
</tr>
<tr>
<td>Supporting nonplaster ceiling</td>
<td>L/240</td>
<td>L/240</td>
<td>L/180</td>
</tr>
<tr>
<td>Not supporting ceiling</td>
<td>L/180</td>
<td>L/180</td>
<td>L/120</td>
</tr>
<tr>
<td>Floor members</td>
<td>L/360</td>
<td>—</td>
<td>L/240</td>
</tr>
<tr>
<td>Exterior walls:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With plaster or stucco finishes</td>
<td>—</td>
<td>L/360</td>
<td>—</td>
</tr>
<tr>
<td>With other brittle finishes</td>
<td>—</td>
<td>L/240</td>
<td>—</td>
</tr>
<tr>
<td>With flexible finishes</td>
<td>—</td>
<td>L/120</td>
<td>—</td>
</tr>
<tr>
<td>Interior partitions:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With plaster or stucco finishes</td>
<td>L/360</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>With other brittle finishes</td>
<td>L/240</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>With flexible finishes</td>
<td>L/120</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Farm buildings</td>
<td>—</td>
<td>—</td>
<td>L/180</td>
</tr>
<tr>
<td>Greenhouses</td>
<td>—</td>
<td>—</td>
<td>L/120</td>
</tr>
</tbody>
</table>

For SI: 1 foot = 304.8 mm.

a. For structural roofing and siding made of formed metal sheets, the total load deflection shall not exceed L /60. For secondary roof structural members supporting formed metal roofing, the live load deflection shall not exceed L /150. For secondary wall members supporting formed metal siding, the design wind load deflection shall not exceed L /90. For roofs, this exception only applies when the metal sheets have no roof covering.

b. Flexible, folding and portable partitions are not governed by the provisions of this section. The deflection criterion for interior partitions is based on the horizontal load defined in Section 1607.14.

c. See Section 2403 for glass supports.

d. The deflection limit for the D+L load combination only applies to the deflection due to the creep component of long-term dead load deflection plus the short-term live load deflection. For wood structural members that are dry at time of installation and used under dry conditions in accordance with the ANSI/AWC NDS, the creep component of the long-term deflection shall be permitted to be estimated as the immediate dead load deflection resulting from 0.5 D. For wood structural members at all other moisture conditions, the creep component of the long-term deflection is permitted to be estimated as the immediate dead load deflection resulting from D. The value of 0.5 D shall not be used in combination with ANSI/AWC NDS provisions for long-term loading.

e. The above deflections do not ensure against ponding. Roofs that do not have sufficient slope or camber to ensure adequate drainage shall be investigated for ponding. See Section 1611 for rain and ponding requirements and Section 1603.4 for roof drainage requirements. Chapter 8 of ASCE 7.
f. The wind load is permitted to be taken as 0.42 times the “component and cladding" loads for the purpose of determining deflection limits herein. Where members support glass in accordance with Section 2403 using the deflection limit therein, the wind load shall be no less than 0.6 times the “component and cladding” loads for the purpose of determining deflection.

g. For steel structural members, the dead load shall be taken as zero.

h. For aluminum structural members or aluminum panels used in skylights and sloped glazing framing, roofs or walls of sunroom additions or patio covers not supporting edge of glass or aluminum sandwich panels, the total load deflection shall not exceed \( l / 60 \). For continuous aluminum structural members supporting edge of glass, the total load deflection shall not exceed \( l / 175 \) for each glass lite or \( l / 60 \) for the entire length of the member, whichever is more stringent. For aluminum sandwich panels used in roofs or walls of sunroom additions or patio covers, the total load deflection shall not exceed \( l / 120 \).

i. For cantilever members, \( l \) shall be taken as twice the length of the cantilever.

**1604.4 Analysis.** Load effects on structural members and their connections shall be determined by methods of structural analysis that take into account equilibrium, general stability, geometric compatibility and both short- and long-term material properties.

Members that tend to accumulate residual deformations under repeated service loads shall have included in their analysis the effects of added eccentricities of deformations expected to occur during their service life.

Any system or method of construction to be used shall be based on a rational analysis in accordance with well-established principles of mechanics. Such analysis shall result in a system that provides a complete load path capable of transferring loads from their point of origin to the load-resisting elements.

The total lateral force shall be distributed to the various vertical elements of the lateral force-resisting system in proportion to their rigidities, considering the rigidity of the horizontal bracing system or diaphragm. Rigid elements assumed not to be a part of the lateral force-resisting system are permitted to be incorporated into buildings provided their effect on the action of the system is considered and provided for in the design. A diaphragm is rigid for the purpose of distribution of story shear and torsional moment when the lateral deformation of the diaphragm is less than or equal to two times the average story drift.

Where required by ASCE 7, provisions shall be made for the increased forces induced on resisting elements of the structural system resulting from torsion due to eccentricity between the center of application of the lateral forces and the center of rigidity of the lateral force-resisting system.

Every structure shall be designed to resist the overturning effects caused by the lateral forces specified in this chapter. See Section 1609 for wind loads

Where sliding is used to isolate the elements, Section 1610 for lateral soil loads and Section 1613 for earthquake loads the effects of friction between sliding elements shall be included as a force.

**1604.8.2 Structural walls.** Walls that provide vertical load-bearing resistance or lateral shear resistance for a portion of the structure shall be anchored to the roof and to all floors and members that provide lateral support for the wall or that are supported by the wall. The connections shall be capable of resisting the horizontal forces specified in Section 1.4.5 of ASCE 7 for walls of structures assigned to Seismic Design Category A and to Section 12.11 of ASCE 7 for walls of structures assigned to all other seismic design categories. Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall. See Sections 1609 for wind design requirements and 1613 for earthquake design requirements.

Delete without substitution:

**1604.9 Counteracting structural actions.** Structural members, systems, components and cladding shall be designed to resist forces due to earthquakes and wind, with consideration of overturning, sliding and uplift. Continuous load paths shall be provided for transmitting these forces to the foundation. Where sliding is used to isolate the elements, the effects of friction between sliding elements shall be included as a force.

Revis as follows:

**1604.10 Wind and seismic detailing.** Lateral force-resisting systems shall meet seismic detailing requirements and limitations prescribed in this code and ASCE 7 Chapters 11, excluding Chapter 12, 13, 15, 17, and Appendix 11A as applicable, even when wind load effects are greater than seismic load effects.

**Reason:** This proposed changes to Section 1604 will harmonize the provision in the code with the 2016 edition of the referenced loading standard ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7-16). Specific reasons provided for the following section proposals:

**1604.3 Serviceability:** This modification clarifies where to find the limit deflections, in Table 1604.3, as well as provides a more accurate pointer to ASCE 7 Chapters 12, 13, 15, and 16 for drift limits applicable to earthquake loadings.
1604.4 Analysis - These modifications eliminate the partial list of forces to be included since it is incomplete. Rather than add provisions to include an exhaustive list, by removing this pointer to only a few of the required sections, to code will remove the ambiguity and liability of the partial list of required sections necessary to consider for overturning effects. Also, the requirement for sliding elements to be considered as a force is moved here from Section 1604.9, which is proposed to be deleted.

1604.8.2 Structural Walls - This modification updates the section reference to ASCE 7 Chapter 1. There are no technical changes, just an update to the correct location.

1604.9 Counteracting structural action - The proposal includes deleting this section because the list of loading considerations is not complete. Rather than try to create an exhaustive list, or keep partial list, removing this section removed the ambiguity and liability of a required list of forced necessary to consider in structural engineering design. Additionally, the requirements for provisions of continuous load paths and the consideration of frictional forces is already covered in ASCE 7 in a more complete manner.

1604.10 Wind and seismic detailing - This modification reflects the current provisions within ASCE 7-16. Appendix 11A was removed from the standard and instead of excluding any particular chapters, this proposed change calls out the primary ASCE 7 Chapter that charge specific parts of the design process. These chapters, in turn reference all other ASCE 7 Sections, other ASCE 7 Chapters, and other standards for all necessary requirements.

Cost Impact: Will not increase the cost of construction

The proposed changes will not impact the cost of construction. This proposal coordinates the IBC with the referenced loading standard ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE 7 will be updated from the 2010 edition to the 2016 edition as an Administrative Update to the 2018 I-Codes.

As of the submission date of this code change proposal, the ASCE 7 Standards Committee has completed the committee balloting on technical changes. The document is designated ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures is expected to be completed, published, and available for purchase prior to the ICC Public Comment Hearings for Group B in October of 2016. Any person interested in obtaining a public comment copy of ASCE 7-16 may do so by contacting James Neckel at ASCE (jneckel "at" asce.org).

2015 International Building Code

TABLE 1604.3
DEFLECTION LIMITS

<table>
<thead>
<tr>
<th>CONSTRUCTION</th>
<th>( L ) or ( L_r )</th>
<th>( S ) or ( W )</th>
<th>( D + (L ) or ( L_r ) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof members:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Supporting plaster or stucco ceiling</td>
<td>( \frac{l}{360} )</td>
<td>( \frac{l}{360} )</td>
<td>( \frac{l}{240} )</td>
</tr>
<tr>
<td>Supporting nonplaster ceiling</td>
<td>( \frac{l}{240} )</td>
<td>( \frac{l}{240} )</td>
<td>( \frac{l}{180} )</td>
</tr>
<tr>
<td>Not supporting ceiling</td>
<td>( \frac{l}{180} )</td>
<td>( \frac{l}{180} )</td>
<td>( \frac{l}{120} )</td>
</tr>
<tr>
<td>Floor members</td>
<td>( \frac{l}{360} )</td>
<td></td>
<td>( \frac{l}{240} )</td>
</tr>
<tr>
<td>Exterior walls:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With plaster or stucco finishes</td>
<td>—</td>
<td>( \frac{l}{360} )</td>
<td>—</td>
</tr>
<tr>
<td>With other brittle finishes</td>
<td>—</td>
<td>( \frac{l}{240} )</td>
<td>—</td>
</tr>
<tr>
<td>With flexible finishes</td>
<td>—</td>
<td>( \frac{l}{120} )</td>
<td>—</td>
</tr>
<tr>
<td>Interior partitions:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>With plaster or stucco finishes</td>
<td>( \frac{l}{360} )</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>With other brittle finishes</td>
<td>( \frac{l}{240} )</td>
<td>—</td>
<td>—</td>
</tr>
</tbody>
</table>
d. The deflection limit for the partitions is based on the horizontal load defined in Section 1607.14.

b. Flexible, folding and portable partitions are not governed by the provisions of this section. The deflection criterion for interior partitions is based on the horizontal load defined in Section 1607.14.

c. See Section 2403 for glass supports.

d. The deflection limit for the \( D + \frac{(L_1 + L_2)}{L_1} \) load combination only applies to the deflection due to the creep component of long-term dead load deflection plus the short-term live load deflection. For wood structural members that are dry at time of installation and used under dry conditions in accordance with the ANSI/AWC NDS, the creep component of the long-term deflection shall be permitted to be estimated as the immediate dead load deflection resulting from 0.5 \( D \). For wood structural members at all other moisture conditions, the creep component of the long-term deflection is permitted to be estimated as the immediate dead load deflection resulting from \( D \). The value of 0.5 \( D \) shall not be used in combination with ANSI/AWC NDS provisions for long-term loading.

e. The above deflections do not ensure against ponding. Roofs that do not have sufficient slope or camber to ensure adequate drainage shall be investigated for ponding. See Chapter 8 of ASCE 7.

f. The wind load is permitted to be taken as 0.42 times the "component and cladding" loads for the purpose of determining deflection limits herein. Where members support glass in accordance with Section 2403 using the deflection limit therein, the wind load shall be no less than 0.6 times the "component and cladding" loads for the purpose of determining deflection.

g. For steel structural members, the dead load shall be taken as zero.

h. For aluminum structural members or aluminum panels used in skylights and sloped glazing framing, roofs or walls of sunroom additions or patio covers not supporting edge of glass or aluminum sandwich panels, the total load deflection shall not exceed \( l/60 \). For continuous aluminum structural members supporting edge of glass, the total load deflection shall not exceed \( l/75 \) for each glass lite or \( l/60 \) for the entire length of the member, whichever is more stringent. For aluminum sandwich panels used in roofs or walls of sunroom additions or patio covers, the total load deflection shall not exceed \( l/120 \).

i. For cantilever members, \( l \) shall be taken as twice the length of the cantilever.

1604.4 Analysis. Load effects on structural members and their connections shall be determined by methods of structural analysis that take into account equilibrium, general stability, geometric compatibility and both short- and long-term material properties.

Members that tend to accumulate residual deformations under repeated service loads shall have included in their analysis the effects of added deformations expected to occur during their service life.

Any system or method of construction to be used shall be based on a rational analysis in accordance with well-established principles of mechanics. Such analysis shall result in a system that provides a complete load path capable of transferring loads from their point of origin to the load-resisting elements.

The total lateral force shall be distributed to the various vertical elements of the lateral force-resisting system in proportion to their rigidities, considering the rigidity of the horizontal bracing system or diaphragm. Rigid elements assumed not to be a part of the lateral force-resisting system are permitted to be incorporated into buildings provided their effect on the action of the system is considered and provided for in the design. A diaphragm is rigid for the purpose of distribution of story shear and torsional moment when the lateral deformation of the diaphragm is less than or equal to two times the average story drift. Where required by ASCE 7, provisions shall be made for the increased forces induced on resisting elements of the structural system resulting from torsion due to eccentricity between the center of application of the lateral forces and the center of rigidity of the lateral force-resisting system.

Every structure shall be designed to resist the overturning effects caused by the forces specified in this chapter, including overturning, uplift, and sliding.

Where sliding is used to isolate the elements, the effects of friction between sliding elements shall be included as a force.

1604.10 Wind and seismic detailing. Lateral force-resisting systems shall meet seismic detailing requirements and limitations prescribed in this code and ASCE 7 Chapters 11, 12, 13, 15, 17, and 18 as applicable, even when wind load effects are greater than seismic load effects.

Exception: References within ASCE 7 to Chapter 14 shall not apply, except as specifically required herein.
Committee Reason: This proposal coordinates IBC provision with the latest edition of the referenced standard, ASCE 7 which was updated in ADM94-16. The modification to Table 1604.3 makes a nomenclature correction. The modification to Section 1604.4 retains current wording that the committee believes is important.

Assembly Action: None

Individual Consideration Agenda

Proponent: Ed Berkel, representing ICC Code Correlation Committee (ccc@icc SAFE.org) requests Disapprove.

Commenter's Reason: The Code Correlation Committee requests Disapproval of this code change proposal in order to bring a correlation issue to the attention of the full membership at the Public Comment Hearings and to allow the membership to coordinate action on this code change proposal with action taken on Code Change Proposal ADM94-16.

ADM94-16 is the administrative update to referenced standards in the I-Codes. One of these standards, ASCE7, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, was proposed for update to ASCE7-16. However, a successful assembly motion requests that the referenced ASCE7 remain at ASCE7-10 as it presently is referenced in the 2015 I-Codes. This code change proposal coordinates with and relies upon reference to ASCE7-16.

The Code Correlation Committee is a standing committee of the International Code Council whose objectives, procedures and organization are set forth in Council Policy CP/#44-13. The objective of the Code Correlation Committee is to maintain technical and editorial consistency among the International Codes and to assist staff in the evaluation and processing of code change proposals and comments that are exclusively editorial.

S63-16
Proposed Change as Submitted

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com)

2015 International Building Code

Delete and substitute as follows:

1604.5 Risk category Occupancy and use categories. Each building and structure shall be assigned a risk category in accordance with Table 1604.5. Where a referenced standard specifies an occupancy category, the risk category shall not be taken as lower than the occupancy category specified therein. Where a referenced standard specifies that the assignment of a risk category be in accordance with ASCE 7, Table 1.5-1, Table 1604.5 shall be used in lieu of ASCE 7, Table 1.5-1.

Each building and structure shall be assigned an occupancy and use category in accordance with Table 1604.5. Where a referenced standard specifies an occupancy category, the occupancy and use category shall not be taken as lower than the occupancy category specified therein. Where a referenced standard specifies that the assignment of a risk category be in accordance with ASCE 7, Table 1.5-1, occupancy and use categories in Table 1604.5 shall be used in lieu of ASCE 7, Table 1.5-1.

Reason:

- “Risk” is subjective, ambiguous, and political (or “what people want”)
- “Occupancy” and “Use” are objective descriptions of rows I, II, III, and IV.

RISK is forced into this Table to imply that “we have it altogether and that we are, indeed, designing for risk.” It is called RISK because of the use of Risk-Targeted" earthquake ground motions. But, let's be honest: we really don't have it all together! It is here, because the code presently uses these "Risk-Targeted" earthquake ground motions. However, there is no supporting argument supporting its present use - the argument, if any, is based on circular reasoning, wherein the premise and the conclusion are one-and-the-same.

The code has us calculate many, many specific things in the design of a building. But there is no equation or means to “calculate RISK!”

To also further rectify the mischaracterization that we can replace the tensor (magnitude and directional) nature of real earthquake ground motion (magnitude, frequency content, duration, aftershocks) with a scalar quantity (number value only); which, because it is fictitious, has little to do with assessing the true effects of scenario earthquakes that can impact the site.

So-called “Risk-Targeted” earthquake ground motions were copied from ASCE protocols for the design of nuclear plants, and they are neither adequate nor applicable for building code applications to protect public safety from the potential earthquake threats that may occur. So-called Risk Models are strongly utilized by the insurance market, but as models have been widely criticized for being non-transparent, subjective, blurring distinctions between assumptions and facts (and too often with a weak understanding of the limitations of the model assumptions.”

Furthermore, just knowing what the risk is . . . I mean what you calculated it to be; doesn't mean that you know "what to do about it!"

Furthermore, there have been no "logical" arguments for implementing RISK-TARGETED's conceptual language into code design requirements; and because of its completely arbitrary and subjective choices for representing risk (10%/50yr; 2%/50 yr; or anything else) are propped up more by well-known "logical fallacies" and ignorance, rather than they are by actual scientific fact and "common sense."

"The man of science has learned to believe in justification, not by faith, but by verification."

"Science is simply common sense at its best, that is, rigidly accurate in observation, and merciless to fallacy in logic."

- Thomas Huxley

Bibliography: See also BIBLIOGRAPHY in Proposal: Figure 1613.3.1 RISK-TARGETED MCE

Cost Breakdown of Nonstructural Building Elements


Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake.

Cost Impact: Will not increase the cost of construction
Will not increase the cost of construction, as it is an *editorial* change in name (terminology) only.

This proposal may or may not affect the cost of construction. This is (1) because detached one- and two-family dwellings must be already built to withstand the lateral forces due to wind; and (2) must include basements, "safe rooms"), or other afforded protections to protect occupants against the deadly impacts of hurricanes and tornadoes.

The point is: Detached one- and two-family need to consider the maximum Magnitude of realistic scenario earthquakes that they could, in fact, experience.

And not be constructed vulnerable to earthquakes, because a flawed numerical hazard model "guesses" incorrectly as to the likelihood or possibility of earthquakes. This should remain a rational and a scientific decision based upon protecting both public safety and property. A second point is that "cost" due to structural elements is almost always less than 80% of the cost of a building!

"In general, better seismic performance is achieved through increased lateral design forces (i.e., base shear), and detailing requirements that improve structural connection strength or structural member behavior in the inelastic range of response. Requirements for seismic bracing and anchorage of nonstructural components reduce potential for nonstructural damage and loss of building (or system) functionality."*

* viii, Executive Summary, NIST GCR 14-917-26

Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee

1988 Uniform Building Code

1990 SEAOC BLUE BOOK

1997 Uniform Building Code

Robert E. Bachman and David R. Bonneville (2000)
Committee Action: Disapproved

Committee Reason: The committee feels the terminology, risk category, needs to be retained in the IBC. The proposal to remove this language would create a conflict between the code and the referenced load standard.

Assembly Motion: As Submitted

Online Vote Results: Failed
Support: 24.28% (67) Oppose: 75.72% (209)

Assembly Action: None

Public Hearing Results

Individual Consideration Agenda

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com) requests Approve as Submitted.

Commenter’s Reason: “If your definition is wrong, you’ll look for the wrong thing.”
- Carol Cleland

“Anyone who conducts an argument by appealing to authority is not using his intelligence; he is just using his memory.”
- Leonardo da Vinci

“Just because it comes from a consensus standard doesn’t mean it isn’t without problems.”
- Jay Crandell

The Committee Reason for Disapproval is not an argument, but simply a statement of the Logical Fallacies (1) Authority: “You said that because an authority [ASCE 7] thinks something, it must therefore be true.” The proposed terminology change (back) to Occupancy and use categories comes from the same level of understanding as that underlying ASCE 7, and it simply is applying the empirical evidence of our recent experiences with global earthquakes for a better, more logical and more consistent approach to earthquake engineering and seismic design; and

(2) Begging the Question [Circular Argument]: “You presented a circular argument in which the conclusion was included in the premise. This logically incoherent argument often arises in situations where people have an assumption that is very ingrained, and therefore taken in their minds as a given. Circular reasoning is bad mostly because it is not good. Example: the word of Zorba [ASCE 7] the Great is flawless and perfect. We know this because it says so in [ASCE 7] The Great and Infallible Book of [ASCE 7’s] Best and Most Truest Things that are Definitely True and Should Not Ever Be Questioned.”

But how can you not question ASCE 7, given its broad-speaking and really irresponsible Disclaimer (“disclaiming any and all liability”) below: “This standard was developed by a consensus standards development process . . . ”While ASCE’s process is designed to promote standards that reflect a fair and reasoned consensus among all interested participants, while preserving the public health, safety, and welfare that is paramount to its mission, it has not made an independent assessment of and does not warrant the accuracy, completeness, suitability, or utility of any information, apparatus, product, or process discussed herein. ASCE [BUMMER!] does not intend, nor should anyone interpret, ASCE’s standards to replace the sound judgment of a competent professional, having knowledge and experience in the appropriate field(s) of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the contents of this standard. ASCE has no authority to enforce compliance with its standards and does not undertake to certify products for compliance or to render any professional services to any person or entity. ASCE disclaims any and all liability for any personal injury, property damage, financial loss or other damages of any nature whatsoever, including without limitation any direct, indirect, special, exemplary, or consequential damages resulting from any person’s use of, or reliance on, this standard. Any individual who relies on this stan- dard assumes full responsibility for such use. ASCE and American Society of Civil Engineers—Registered in U.S. Patent and Trademark Office

This change in language/terminology helps to clarify the code, per Reason Statement of S71-16:

- “Risk” is subjective, ambiguous, and political (or “what people want”)
“Occupancy” and “Use” are objective descriptions of rows I, II, III, and IV.

---

“All sciences are vain and full of errors that are not born of Experience, the mother of all Knowledge.”

- Leonardo DaVinci

This recently published article Reality Check: Seismic Hazard Models You Can Trust provides the comprehensive understanding (based on empirical evidence) of why these and other associated proposed changes are needed to provide a more reasonable and more practical approach to seismic safety for the general public that is exposed to major earthquake risk – namely because the “current probabilistic methods to quantify earthquake hazards have serious problems.”

https://eos.org/opinions/reality-check-seismic-hazard-models-you-can-trust

Bibliography: ASCE 7 and SEI Standards
http://www.asce.org/structural-engineering/asce-7-and-sei-standards/
Click on ASCE 7 cover icon for a preview, including liability disclaimer statement


http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630
Proposed Change as Submitted

Proponent: Ronald Hamburger, SIMPSON GUMPERTZ & HEGER, representing SELF; and JENNIFER GOUPIL, AMERICAN SOCIETY OF CIVIL ENGINEERS, representing SELF (rohamburger@sgh.com); Jennifer Goupil (jgoupil@asce.org)

2015 International Building Code
Add new definition as follows:

SECTION 202 DEFINITIONS

TSUNAMI DESIGN GEODATABASE. The ASCE database (version 2016-1.0) of Tsunami Design Zone maps and associated design data for the states of Alaska, California, Hawaii, Oregon, and Washington.

SECTION 202 DEFINITIONS

TSUNAMI DESIGN ZONE. An area identified on the Tsunami Design Zone map between the shoreline and the inundation limit, within which certain structures designated in Chapter 16 are designed for or protected from inundation.

Revise as follows:

1604.5 Risk category. Each building and structure shall be assigned a risk category in accordance with Table 1604.5. Where a referenced standard specifies an occupancy category, the risk category shall not be taken as lower than the occupancy category specified therein. Where a referenced standard specifies that the assignment of a risk category be in accordance with ASCE 7, Table 1.5-1, Table 1604.5 shall be used in lieu of ASCE 7, Table 1.5-1.

Exception: The assignment of buildings and structures to Tsunami Risk Categories III and IV is permitted to be assigned in accordance with Section 6.4 of ASCE 7.

Add new text as follows:

SECTION 1615 TSUNAMI LOADS

1615.1 General. The design and construction of Risk Category III and IV buildings and structures located in the Tsunami Design Zones defined in the Tsunami Design Geodatabase shall be in accordance with Chapter 6 of ASCE 7, except as modified by this code.

1615.2 Definitions. The following terms are defined in Chapter 2:

TSUNAMI DESIGN GEODATABASE.
TSUNAMI DESIGN ZONE.

Reason: Many coastal areas in the western United States are subject to potentially destructive tsunamis. There are many coastal communities in Alaska, Washington, Oregon, California, and Hawaii where there is a need for tsunami-resistant design of critical infrastructure and essential facilities that provide vital services necessary for post-disaster response and recovery, and enable the continued functioning of the community. The public safety risk has been only partially mitigated through warning and preparedness of evacuation; there are many areas in these five states where complete evacuation prior to tsunami arrival cannot be ensured. Accordingly, some communities also have a need for a standard for designated tsunami vertical evacuation refuge structures as an alternative to high ground.

The American Society of Civil Engineers (ASCE) have supported a 5-year effort to address these needs and develop provisions for ASCE 7 that have incorporated the last 10 years of advances in tsunami engineering research since the 2004 Indian Ocean earthquake and tsunami. Chapter 6 Tsunami Loads and Effects is a new chapter in ASCE 7. It is important to realize that the scope of this proposal is limited to Tsunami Risk Category III and IV structures, and it has an exception so that the local jurisdiction can evaluate the physical and demographic context of the tsunami-inundated zone in assigning these categories to the facilities they deem to be vital to public health, safety, and welfare. There would be no mandatory requirements for Risk Category I and II buildings and structures.

ASCE is publishing a new tsunami design guide with worked examples to assist structural engineers applying these new provisions.


Cost Impact: Will increase the cost of construction
This proposal may increase the cost of construction depending on the tsunami inundation depth at the structure. Cost studies have shown this increase to be very small, given that the Risk Category III and IV structures in these five western states will already be designed for high seismic loads and ductile detailing. There may be some enhanced tsunami design necessary for vertical load carrying elements of minimal dimensions and capacity. As with other flooding effects, foundations must resist scour.
This proposal adds the pointer in the IBC to refer to the tsunami load provisions in the referenced loading standard ASCE 7. ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures will be updated from the 2010 edition to the 2016 edition as an Administrative Update to the 2018 I-Codes.
As of the submission date of this code change proposal, the ASCE 7 Standard Committee has completed the committee balloting on the technical changes. The document designated ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures is expected to be completed, published, and available for purchase prior to the ICC Public Comment hearings for Group B in October 2016. Any person interested in obtaining a public comment copy of ASCE 7-16 may do so by contacting James Neckel at ASCE.

Public Hearing Results

Committee Action: Approved as Submitted
Committee Reason: This proposal to address tsunami loads is desperately needed. It only affects Risk Categories III & IV and it is not applicable to existing structures.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:
Proponent: Gary Ehrlich, National Association of Home Builders, representing National Association of Home Builders (gehrich@nahb.org) requests Approve as Modified by this Public Comment.
Modify as Follows:

2015 International Building Code

1615.1 General. The design and construction of Risk Category III and IV buildings and structures located in the Tsunami Design Zones defined in the Tsunami Design Geodatabase shall be in accordance with Chapter 6 of ASCE 7, except as modified by this code.

Commenter's Reason: The purpose of this public comment is to explicitly limit the application of the new tsunami design requirements in ASCE 7-16 to Risk Category III and IV buildings. NAHB is concerned that the door has been left open, by both the ASCE 7 Chapter 6 commentary and this code change, to encourage communities to amend Section 1615.1 to include Risk Category II buildings, and specifically to include low-rise multifamily residential construction.
The cost studies presented to the ASCE 7 Main Committee were of 6-story structural steel frame and concrete frame or bearing wall buildings. No studies were presented of lower-rise buildings, many of which (especially in the multifamily arena) use light frame wood or cold-formed steel construction. Such buildings would need to convert one or more light-frame floors to structural steel or concrete, and due to the lighter-weight structural the lateral loads on the seismic force-resisting system for the lower floors will be lower. Thus, the design impact and cost for such buildings could be substantially higher than that presented to the ASCE committee in examples and claimed in the cost substantiation for this code change. This could not only compromise affordability of multifamily residential construction in many communities, but could act as an effective land use restriction to move construction out of the tsunami design zone.
Most engineers applying ASCE 7-16 and the 2018 I-Codes and doing tsunami-resistant design will be seeing these provisions for the first time. Many elements of Chapter 6 are complex, and not all of the many significant questions raised about clarity and enforceability of the design provisions were addressed during balloting. Plus, one will probably need to hire a civil engineer, coastal engineer, or other expert to run the analysis to determine just what the applicable inundation depth from the design tsunami is. There will be a substantial learning curve for engineers to understand and apply these provisions, and NAHB is concerned that simply relying on engineers purchasing a “design guide with worked examples” will be insufficient to insure accurate, cost-effective designs.

Proponent: Ed Berkel, representing ICC Code Correlation Committee (ccc@iccsafe.org) requests Disapprove.

Commenter's Reason: The Code Correlation Committee requests Disapproval of this code change proposal in order to bring a correlation issue to the attention of the full membership at the Public Comment Hearings and to allow the membership to coordinate action on this code change proposal with action taken on Code Change Proposal ADM94-16.

ADM94-16 is the administrative update to referenced standards in the I-Codes. One of these standards, ASCE7, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, was proposed for update to ASCE7-16. However, a successful assembly motion requests that the referenced ASCE7 remain at ASCE7-10 as it presently is referenced in the 2015 I-Codes. This code change proposal coordinates with and relies upon reference to ASCE7-16.

The Code Correlation Committee is a standing committee of the International Code Council whose objectives, procedures and organization are set forth in Council Policy CP#44-13. The objective of the Code Correlation Committee is to maintain technical and editorial consistency among the International Codes and to assist staff in the evaluation and processing of code change proposals and comments that are exclusively editorial.
Proposed Change as Submitted

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com)

2015 International Building Code
Revise as follows:

TABLE 1604.5
RISK CATEGORY OCCUPANCY AND USE CATEGORIES OF BUILDINGS AND OTHER STRUCTURES

<table>
<thead>
<tr>
<th>RISK CATEGORY AND USE CATEGORIES</th>
<th>NATURE OF OCCUPANCY</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to:</td>
</tr>
<tr>
<td></td>
<td>• Agricultural facilities.</td>
</tr>
<tr>
<td></td>
<td>• Certain temporary facilities.</td>
</tr>
<tr>
<td></td>
<td>• Minor storage facilities.</td>
</tr>
<tr>
<td>II</td>
<td>Buildings and other structures except those listed in Risk Categories I, III and IV.</td>
</tr>
<tr>
<td>III</td>
<td>Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to:</td>
</tr>
<tr>
<td></td>
<td>• Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300.</td>
</tr>
<tr>
<td></td>
<td>• Buildings and other structures containing Group E occupancies with an occupant load greater than 250.</td>
</tr>
<tr>
<td></td>
<td>• Buildings and other structures containing educational occupancies for students above the 12th grade with an occupant load greater than 500.</td>
</tr>
<tr>
<td></td>
<td>• Group I-2 occupancies with an occupant load of 50 or more resident care recipients but not having surgery or emergency treatment facilities.</td>
</tr>
<tr>
<td></td>
<td>• Group I-3 occupancies.</td>
</tr>
<tr>
<td></td>
<td>• Any other occupancy with an occupant load greater than 5,000.</td>
</tr>
<tr>
<td></td>
<td>• Power-generating stations, water treatment facilities for potable water, wastewater treatment facilities and other public utility facilities not included in Risk Category IV.</td>
</tr>
<tr>
<td></td>
<td>• Buildings and other structures not included in Risk Category IV containing quantities of toxic or explosive materials that exceed maximum allowable quantities per control area as given in Table 307.1(1) or 307.1(2) or per outdoor control area in accordance with the International Fire Code; and are sufficient to pose a threat to the public if released.</td>
</tr>
<tr>
<td>IV</td>
<td>Buildings and other structures designated as essential facilities, including but not limited to:</td>
</tr>
<tr>
<td></td>
<td>• Group I-2 occupancies having surgery or emergency treatment facilities.</td>
</tr>
<tr>
<td></td>
<td>• Fire, rescue, ambulance and police stations and emergency vehicle garages.</td>
</tr>
<tr>
<td></td>
<td>• Designated earthquake, hurricane or other emergency shelters.</td>
</tr>
<tr>
<td></td>
<td>• Designated emergency preparedness, communications and operations centers and other facilities required for emergency response.</td>
</tr>
<tr>
<td></td>
<td>• Power-generating stations and other public utility facilities required as emergency backup facilities for Risk Category IV structures.</td>
</tr>
<tr>
<td></td>
<td>• Buildings and other structures containing quantities of highly toxic materials that exceed maximum allowable quantities per control area as given in Table 307.1(2) or per outdoor control area in accordance with the International Fire Code; and are sufficient to pose a threat to the public if released.</td>
</tr>
<tr>
<td></td>
<td>• Aviation control towers, air traffic control centers and emergency aircraft hangars.</td>
</tr>
<tr>
<td></td>
<td>• Buildings and other structures having critical national defense functions.</td>
</tr>
<tr>
<td></td>
<td>• Water storage facilities and pump structures required to maintain water pressure for fire suppression.</td>
</tr>
</tbody>
</table>

a. For purposes of occupant load calculation, occupancies required by Table 1004.1.2 to use gross floor area calculations shall be permitted to use net floor areas to determine the total occupant load.

b. Where approved by the building official, the classification of buildings and other structures as Risk Category III or IV based on their quantities of toxic, highly toxic or explosive materials is permitted to be reduced to Risk Category II, provided it can be demonstrated by a hazard assessment in accordance with Section 1.5.3 of ASCE 7 that a release of the toxic, highly toxic or explosive materials is not sufficient to pose a threat to the public.
Reason:
- For consistancy in language / terminology with section 1604.5 Occupancy and Use Categories.
- footnote b is superfluous, since they either "are sufficient to pose a risk to the public if released" . . . or they aren't!

Bibliography:
See also BIBLIOGRAPHY in Proposal: Figure 1613.3.1 RISK-TARGETED MCE_R

Cost Breakdown of Nonstructural Building Elements
Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake.
http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032

Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Permalink: http://dx.doi.org/10.1061/(ASCE)CF.1943-5509.0000630
Read More: http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630
Homeowner's Guide to Earthquake Safety

Retrofitting Questions and Answers
Earthquake Safety, Inc., 2015 (web based)
http://www.earthquakesafety.com/earthquake-retrofitting-faq.html

Cost and Seismic Design
https://www.google.com/#q=cost+and+seismic+design+christopher+arnold+pdf

Earthquake Architecture website
http://www.iitk.ac.in/nicee/wcee/article/14_05-06-0185.PDF

Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee

1988 Uniform Building Code

1990 SEAOC BLUE BOOK

1997 Uniform Building Code

Robert E. Bachman and David R. Bonneville (2000)

Cost Impact: Will not increase the cost of construction
Will not increase the cost of construction

Will not increase the cost of construction, as it is an editorial change in name (terminology) only.
Also, footnote b is superfluous, since they either "are sufficient to pose a risk to the public if released" . . . or they aren't!

This proposal may or may not affect the cost of construction. This is (1) because detached one- and two-family dwellings must be already built to withstand the lateral forces due to wind; and (2) must include basements, "safe rooms"), or other afforded
protections to protect occupants against the deadly impacts of hurricanes and tornadoes.

The point is; Detached one- and two-family need to consider the maximum Magnitude of realistic scenario earthquakes that they could, in fact, experience.

And not be constructed vulnerable to earthquakes, because a flawed numerical hazard model "guesses" incorrectly as to the likelihood or possibility of earthquakes. This should remain a rational and a scientific decision based upon protecting both public safety and property. A second point is that "cost" due to structural elements is almost always less than 80% of the cost of a building!

"In general, better seismic performance is achieved through increased lateral design forces (i.e., base shear), and detailing requirements that improve structural connection strength or structural member behavior in the inelastic range of response. Requirements for seismic bracing and anchorage of nonstructural components reduce potential for nonstructural damage and loss of building (or system) functionality."

* viii, Executive Summary, NIST GCR 14-917-26
NEHRP Consultants Joint Venture A partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering.

In general, where costs might be increased, cost premiums above requirements for wind tend to fall within a range of +1-3%. For cases where seismic requirements would be now additional to what previous codes either applied/neglected/failed to enforce, estimates probably would fall within the range of 0.25 - 1%.

**Public Hearing Results**

Committee Action: Disapproved
Committee Reason: Disapproval is consistent with prior action to retain "risk category".

Assembly Action: None

**Individual Consideration Agenda**

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com) requests Approve as Submitted.

"It is better to be right than to be consistent."
- Winston Churchill

"If your definition is wrong, you'll look for the wrong thing."
- Carol Cleland

"Anyone who conducts an argument by appealing to authority is not using his intelligence; he is just using his memory."
- Leonardo da Vinci

"Just because it comes from a consensus standard doesn't mean it isn't without problems."
- Jay Crandell

The Committee Reason for Disapproval is not an argument, but simply a statement of the Logical Fallacies (1) Authority: "You said that because an authority [ASCE 7] thinks something, it must therefore be true." The proposed terminology change (back) to Occupancy and use categories comes from the same level of understanding as that underlying ASCE 7, and it simply is applying the empirical evidence of our recent experiences with global earthquakes for a better, more logical and more consistent approach to earthquake engineering and seismic design; and
(2) Begging the Question [Circular Argument]: "You presented a circular argument in which the conclusion was included in the premise. This logically incoherent argument often arises in situations where people have an assumption that is very ingrained, and therefore taken in their minds as a given. Circular reasoning is bad mostly because it is not good. Example: the word of Zorba [ASCE 7] the Great is flawless and perfect. We know this because it says so in [ASCE 7] The Great and Infallible Book of [ASCE 7's] Best and Most Truest Things that are Definitely True and Should Not Ever Be Questioned."

But how can you not question ASCE 7, given its broad-speaking and really irresponsible Disclaimer ("disclaiming any and all liability") below:

"This standard was developed by a consensus standards development process . . .
“While ASCE’s process is designed to promote standards that reflect a fair and reasoned consensus among all interested participants, while preserving the public health, safety, and welfare that is paramount to its mission, it has not made an independent assessment of and does not warrant the accuracy, completeness, suitability, or utility of any information, apparatus, product, or process discussed herein. ASCE does not intend, nor should anyone interpret, ASCE’s standards to replace the sound judgment of a competent professional, having knowledge and experience in the appropriate field(s) of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the contents of this standard. ASCE has no authority to enforce compliance with its standards and does not undertake to certify products for compliance or to render any professional services to any person or entity. ASCE disclaims any and all liability for any personal injury, property damage, financial loss or other damages of any nature whatsoever, including without limitation any direct, indirect, special, exemplary, or consequential damages resulting from any person’s use of, or reliance on, this standard. Any individual who relies on this standard assumes full responsibility for such use. ASCE and American Society of Civil Engineers — Registered in U.S. Patent and Trademark Office

This change in language/terminology helps to clarify the code, per Reason Statements of S71-16 and S73-16:

- “Risk” is subjective, ambiguous, and political (or “what people want”)
- “Occupancy” and “Use” are objective descriptions of rows I, II, III, and IV...
"All sciences are vain and full of errors that are not born of Experience, the mother of all Knowledge."

- Leonardo da Vinci

This recently published article Reality Check: Seismic Hazard Models You Can Trust provides the comprehensive understanding (based on empirical evidence) of why these and other associated proposed changes are needed to provide a more reasonable and more practical approach to seismic safety for the general public that is exposed to major earthquake risk – namely because the "current probabilistic methods to quantify earthquake hazards have serious problems."


Bibliography: ASCE 7 and SEI Standards
click on ASCE 7 cover icon for a preview, including liability disclaimer statement

Response Assessment [Cost Breakdown] of Nonstructural Building Elements

Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake
doi: http://dx.doi.org/10.1193/1.4000032 (http://dx.doi.org/10.1193/1.4000032)

Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Permalink: http://dx.doi.org/10.1061/(ASCE)CF.1943-5509.0000630 (http://dx.doi.org/10.1061/(ASCE)CF.1943-5509.0000630)
Homeowner's Guide to Earthquake Safety

Retrofitting Questions and Answers
Earthquake Safety, Inc., 2015 (web based)
http://www.earthquakesafety.com/earthquake-retrofitting-faq.html

Cost and Seismic Design
https://www.google.com/#q=cost+and+seismic+design+christopher+arnold+pdf

Earthquake Architecture website
http://www.iitk.ac.in/nicee/w cee/article/14_05-06-0185.PDF

Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee

1988 Uniform Building Code

1990 SEAOC BLUE BOOK Recommended Lateral Force Requirements and Commentary

1999 SEAOC BLUE BOOK Recommended Lateral Force Requirements and Commentary
http://www.buildersbook.com/000S99.html

SEAOC Blue Book - Seismic Design Recommendations Preface to the Online Edition
http://www.seaoc.org/bluebook/index.html

Building Codes, Standards and Resource Documents: A Status Report
http://skghoshassociates.com/sk_publication/PCI_March02_bldg_codes_stand.pdf

1997 Uniform Building Code


Proposed Change as Submitted

Proponent: Joseph Cain, SunEdison, representing Solar Energy Industries Association (SEIA) (joecainpe@aol.com)

2015 International Building Code

Revise as follows:

### TABLE 1604.5

**RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES**

<table>
<thead>
<tr>
<th>RISK CATEGORY</th>
<th>NATURE OF OCCUPANCY</th>
</tr>
</thead>
</table>
| I             | Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to:  
• Agricultural facilities.  
• Certain temporary facilities.  
• Minor storage facilities.  
• Ground-mounted *photovoltaic panel* systems with no use underneath. |
| II            | Buildings and other structures except those listed in Risk Categories I, III and IV. |
| III           | Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to:  
• Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300.  
• Buildings and other structures containing Group E occupancies with an occupant load greater than 250.  
• Buildings and other structures containing educational occupancies for students above the 12th grade with an occupant load greater than 500.  
• Group I-2 occupancies with an occupant load of 50 or more resident care recipients but not having surgery or emergency treatment facilities.  
• Group I-3 occupancies.  
• Any other occupancy with an occupant load greater than 5,000.  
• Power-generating stations, water treatment facilities for potable water, wastewater treatment facilities and other public utility facilities not included in Risk Category IV.  
• Buildings and other structures not included in Risk Category IV containing quantities of toxic or explosive materials that:  
  - Exceed maximum allowable quantities per control area as given in Table 307.1(1) or 307.1(2) or per outdoor control area in accordance with the *International Fire Code*; and  
  - Are sufficient to pose a threat to the public if released. |
| IV            | Buildings and other structures designated as essential facilities, including but not limited to:  
• Group I-2 occupancies having surgery or emergency treatment facilities.  
• Fire, rescue, ambulance and police stations and emergency vehicle garages.  
• Designated earthquake, hurricane or other emergency shelters.  
• Designated emergency preparedness, communications and operations centers and other facilities required for emergency response.  
• Power-generating stations and other public utility facilities required as emergency backup facilities for Risk Category IV structures.  
• Buildings and other structures containing quantities of highly toxic materials that:  
  - Exceed maximum allowable quantities per control area as given in Table 307.1(2) or per outdoor control area in accordance with the *International Fire Code*; and  
  - Are sufficient to pose a threat to the public if released.  
• Aviation control towers, air traffic control centers and emergency aircraft hangars.  
• Buildings and other structures having critical national defense functions.  
• Water storage facilities and pump structures required to maintain water pressure for fire suppression. |

a. For purposes of occupant load calculation, occupancies required by Table 1004.1.2 to use gross floor area calculations shall be permitted to use net floor areas to determine the total occupant load.

b. Where approved by the building official, the classification of buildings and other structures as Risk Category III or IV based on their quantities of toxic, highly toxic or explosive materials is permitted to be reduced to Risk Category II, provided it can be demonstrated by a hazard assessment in accordance with Section 1.5.3 of ASCE 7 that a release of the toxic, highly toxic or explosive materials is not sufficient to pose a threat to the public.

**Reason:** Table 1604.5 is presently silent for ground-mounted solar photovoltaic panel systems. The Solar Energy Industries...
Association (SEIA) is aware of a broad range of interpretation by local authorities regarding proper assignment of Risk Category for ground-mounted PV systems. This is especially true -- and especially impactful -- for large-scale (often referred to as "utility scale") solar power plants. Given the same set of construction drawings, different building department staff can reach different conclusions, based on different rationale. Different building departments have reviewed the same plant design and determined it was Risk Category I, or Risk Category II, or Risk Category III. A few reviewers have even claimed the same design should be assigned as Risk Category IV. Owing to this broad range of opinions and beliefs, the solar industry cannot design a solar power plant without first asking the building code official to make this determination, and the design features and cost of a solar power plant are therefore dependent on individual opinions and beliefs of reviewers. This inconsistency often creates an unnecessary increase in the cost of construction.

This inconsistency in the assignment of Risk Category is sometimes based on the Risk Category III item that reads: "Power-generating stations, water treatment facilities for potable water, wastewater treatment facilities and other public utility facilities not included in Risk Category IV."

This proposal offers a solution by creating a new definition of Electrical Power Generating Facilities, using language consistent with ASCE 7-10 Section 15.5.4.1, Electrical Power Generating Facilities. The newly defined term is then used under Risk Category III in Table 1604.5. New language is added under Risk Category I (one) in Table 1604.5 to clarify that RC I is appropriate for ground-mounted photovoltaic panel systems with no use underneath. Other portions of a power plant for which failure might impact the grid -- such as substations -- are not included in this description under Risk Category I.

Justification is provided in the following paragraphs:

ASCE 7-10 Commentary C1.5 states: "Risk Category III includes buildings that house a large number of persons in one place. ... This category has also included structures associated with utilities required to protect the health and safety of a community, including power generating stations and water treatment and sewage treatment plants. ... Failures of power plants that supply electricity on the national grid can cause substantial economic losses and disruption to civilian life when their failures can trigger other plants to go offline in succession. The result can be massive and potentially extended power outage, shortage, or both that lead to huge economic losses because of idled industries and a serious disruption of civilian life because of inoperable subways, road traffic signals, and so forth.

1. Ground-mounted photovoltaic panel systems with no use underneath do not "represent a substantial hazard to human life in the event of failure. Therefore, they do not meet this criteria for Risk Category III.

2. Photovoltaic panel systems are by their nature an intermittent power source. They convert sunlight to electricity, producing power during daylight hours only. This is an interrupted power source by its nature. Photovoltaic power systems do not cause substantial economic losses and disruption to civilian life if they fail or during night-time hours. Where structural failures have occurred in ground-mounted PV panel systems, those failures have been localized and did not trigger a complete shut-down of a power plant. Where electrical faults are detected, individual inverters can shut down portions of a power plant, without any disruption to civilian life. Therefore, they do not meet this criteria for Risk Category III.

IBC Table 1604.5 describes Risk Category IV structures as "Buildings and other structures designated as essential facilities," and includes "Power-generating stations ... required as emergency backup facilities for Risk Category IV structures."

3. The intermittent nature of power generation makes ground-mounted photovoltaic panel systems an extremely unlikely choice as an on-site, sole-source required emergency backup facility for a Risk Category IV structure, even if paired with an energy storage system. Therefore, they do not meet this criteria for Risk Category IV.

IBC Table 1604.5 describes Risk Category I structures as: "Buildings and other structures that represent a low hazard to human life in the event of failure." ASCE 7-10 Commentary C1.5 states: "Risk Category I structures generally encompass buildings and structures that normally are unoccupied and that would result in negligible risk to the public should they fail."

4. Ground-mounted photovoltaic panel systems with no use underneath are not occupied. Facilities including ground-mounted photovoltaic panel systems are generally located within a fenced area staffed by a small team of trained and qualified individuals who monitor performance and provide maintenance. Where structural failures have occurred, they have been triggered by wind events. During wind events, the trained and qualified staff can be expected to have heightened awareness. This scenario represents a low hazard to human life. Therefore, these systems meet this criteria for Risk Category I.

5. As these systems do not meet the criteria for Risk Categories III or IV, and they do meet the criteria for Risk Category I, they are not Risk Category II.

For a parallel perspective, note that a joint publication of the American Society of Civil Engineers (ASCE) and the American Wind Energy Association (AWEA) recommends Occupancy/Risk Category II for wind turbine structures.

6. Ground-mounted photovoltaic panel systems represent a lower level of hazard to human life than wind turbines in the event of failure. Therefore, it follows that if ASCE recommends assignment of Risk Category II for wind turbines, the lower Risk Category for ground-mounted solar is further justified.
Cost Impact: Will not increase the cost of construction
Assignment as Risk Category I will not increase the cost of construction, and will avoid unnecessary increase in cost of construction owing to arbitrary assignments to higher Risk Categories.

Public Hearing Results

Committee Reason: The committee believes that the general provisions adequately address the risk category classification. No specific justification was provided for adding the proposed occupancy. To do so will encourage other industries to request similar exceptions.

Assignment as Risk Category I will not increase the cost of construction, and will avoid unnecessary increase in cost of construction owing to arbitrary assignments to higher Risk Categories.

Individual Consideration Agenda

Public Comment 1:
Proponent: Joseph Cain, representing Solar Energy Industries Association (SEIA) (JoeCainPE@gmail.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

TABLE 1604.5
RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

<table>
<thead>
<tr>
<th>RISK CATEGORY</th>
<th>NATURE OF OCCUPANCY</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to:</td>
</tr>
<tr>
<td></td>
<td>• Agricultural facilities.</td>
</tr>
<tr>
<td></td>
<td>• Certain temporary facilities.</td>
</tr>
<tr>
<td></td>
<td>• Minor storage facilities.</td>
</tr>
<tr>
<td></td>
<td>• Ground-mounted photovoltaic panel systems with no use underneath in areas secured to prevent unauthorized access except those required as emergency backup facilities for Risk Category IV structures.</td>
</tr>
<tr>
<td>II</td>
<td>Buildings and other structures except those listed in Risk Categories I, III and IV.</td>
</tr>
<tr>
<td>III</td>
<td>Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to:</td>
</tr>
<tr>
<td></td>
<td>• Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300.</td>
</tr>
<tr>
<td></td>
<td>• Buildings and other structures containing Group E occupancies with an occupant load greater than 250.</td>
</tr>
<tr>
<td></td>
<td>• Buildings and other structures containing educational occupancies for students above the 12th grade with an occupant load greater than 500.</td>
</tr>
<tr>
<td></td>
<td>• Group I-2 occupancies with an occupant load of 50 or more resident care recipients but not having surgery or emergency treatment facilities.</td>
</tr>
<tr>
<td></td>
<td>• Group I-3 occupancies.</td>
</tr>
<tr>
<td></td>
<td>• Any other occupancy with an occupant load greater than 5,000.</td>
</tr>
<tr>
<td></td>
<td>α</td>
</tr>
</tbody>
</table>

S74-16 : TABLE 1604.5-
CAIN10911

| IV | Buildings and other structures designated as essential facilities, including but not limited to:
|   | • Group I-2 occupancies having surgery or emergency treatment facilities.
|   | • Fire, rescue, ambulance and police stations and emergency vehicle garages.
|   | • Designated earthquake, hurricane or other emergency shelters.
|   | • Designated emergency preparedness, communications and operations centers and other facilities required for emergency response.
|   | • Power-generating stations and other public utility facilities required as emergency backup facilities for Risk Category IV structures.
|   | • Buildings and other structures containing quantities of highly toxic materials that:
|   | Exceed maximum allowable quantities per control area as given in Table 307.1(2) or per outdoor control area in accordance with the *International Fire Code*; and
|   | Are sufficient to pose a threat to the public if released.\(^1\)
|   | • Aviation control towers, air traffic control centers and emergency aircraft hangars.
|   | • Buildings and other structures having critical national defense functions.
|   | • Water storage facilities and pump structures required to maintain water pressure for fire suppression.

|   | a. For purposes of occupant load calculation, occupancies required by Table 1004.1.2 to use gross floor area calculations shall be permitted to use net floor areas to determine the total occupant load.
|   | b. Where approved by the building official, the classification of buildings and other structures as Risk Category III or IV based on their quantities of toxic, highly toxic or explosive materials is permitted to be reduced to Risk Category II, provided it can be demonstrated by a hazard assessment in accordance with Section 1.5.3 of ASCE 7 that a release of the toxic, highly toxic or explosive materials is not sufficient to pose a threat to the public.

**Commenter's Reason:** The proponent is seeking Approval As Modified by Public Comment, having added two additional constraints to respond to testimony at the Committee Action Hearings.

The language “in areas secured to prevent unauthorized access” is added to address the concern that the general public be kept away from the ground-mounted photovoltaic panel systems. This is generally the case, as perimeter fencing is usual used to reduce losses from theft, and power plants have only trained operations and maintenance staff on-site. This added language should address the "behind the fence" concerns.

Language is also added to constrain the Risk Category I designation to those photovoltaic panel systems that do not provide emergency backup power to Essential Services Facilities or any other Risk Category IV buildings and other structures. This should address other concerns expressed by the Committee during the Committee Action Hearings.

The "Nature of Occupancy" for Risk Category I is defined in Table 1604.5 as "Buildings and other structures that represent a low hazard to human life in the event of failure." By specific reference, Risk Category I buildings and other structures include, but are not limited to:

- Agricultural facilities
- Certain temporary facilities
- Minor storage facilities

It is reasonable to assume that agricultural facilities and minor storage facilities are occasionally occupied by humans. Ground-mounted photovoltaic panel systems have no human occupancy beneath them. They are not occupied structures, as are the other uses in the list. Therefore, ground-mounted photovoltaic panel systems represent a lower hazard to human life in the event of failure that those other uses that are specifically listed.

The additional constraints/restrictions created by the language of this public comment further reduces the already low hazard to human life in the event of failure.
Proposed Change as Submitted

Proponent: Jennifer Goupil, AMERICAN SOCIETY OF CIVIL ENGINEERS, representing SELF (jgoupil@asce.org)

2015 International Building Code

Revise as follows:

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

1. The load combinations specified in Section 1605.2, 1605.3.1 or 1605.3.2;
2. The load combinations specified in Chapters 18 through 23; and
3. The seismic load effects including overstrength factor in accordance with Section 12.4.3 2.3.6 and 2.4.5 of ASCE 7 where required by Section 12.2.5.2 Chapter 12, 12.3.3.3 or 12.10.2.1 13, and 15 of ASCE 7. With the simplified procedure of ASCE 7 Section 12.14, the seismic load effects including overstrength factor in accordance with Section 12.14.3.2 and Chapter 2 of ASCE 7 shall be used.

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

Where the load combinations with overstrength factor in Section 12.4.3.2 2.3.6 and 2.4.5 of ASCE 7 apply, they shall be used as follows:

3.1. The basic combinations for strength design with overstrength factor in lieu of Equations 16-5 and 16-7 in Section 1605.2.
3.2. The basic combinations for allowable stress design with overstrength factor in lieu of Equations 16-12, 16-14 and 16-16 in Section 1605.3.1.
3.3. The basic combinations for allowable stress design with overstrength factor in lieu of Equations 16-21 and 16-22 in Section 1605.3.2.

1605.2.1 Other loads. Where flood loads, \( F_a \), are to be considered in the design, the load combinations of Section 2.3.3 2.3.2 of ASCE 7 shall be used. Where self-straining loads, \( T \), are considered in design, their structural effects in combination with other loads shall be determined in accordance with Section 2.3.5 2.3.4 of ASCE 7. Where an ice-sensitive structure is subjected to loads due to atmospheric icing, the load combinations of Section 2.3.4 2.3.3 of ASCE 7 shall be considered.

1605.3.2 Alternative basic load combinations. In lieu of the basic load combinations specified in Section 1605.3.1, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following combinations. When using these alternative basic allowable stress load combinations that include wind or seismic loads, allowable stresses are permitted to be increased or load combinations reduced where permitted by the material chapter of this code or the referenced standards. For load combinations that include the counteracting effects of dead and wind loads, only two-thirds of the minimum dead load likely to be in place during a design wind event shall be used. When using allowable stresses that have been increased or load combinations that have been reduced as permitted by the material chapter of this code or the referenced standards, where wind loads are calculated in accordance with Chapters 26 through 31 of ASCE 7, the coefficient (\( \omega \)) in the following equations shall be taken as 1.3. For other wind loads, (\( \omega \)) shall be taken as 1. When allowable stresses have not been increased or load combinations have not been reduced as permitted by the material chapter of this code or the referenced standards, (\( \omega \)) shall be taken as 1. When using these alternative basic load combinations to evaluate sliding, overturning and soil bearing at the soil-structure interface, the reduction of foundation overturning from Section 12.13.4 in ASCE 7 shall not be used. When using these alternative basic load combinations for proportioning foundations for loadings, which include seismic loads, the vertical seismic load effect, \( E_v \), in Equation 12.4-4 of ASCE 7 is permitted to be taken equal to zero.

\[
D + L + (L_I \text{ or } S \text{ or } R) \quad (\text{Equation 16-17})
\]
\[
D + L + 0.6 \omega W \quad (\text{Equation 16-18})
\]
\[
D + L + 0.6 \omega W + S/2 \quad (\text{Equation 16-19})
\]
\[
D + L + S + 0.6 \omega W/2 \quad (\text{Equation 16-20})
\]
\[
D + L + S + E/1.4 \quad (\text{Equation 16-21})
\]
\[
0.9 D + E/1.4 \quad (\text{Equation 16-22})
\]

Exceptions:

1. Crane hook loads need not be combined with roof live loads or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m²) or less and roof live loads of 30 psf (1.44 kN/m²) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m²), 20 percent shall be combined with seismic loads.
**Reason:** This proposal is a coordination proposal to bring the 2018 IBC up to date with the provisions for the 2016 edition of *ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7-16). In the 2016 edition of the standard, the seismic load combinations were relocated to Chapter 2 Load Combinations, therefore providing all applicable load combinations in one chapter.

Section 1605 transcribes the ASCE 7 Load Combinations to the IBC, and it is important to note that this proposal does not result in any substantive technical changes as all Load Combinations presently used by structural engineers remain consistent with previous requirements of the standard and the code. This proposal only coordinates the location and intent of the load combinations.

**Cost Impact:** Will not increase the cost of construction

The proposed changes will not impact the cost of construction. This proposal coordinates the IBC with the referenced loading standard *ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. ASCE 7 will be updated from the 2010 edition to the 2016 edition as an Administrative Update to the 2018 I-Codes.

As of the submission date of this code change proposal, the ASCE 7 Standards Committee has completed the committee balloting on technical changes. The document is designated *ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures* is expected to be completed, published, and available for purchase prior to the ICC Public Comment Hearings for Group B in October of 2016. Any person interested in obtaining a public comment copy of ASCE 7-16 may do so by contacting James Neckel at ASCE (jneckel "at" asce.org).

---

**Public Hearing Results**

**Committee Action:** Approved as Submitted

**Committee Reason:** This code change updates IBC provisions to coordinate with the latest edition of the referenced standard, ASCE 7, which was updated in ADM94-16.

**Assembly Action:** None

---

**Individual Consideration Agenda**

**Proponent:** Ed Berkel, representing ICC Code Correlation Committee (ccc@iccsafe.org) requests Disapprove.

**Commenter's Reason:** The Code Correlation Committee requests Disapproval of this code change proposal in order to bring a correlation issue to the attention of the full membership at the Public Comment Hearings and to allow the membership to coordinate action on this code change proposal with action taken on Code Change Proposal ADM94-16.

ADM94-16 is the administrative update to referenced standards in the I-Codes. One of these standards, ASCE7, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, was proposed for update to ASCE7-16. However, a successful assembly motion requests that the referenced ASCE7 remain at ASCE7-10 as it presently is referenced in the 2015 I-Codes. This code change proposal coordinates with and relies upon reference to ASCE7-16.

The Code Correlation Committee is a standing committee of the International Code Council whose objectives, procedures and organization are set forth in Council Policy CP#44-13. The objective of the Code Correlation Committee is to maintain technical and editorial consistency among the International Codes and to assist staff in the evaluation and processing of code change proposals and comments that are exclusively editorial.

---

**S77-16**
The load combinations specified in Section 1605.2, 1605.3.1 or 1605.3.2; ASCE 7 Section 2.3, the basic combinations for strength design with overstrength factor in lieu of Equations 16-5 and 16-7 in Section 1605.2.

2.3

Applicable loads shall be considered, including both earthquake and wind, the Strength Load Combinations specified in accordance with ASCE 7 Section 2.3, the Allowable Stress Design Load Combinations specified load combinations. Each load combination shall also be investigated with one in ASCE 7 Section 2.4 or more the Alternative Allowable Stress Design Load Combinations of the variable loads set.

Exception: The modifications to zero.

Where the load combinations with overstrength factor in Section 12.4.3.2 Load Combinations of ASCE 7 apply Section 2.3, they ASCE 7 Section 2.4, and Section 1605.2 specified in ASCE 7 Chapter 18 and 19 shall be used as follows:

1. The basic combinations for strength design with overstrength factor in lieu of Equations 16.5 and 16.7 in Section 1605.2.

2. The basic combinations for allowable stress design with overstrength factor in lieu of Equations 16-12, 16-14 and 16-16 in Section 1605.3.1.

3. The basic combinations for allowable stress design with overstrength factor in lieu of Equations 16-21 and 16-22 in Section 1605.3.2.

1605.1.1 Stability. Regardless of which load combinations are used to design for strength, where overall structure stability (such as stability against overturning, sliding, or buoyancy) is being verified, use of the load combinations specified in ASCE 7 Section 2.3, ASCE 7 Section 2.4, and Section 1605.2 or 1605.3 shall be permitted. Where the load combinations specified in ASCE 7 Section 1605.2 or 1605.3 are used, strength reduction factors applicable to soil resistance shall be provided by a registered design professional. The stability of retaining walls shall be verified in accordance with Section 1807.2.3.

1605.3.2 1605.2 Alternative basic load combinations. In lieu of the basic load combinations specified in Load Combinations of ASCE 7 Section 1605.3 or 2.4, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following combinations. When using these alternative basic allowable stress load combinations that include wind or seismic loads, allowable stresses are permitted to be increased or load combinations reduced where permitted by the material chapter of this code or the referenced standards. For load combinations that include the counteracting effects of dead and wind loads, only two-thirds of the minimum dead load likely to be in place during a design wind event shall be used. When using allowable stresses that have been increased or load combinations that have been reduced as permitted by the material chapter of this code or the referenced standards, where wind loads are calculated in accordance with Chapters 26 through 31 of ASCE 7, the coefficient \( \omega \) in the following equations shall be taken as 1.3. For other wind loads, \( \omega \) shall be taken as 1. When allowable stresses have not been increased or load combinations have not been reduced as permitted by the material chapter of this code or the referenced standards, \( \omega \) shall be taken as 1. When using these alternative load combinations to evaluate sliding, overturning and soil bearing at the soil-structure interface, the reduction of foundation overturning from Section 12.13.4 in ASCE 7 shall not be used. When using these alternative basic load combinations for proportioning foundations for loadings, which include seismic loads, the vertical seismic load effect, \( E_V \), in Equation 12.4-4 of ASCE 7 is permitted to be taken equal to zero.

\[
\begin{align*}
D + L + (L_r \text{ or } S \text{ or } R) & \quad (\text{Equation 16-17}) \\
D + L + 0.6 \omega W & \quad (\text{Equation 16-18}) \\
D + L + 0.6 \omega W + S/2 & \quad (\text{Equation 16-19}) \\
D + L + S + 0.6 \omega W/2 & \quad (\text{Equation 16-20}) \\
D + L + S + E/1.4 & \quad (\text{Equation 16-21}) \\
0.9 D + E/1.4 & \quad (\text{Equation 16-22})
\end{align*}
\]

Exceptions:

1. Crane hook loads need not be combined with roof live loads or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m²) or less and roof live loads of 30 psf (1.44 kN/m²) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m²), 20 percent shall be combined with seismic loads.

3. Where required by ASCE 7 Chapters 12, 13, and 15, the Load Combinations including overstrength of ASCE 7 Section 2.3.6 shall be used.

Delete without substitution:

1605.2 Load combinations using strength design or load and resistance factor design. Where strength design or load and resistance factor design is used, buildings and other structures, and portions thereof, shall be designed to resist the most critical effects resulting from the following combinations of factored loads:

\[ D + F \]
\[ D + H + F + L \]
\[ D + H + F + 0.75(L + S) \]
\[ D + H + F + 0.75(L + S) \]
\[ D + H + F + 0.75(L + S) \]
\[ D + H + F + 0.75(L + S) \]
\[ D + H + F + 0.75(L + S) \]
\[ 0.6D + 0.6W + H \]
\[ 0.6(D + F) + 0.75E + H \]

where:

- ** Exceptions:**
  1. Where other factored load combinations are specifically required by other provisions of this code, such combinations shall take precedence.
  2. Where the effect of \( H \) resists the primary variable load effect, a load factor of 0.9 shall be included with \( H \) where \( H \) is permanent and \( H \) shall be set to zero for all other conditions.

1605.2.1 Other loads. Where flood loads, \( F_{\text{ar}} \), are to be considered in the design, the load combinations of Section 2.3.3 of ASCE 7 shall be used. Where self-straining loads, \( T \), are considered in design, their structural effects shall be combined with other loads in accordance with Section 2.3.5 of ASCE 7. Where an ice-sensitive structure is subjected to loads due to atmospheric icing, the load combinations of Section 2.3.4 of ASCE 7 shall be considered.

1605.3 Load combinations using allowable stress design.

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

\[ D + F \]
\[ D + H + F + L \]
\[ D + H + F + 0.75(L + S) \]
\[ D + H + F + 0.75(L + S) \]
\[ D + H + F + 0.75(L + S) \]
\[ D + H + F + 0.75(L + S) \]
\[ 0.6D + 0.6W + H \]
\[ 0.6(D + F) + 0.75E + H \]

* Exceptions:*

1. Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m²) or less and roof live loads of 30 psf (1.44 kN/m²) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m²), 20 percent shall be combined with seismic loads.
3. Where the effect of \( H \) resists the primary variable load effect, a load factor of 0.6 shall be included with \( H \) where \( H \) is permanent and \( H \) shall be set to zero for all other conditions.
4. In Equation 16-16, the wind load, \( W \), is permitted to be reduced in accordance with Exception 2 of Section 2.4.1 of ASCE 7.
5. In Equation 16-16, 0.6 \( D \) is permitted to be increased to 0.9 \( D \) for the design of special reinforced masonry shear walls complying with Chapter 21.

1605.3.1.1 Stress increases. Increases in allowable stresses specified in the appropriate material chapter or the referenced standards shall not be used with the load combinations of Section 1605.3.1, except that increases shall be permitted in
1605.3.2.1 - Other loads. Where F, H or T are to be considered in the design, each applicable load shall be added to the combinations specified in Section 1605.3.2. Where self-straining loads, T, are considered in design, their structural effects in combination with other loads shall be determined in accordance with Section 2.4.4 of ASCE 7.

Reason: Since 2000, the IBC has contained three separate groups of load combinations included the following: (1) Strength Load Combinations (1605.2); (2) Basic Allowable Stress Load Combinations (1605.3.1); and (3) Alternative Allowable Stress Load Combinations are transcribed directly from an earlier edition of the ASCE 7 Standard. The third set of combinations are a legacy from the codes that predate the IBC. This proposal is intended to remove minor discrepancies in requirements between the IBC and ASCE 7 standards version of the Strength and Basic Load Combinations by eliminating the duplication of this material. Further, it is hoped that removed of the Strength and Basic Load Combinations from the IBC will reduce the likelihood of design errors that many engineers have been making when applying the Basic Allowable Stress Combinations. The Alternative Allowable Stress Combinations permit the use of a 1/3 increase in allowable stresses when evaluating Load Combinations containing short-term transient loads including wind and seismic. The Basic Allowable Stress Combinations do not do this, but instead, apply a factor of 0.75 to the transient loads including live, snow, wind, and seismic, when more than one of these loadings is considered simultaneously. The ASCE 7 combinations further permit increases in allowable stresses only when the material, such as wood, has increased available strength under short-term loading as opposed to long-duration loading. These further increases are not intended to be used for the design of masonry, concrete, or steel structures when using the Basic Allowable Stress Load Combinations because the strength of these materials does not have significant duration dependence. Unfortunately, and despite specific commentary within the IBC to discourage this, many engineers routinely apply the 1/3 increase to all allowable stresses when designing using the Basic Allowable Stress Design Load Combinations. This creates a potentially dangerous situation in which safety margins of structures designed in this manner are substantially reduced.

By removing the transcriptions of the ASCE 7 Load Combinations from the IBC, in addition to avoiding duplication of nearly identical materials, we hope to reduce the likelihood that designers will misapply the 1/3 increase factor applicable to the Alternate Allowable Stress Load Combinations. With the adoption of this proposal, the IBC will refer to ASCE 7 for the Strength and Basic Allowable Stress Load Combinations, where there is no mention of the 1/3 increase factor. The Alternate Allowable Stress Load Combinations will remain in the IBC with the permissible 1/3 increase. It is important to note that this proposal does not result in any substantive technical change as all Load Combinations presently used by engineers will remain available to them. The requirement that engineers reference ASCE 7 to determine the Strength and Basic Allowable Stress Load Combinations is not burdensome as engineers already must reference ASCE 7 to compute the values of the various loading required for design.

Cost Impact: Will not increase the cost of construction

The proposed change will not impact the cost of construction. This proposal is a re-organization of the pointers in the IBC to refer to the load combinations in the referenced loading standard ASCE 7. ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures will be updated from the 2010 edition to the 2016 edition as an Administrative Update to the 2018 I-Codes.

As of the submission date of this code change proposal, the ASCE 7 Standards Committee has completed the committee balloting on the technical changes. The document designated ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures is expected to be completed, published, and available for purchase prior to the ICC Public Comment Hearings for Group B in October 2016. Any person interested in obtaining a public comment copy of ASCE 7-16 may do so by contacting James Neckel at ASCE (jneckel "at" asce.org).
Committee Action: Approved as Modified

Modification:

2015 International Building Code

1605.1 General. Buildings and other structures and portions thereof shall be designed to resist the Strength Load Combinations specified in ASCE 7 Section 2.3, the Allowable Stress Design Load Combinations specified in ASCE 7 Section 2.4 or the Alternative Allowable Stress Design Load Combinations of Section 1605.2.

Exception:

1. The modifications to the Load Combinations of ASCE 7 Section 2.3, ASCE 7 Section 2.4, and Section 1605.2 specified in ASCE 7 Chapter 18 and 19 shall apply.

2. When the allowable stress load combinations of ASCE 7 Section 2.4 are used, flat roof snow loads of 30 psf (1.44 kN/m²) and roof live loads of 30 psf (1.44 kN/m²) or less need not be combined with seismic load.

Committee Reason: The committee feels we don't need parallel efforts and is concerned with the maintenance of the duplicate load combination provisions. ASCE has committed making such information available online. The load combinations are not necessarily part of the core information that should be available for building officials in the code. The modification retains the exception relating to snow load.

Assembly Action: None

Individual Consideration Agenda

Proponent: Vincent Sagan, Thomas Associates, representing Metal Building Manufacturers Association (vsagan@mbma.com) requests Approve as Modified by this Public Comment.

Further Modify as Follows:

2015 International Building Code

1605.1 General. Buildings and other structures and portions thereof shall be designed to resist the Strength Load Combinations specified in ASCE 7 Section 2.3, the Allowable Stress Design Load Combinations specified in ASCE 7 Section 2.4 or the Alternative Allowable Stress Design Load Combinations of Section 1605.2.

Exceptions:

1. The modifications to the Load Combinations of ASCE 7 Section 2.3, ASCE 7 Section 2.4, and Section 1605.2 specified in ASCE 7 Chapter 18 and 19 shall apply.

2. When the allowable stress load combinations of ASCE 7 Section 2.4 are used, flat roof snow loads of 30 psf (1.44 kN/m²) and roof live loads of 30 psf (1.44 kN/m²) or less need not be combined with seismic load. Where flat roof snow loads exceed 30 psf (1.44 kN/m²), 20 percent shall be combined with seismic loads.

Commenter's Reason: Code Change Proposal S78-16 deleted the Section 1605.3, Load combinations using allowable stress design. The reason statement justified the deletion by stating it was removing duplication of material that is in the ASCE 7 Standard. However, Exception 2 in the IBC was not duplicated in ASCE 7. Floor Modification S78-16 Hamburger 2, added back part of an existing exception in the allowable stress load combinations, addressing when flat roof snow loads and roof live loads of 30 psf or less. The original exception included flat roof snow loads greater than 30 psf. This part of the exception should also be added back.

Proponent: Ed Berkel, representing ICC Code Correlation Committee (ccc@iccsafe.org) requests Disapprove.

Commenter's Reason: The Code Correlation Committee requests Disapproval of this code change proposal in order to bring a correlation issue to the attention of the full membership at the Public Comment Hearings and to allow the membership to coordinate action on this code change proposal with action taken on Code Change Proposal ADM94-16.
ADM94-16 is the administrative update to referenced standards in the I-Codes. One of these standards, ASCE7, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, was proposed for update to ASCE7-16. However, a successful assembly motion requests that the referenced ASCE7 remain at ASCE7-10 as it presently is referenced in the 2015 I-Codes. This code change proposal coordinates with and relies upon reference to ASCE7-16.

The Code Correlation Committee is a standing committee of the International Code Council whose objectives, procedures and organization are set forth in Council Policy CP#44-13. The objective of the Code Correlation Committee is to maintain technical and editorial consistency among the International Codes and to assist staff in the evaluation and processing of code change proposals and comments that are exclusively editorial.
2015 International Building Code

Add new text as follows:

1605.4 Structural integrity load combinations - alternate load path method Where specifically required by Sections 1615 through 1617, elements and components shall be designed to resist the forces calculated using the following combination of factored loads:

D + f1 L + f2 W  

Equation 16-23

Where:

f1 = 0.25 for buildings in Risk Category II.

f1 = 0.5 for buildings in Risk Category III or IV.

f2 = 0 for buildings in Risk Category II.

f2 = 0.33 for buildings in Risk Category III or IV.

The live load component f1L need not be greater than the reduced live load.

1605.5 Structural integrity load combinations - vehicular impact and gas explosions Where specifically required by Sections 1615.5 and 1615.6, elements and components shall be designed to resist the forces calculated using the following combination of factored loads:

1.2D + Ak (0.5L or 0.2S)  

Equation 16-24

0.9D + Ak + 0.2W  

Equation 16-25

where:

Ak is the load effect of the vehicular impact or gas explosion.

1605.6 Structural integrity load combinations - specific local resistance method. Where the specific local resistance method is used in a key element analysis, the specified local loads shall be used as specified in Section 1617.7.

SECTION 1615 STRUCTURAL INTEGRITY DEFINITIONS

1615.1 Definitions The following words and terms shall, for the purposes of this section, have the meanings shown herein.

ALTERNATE LOAD PATH. A secondary or redundant load path capable of transferring the load from one structural element to other structural elements.

ALTERNATE LOAD PATH METHOD. A design approach that accounts for an extreme event by providing alternate load paths for elements that are no longer able to carry load. In an alternate load path design, key elements are considered notionally removed, one at a time, and the structure is designed to transfer the loads from the removed element to other structural elements, as required by Section 1616.
ASPECT RATIO. The height of any portion of a building divided by its least dimension at the elevation from which the height is being measured.

COLLAPSE. Failure of a structural element to the extent that it can no longer support any load.

ELEMENT. A structural member or structural assembly.

KEY ELEMENT. An element of the structural system, including its connections, that meets one or more of the following criteria:

1. An element which when lost, results in more than local collapse.
2. An element that braces a key element, the failure of which results in failure of the key element (further secondary elements need not be considered key elements).
3. An element whose tributary area exceeds 3,000 square feet (279 square meters) on a single level.

LOCAL COLLAPSE. Failure of a structural element that results in the collapse of areas being directly supported by that element and not extending vertically more than three stories.

RESPONSE RATIO. The ratio of an ultimate response quantity (e.g., deflection) to its value at yield.

ROTATION. The angle, measured at the ends of a member, whose tangent is equal to the deflection of the member at mid-span divided by half the length of the member.

SPECIFIC LOCAL LOAD. A load applied to a structural element or structural system as specified in Section 1616.7.

SPECIFIC LOCAL RESISTANCE METHOD. A design approach that accounts for extreme event loads by providing sufficient strength for elements that may fail. In a specific local resistance design, key elements are designed for specific local loads as required by Section 1617.

SECTION 1616 STRUCTURAL INTEGRITY - PRESCRIPTIVE REQUIREMENTS

1616.1 Scope. The intent of these provisions is to enhance structural performance under extreme event scenarios by providing additional overall system redundancy and local robustness. All structures shall be designed to satisfy the prescriptive requirements of this Section.

Exception: Structures in Risk Category I of Table 1604.5 and structures in occupancy group R-3 are exempt from the requirements of Sections 1615 through 1617.

1616.2 Continuity and Ties. All structural elements shall have a minimum degree of continuity and shall be tied together horizontally and vertically as specified in Chapter 19, Chapter 21, and Chapter 22, for concrete, masonry and steel, respectively.

1616.3 Lateral Bracing. Floor and roof diaphragms or other horizontal elements shall be tied to the lateral load-resisting system.

1616.4 Vehicular Impact. Structural columns that are directly exposed to vehicular traffic shall be designed for vehicular impact. Structural columns that are adequately protected by bollards, guard walls, vehicle arrest devices or other elements do not need to be designed for vehicular impact. The load combinations for vehicular impact shall be as specified in Section 1605.5.
Specific loads for vehicular impact shall be as follows:

1. Exterior corner columns shall be designed for a concentrated load of 40 kips applied horizontally in any direction from which a vehicle can approach at a height of either 18 inches (457 mm) or 36 inches (914 mm) above the finished driving surface, whichever creates the worst effect.

2. All other exterior columns exposed to vehicular traffic, and columns within loading docks, and columns in parking garages along the driving lane shall be designed for a concentrated load of 20 kips applied horizontally in any direction from which a vehicle can approach at a height of either 18 inches (457 mm) or 36 inches (914 mm) above the finished driving surface, whichever creates the worst effect.

1616.5 Gas Explosions In buildings with gas piping operating at pressures in excess of 15 psig (103 kPa gauge), all key elements and their connections within 15 feet (4572 mm) of such piping shall be designed to resist a potential gas explosion. The structure shall be designed to account for the potential loss of the affected key elements one at a time by the alternate load path method. Load combinations for the alternate load path shall be as specified in Section 1605.4. In lieu of the alternate load path method, the affected key elements shall be designed to withstand a load of 430 psf (20.6 kPa) applied using the load combinations specified in Section 1605.5. The load shall be applied along the entire length of the element, and shall be applied in the manner and direction that produces the most damaging effect.

Exceptions:

1. If a structural enclosure designed to resist the specified pressure is provided around the high-pressure gas piping, only the key elements within the structural enclosure need to comply with this section.
2. A reduced pressure for gas explosions can be used based on an engineering analysis approved by the building official.

1616.5.1 Explosion Prevention and Deflagration Venting The structural design and installation of explosion prevention systems and deflagration venting shall be in accordance with the requirements of Appendices E and G of the International Fuel Gas Code, as well as the International Fire Code, and the rules and regulations of the building official.

CHAPTER PART 1617 — STRUCTURAL INTEGRITY - KEY ELEMENT ANALYSIS

1617.1 Scope A Key Element Analysis shall be performed for the following buildings:

1. Buildings included in Structural Occupancy Category IV as defined in this chapter.
2. Buildings with the aspect ratios of seven or greater.
3. Buildings greater than 600 feet (183 m) in height or more than 1,000,000 square feet (92 903 m2) in gross floor area.
4. Buildings taller than seven stories where any element, except for walls greater than 10 feet (3.048 meters) in length, supports in aggregate more than 15 percent of the building area.
5. Buildings designed for areas with 3,000 or more occupants in one area in close proximity, including fixed seating and grandstand areas.
6. When specifically ordered by the building official.

1617.2 Load Combinations Where specifically required by Section 1617.1, elements and components shall be designed to resist the forces calculated using the combination specified in Section 1605.4 or 1605.6 as applicable.

1617.3 Seismic and Wind When the code-prescribed seismic or wind design produces greater effects, the seismic or wind design shall govern, but the detailing requirements and limitations prescribed in this and referenced sections shall also be followed.

1617.4 Joints Where a structure is divided by joints that allow for movement, each portion of the structure between joints shall be considered as a separate structure.

1617.5 Key Element Analysis Where key elements are present in a structure, the structure shall be designed to account for their potential loss one at a time by the alternate load path method, or by the specific local resistance method as specified in Section 1617.6.

1617.6 The Specific Local Resistance Method Where the specific local resistance method is used key elements shall be designed using specific local loads as follows:
1. Each compression element shall be designed for a concentrated load equal to 2 percent of its axial load but not less than 15 kips, applied at midspan in any direction, perpendicular to its longitudinal axis. This load shall be applied in combination with the full dead load and 50 percent of the live load in the compression element.

2. Each bending element shall be designed for the combination of the principal acting moments plus an additional moment, equal to 10 percent of the principal acting moment applied in the perpendicular plane.

3. Connections of each tension element shall be designed to develop the smaller of the ultimate tension capacity of the member or three times the force in the member.

4. All structural elements shall be designed for a reversal of load. The reversed load shall be equal to 10 percent of the design load used in sizing the member.

1617.7 Design Criteria Alternate load path method and/or specific local resistance method for key elements shall conform to the appropriate design criteria as determined from Sections 1617.8, 1617.9 and 1617.10. Load combinations for the alternate load path method shall be as specified in Section 1605.4.

1617.8 Analysis Procedures All structural analysis for specific local loads or alternate load paths shall be made by one of the following methods:

1617.8.1 Static Elastic Analysis For analysis of this type, dynamic effects of member loss or dynamic effects of specific local loads need not be considered. The structural demand is obtained from linear static analysis. However, structural member capacity is based on ultimate capacity of the entire cross section. The demand/capacity ratio of structural elements shall not exceed one.

1617.8.2 Dynamic Inelastic Analysis For analysis of this type, dynamic effects of member loss or specific local loads shall be considered. The structure does not need to remain elastic; however, the response ratio and rotation limits obtained from Table 1617.8 shall not be exceeded.

1617.8.3 Energy methods Static inelastic analysis using energy equilibrium may also be used. The structure does not need to remain elastic; however, the response ratio and rotation limits obtained from Table 1617.8 shall not be exceeded.

<table>
<thead>
<tr>
<th>TABLE 1617.8 Response Ratio and Rotation Limits a</th>
</tr>
</thead>
<tbody>
<tr>
<td>Element</td>
</tr>
<tr>
<td>Concrete Slabs</td>
</tr>
<tr>
<td>Post-Tensioned Beams</td>
</tr>
<tr>
<td>Concrete Beams</td>
</tr>
<tr>
<td>Concrete Columns</td>
</tr>
<tr>
<td>Long Span Acoustical Deck</td>
</tr>
<tr>
<td>Open Web Steel Joists</td>
</tr>
<tr>
<td>Steel Beams</td>
</tr>
<tr>
<td>Steel Columns</td>
</tr>
</tbody>
</table>

For SI: 1 degree = 0.01745 rad.

Table 1617.8 is intended for SDOF and simplified MDOF response calculations and a low level of protection. Table 1617.8 does not apply for explicit finite element methods that calculate the performance of the structural elements in response to the specified loading intensity.

1617.9 Minimum Response Structural response of elements determined using a dynamic inelastic analysis shall not be less than 80 percent of the structural response determined using a static elastic analysis.

1617.10 Strength Reduction Factors For structural design for specific local loads or alternate load paths, all strength reduction factors may be taken as one.

SECTION 1909 STRUCTURAL INTEGRITY REQUIREMENTS

1909.1 General Reinforced concrete structures shall meet all the requirements of Sections 1909.1 through 1909.3. Concrete slabs on metal deck shall be governed by the provisions of Chapter 22. Reinforcement provided for gravity, seismic and wind forces or for other purposes may be regarded as forming part of, or the whole of, these requirements. Reinforcing provided for one requirement may be counted towards the other requirements.
1909.2 Continuity and Ties  The structural integrity requirements of ACI 318, Section 13.3.8.5 and 7.13 shall apply. In addition, the following requirements shall be met.

1909.2.1 Slab Reinforcement  At all floor and roof levels, slabs shall have a mat of bottom reinforcement in two perpendicular (or roughly perpendicular) directions. Reinforcement in this bottom mat shall be made continuous with lap, mechanical or welded tension splices.

1909.2.1.1 Bottom Mat Reinforcement  In each direction, the bottom mat reinforcement shall be not less than the steel required for temperature reinforcement. The bottom mat reinforcement shall be anchored at discontinuous edges within the column strip, reentrant corners, elevation changes and anywhere else the continuity of the reinforcing is interrupted.

Exception: Flat plate middle strip bottom mat reinforcing perpendicular to discontinuous slab edges. In addition, the main bottom mat reinforcement in one-way slabs shall be anchored at discontinuous edges.

1909.2.2 Peripheral Ties  At each floor and roof level, reinforcement forming a continuous peripheral tie shall be provided. Peripheral ties shall be located within perimeter beams or walls, where they occur, or within 4 feet (1219 mm) of the edge of slab, where perimeter beams or walls do not occur. Continuous tie reinforcement shall be equal to half of the bottom reinforcement within the edge or edge strip for two-way slabs but not less than two bars.

1909.2.3 Horizontal Ties  At each column, beam reinforcement or slab bottom reinforcement shall be provided at each level that can develop a tension force equal to the maximum of Item 1 or 2:

1. Three times the load entering the column at that level, using a load combination of 1.0 x DL (self weight of structure only).
2. One and a half times the load entering the column at that level using the load combinations of (1.2 DL + 1.6 LL) or 1.4 DL.
3. For transfer elements only, in lieu of Item 1 or 2 the horizontal reinforcement shall be anchored at all supports.

1909.2.3.1 Bottom Reinforcing  This beam or slab bottom reinforcement shall be distributed around the column perimeter and shall be extended on all sides of the column into the adjacent slab for at least one-third of the span length. Where reinforcing bars cannot be extended beyond the column (e.g., at slab edges and openings), they shall be hooked or otherwise developed within the column.

1909.2.4 Vertical Ties  Each column and each wall carrying vertical load shall be vertically tied continuously from its lowest to highest level. The vertical ties composed of vertical column reinforcement shall be capable of resisting a tensile force equal to the maximum design dead and live load received by the column or wall from any one story within four floors below.

1909.3 Precast Concrete General  Precast concrete structural elements shall be reinforced to meet all of the requirements of this section. However, reinforcement provided for gravity, seismic and wind forces and for other purposes may be regarded as forming part of, or the whole of, these requirements. Reinforcing provided for one requirement may be counted towards the other requirements.

1909.3.1 Continuity and Ties  The structural integrity requirements of ACI 318, Section 16.5, shall apply. In precast and composite structures, ties within precast structural elements shall be continuous and shall be anchored to the supporting structure. In addition to Sections 1909.2.2 and 1909.2.4, the following requirements shall be met.

1909.3.1.1 End Connections  End connections of all precast slabs, beams and girders shall have an axial tension capacity equal to the larger of the vertical shear capacity of the connection at either end, or at least 2 percent of the maximum factored vertical dead and live load in the precast compression element, whichever is larger, but not less than 20 kips or 2,500 pounds per linear foot of slab (36.48 kN/m). Where more than one element frames in one direction, none of the elements or connections shall have an axial tension capacity of less than 1 percent of the column load but not less than 20 kips.

1909.3.1.2 Side Connections  Side connections of all precast elements shall have an axial tension capacity not less than the steel required for temperature reinforcement of the larger element at either side.

1909.3.1.3 Connection Forces  For design of the connections, the transverse shear force and the axial tensile force need not be considered to act simultaneously.

1909.3.2 Joints  Joints in precast structures shall not rely on friction due to gravity to transfer load.
1909.3.3 Bearing  The net bearing area shall not be less than 2 inches (51 mm) wide and 3 inches (76 mm) long in the direction of the member.

SECTION 2114 STRUCTURAL INTEGRITY REQUIREMENTS

2114.1 General Load-bearing masonry structures shall be reinforced to meet all of the requirements of this section. However, reinforcement provided for gravity, seismic or wind forces or for other purposes may be regarded as satisfying part of, or the whole of, these requirements. Reinforcement provided for one requirement may be counted towards the other requirements.

2114.2 Continuity and Ties Load-bearing masonry structures shall be reinforced to obtain a continuous system of vertical and horizontal ties. Continuity of all ties shall be ensured by providing lap, welded or mechanical tension splices. The following requirements shall be met for walls, columns and piers:

2114.2.1 Horizontal Ties At each floor and roof level, continuous horizontal ties shall be provided in all load-bearing masonry walls, and around the perimeter of the building. Minimum horizontal tie reinforcement shall be not less than the equivalent of two No. 4 bars.

2114.2.1.1 Location of Horizontal Ties Ties shall be located within the thickness of walls or beams, where they occur, or within 1 foot (305 mm) of the edge of slab, where walls or beams do not occur.

2114.2.1.2 End Connections of Horizontal Ties All horizontal ties shall be terminated in a perpendicular horizontal tie. Where no perpendicular horizontal tie exists within 4 feet (1219 mm) of the end of a wall, the horizontal tie shall be anchored at the end of the wall. The vertical reinforcement at the end of such walls shall not be less than two No. 4 bars placed within 16 inches (406 mm) of the end of the wall. This vertical reinforcement shall be continuous from the lowest to highest level of the wall, and anchored at each end in a horizontal tie or the foundation element.

2114.2.2 End Connections Where slab or beam elements are supported on a masonry wall, column or pier, the connection shall be designed to sustain an axial tension capacity equal to the greater of the vertical shear capacity of the connected element at either end or two percent of the maximum factored vertical dead and live load in the compression masonry element. The design of the end connections shall ensure the transfer of such loads to horizontal or vertical ties.

Where more than one element frames in one direction, none of the elements or connections shall have an axial tension capacity of less than one percent of the vertical load.

For the design of the connections, the transverse shear force and the axial tensile force need not be considered to act simultaneously.

The reinforcement of the end connections shall be equivalent to at least one No. 4 bar, at a maximum spacing of 24 inches (610 mm) on center. Where end connections occur at a masonry pier or column, reinforcement equivalent to a minimum of four fully developed No. 4 bars shall be provided. The reinforcement shall be distributed around the perimeter of the column or pier. The minimum anchorage into both the slab and the masonry compression element shall be equivalent to the capacity of the fully developed No. 4 bar.

Where the floor extends on both sides of a bearing wall, the portion of the tie within the slab shall alternate between both sides.

2114.2.3 Vertical Ties Each column, pier and wall shall be vertically tied continuously from its lowest to highest level. The vertical reinforcement shall be terminated in a horizontal tie or foundation or their equivalent. Where openings in bearing walls greater than 24 inches (610 mm) in height occur, ties shall be provided at each side of the opening that extend and are anchored in the masonry above and below the opening. Vertical ties shall be placed on both sides of control joints in bearing walls.

2114.2.3.1 Vertical Ties Reinforcing Vertical tie reinforcing shall not be less than the equivalent of one No. 4 bar, at a maximum spacing of 48 inches (1219 mm) on center. A minimum of four continuous No. 4 bars shall be provided per masonry column or pier.

SECTION 2212 STRUCTURAL INTEGRITY REQUIREMENTS

2212.1 General Steel structures shall be designed to meet all of the requirements of this section. However, details provided for gravity, seismic and wind forces and for other purposes may be regarded as forming part of, or the whole of, these requirements. Detailing provided for one requirement may be counted towards the other requirements.
Exceptions:

1. One-story structures less than 5,000 square feet (465 m²) not to exceed 15 feet (4572 mm) in height.
2. Structures in Group R-3 occupancy not more than three stories in height.

2212.2 Continuity and Ties The requirements of this section shall be met.

2212.2.1 Bolt Quantity All bolted connections shall have at least two bolts.

2212.2.2 Bolt Pretension Bolted connections of all columns, beams, braces and other structural elements that are part of the lateral load resisting system shall be designed as bearing-type connections with pretensioned bolts or as slip critical connections.

2212.2.3 Connection Tensile Strength End connections of all beams and girders shall have a minimum available tensile strength equal to the larger of the available vertical shear strength of the connections at either end, but not less than 10 kips (45 kN). For the design of the connections, the shear force and the axial tensile force need not be considered to act simultaneously. For the purpose of satisfying these integrity provisions only, bearing bolts in connections with short-slotted holes parallel to the direction of the tension force and inelastic deformation are permitted. For the purpose of this provision, a connection shall be considered compliant if it meets the following requirements:

1. For single-plate shear connections, the available tensile strength shall be determined only for the limit state of bolt bearing on the plate and beam web.
2. For single angle and double angle shear connections, the available tensile strength shall be determined for the limit states of bolt bearing on the angles and beam web and for tension yielding on the gross area of the angles.
3. All other connections shall be designed for the required tension force noted above in accordance with the provisions of AISC 360.

2212.2.4 Brace Strength Elements and their connections that brace compression members shall have a minimum available tensile strength equal to at least 2 percent of the required compressive strength of the member being braced, but not less than 10 kips (45 kN). For design of these bracing connections, the shear force and the tensile force need not be considered to act simultaneously. Where more than one element braces a compression member at a point in one direction, all elements and connections shall have a minimum available tensile strength equal to at least 1 percent of the required compressive strength of the member being braced but not less than 10 kips (45 kN).

2212.3 Composite Construction For steel framing members and/or decking acting compositely with concrete slabs, the following requirements shall be met:

1. Shear studs shall not be less than 1/2 inch (12.7 mm) in diameter. The spacing of shear studs shall not be greater than one every 12 inches (305 mm) averaged over the length of the beam.
2. Connections at the discontinuous edges of permanent metal decking to supporting members shall have a minimum connection strength in the direction parallel to the rib of the deck equal to the shear strength of a 3/4-inch (19.1 mm) puddle weld every 12 inches (305 mm) on center.
3. Side lap connections of permanent metal decking shall have a minimum strength equal to the strength of a button punch every 24 inches (610 mm) on center.
4. Welded wire fabric reinforcement in concrete slabs shall be continuous over all supports and in all spans. Minimum area of continuous reinforcement shall be 0.0015 times the area of concrete. The welded wire fabric reinforcement shall have tension splices and be anchored at discontinuous edges.

Reason: Structures should be designed to avoid disproportionate collapse. Typical structures should have minimal prescriptive requirements. Significant structures should be designed with additional resiliency.

Cost Impact: Will increase the cost of construction
Will have a nominal increase for typical structures subject to the prescriptive requirements and a small increase for significant structures. Benefit to society should outweigh the very slight increase in construction cost.

Public Hearing Results
Committee Reason: This proposal contains a lot of information that is unclear. A suggested modification was too complex to understand what was or was not included in its scope. The committee encourages the proponent to continue to work with the ASCE committee through their consensus process.

Assembly Action: None

Individual Consideration Agenda

Proponent: Scott Campbell, representing Portland Cement Association (scampbell@cement.org) requests Approve as Submitted.

Commenter's Reason: Including specific provisions for structural integrity design will be beneficial for public safety and property protection by ensuring design of critical elements to all applicable loads or by provision of an alternate load path. The proposed code change is both comprehensive and clear, and will achieve the goal of providing for adequate structural integrity for buildings subject to the requirements.

S81-16
S88-16
IBC: 1607.4, 1607.9.3.

Proposed Change as Submitted

Proponent: Jennifer Goupil, AMERICAN SOCIETY OF CIVIL ENGINEERS, representing SELF (jgoupil@asce.org)

2015 International Building Code

Revise as follows:

1607.4 Concentrated live loads. Floors, roofs, and other similar surfaces shall be designed to support the uniformly distributed live loads prescribed in Section 1607.3 or the concentrated live loads, given in Table 1607.1, whichever produces the greater load effects. Unless otherwise specified, the indicated concentration shall be assumed to be uniformly distributed over an area of $2^{1/2}$ feet by $2^{1/2}$ feet (762 mm by 762 mm) and shall be located so as to produce the maximum load effects in the structural members.

1607.9.3 Elements supporting hoists for façade access equipment. In addition to any other applicable live loads, structural elements that support hoists for façade access equipment shall be designed for a live load consisting of the larger of the rated load of the hoist times 2.5 and or the stall load of the hoist, whichever is larger.

Reason: This proposed changes to Section 1607 will harmonize the provision in the code with the 2016 edition of the referenced loading standard ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7-16).

Section 1607.4 Concentrated live loads - Proposed addition specifically includes roofs to the requirements for concentrated live loads. This proposed change will align the requirements of the code with the standard ASCE 7.

Section 1607.9.3 Elements supporting hoists for facade access equipment - Proposed revisions clarifies that the larger of the two loads is required, not both.

Cost Impact: Will not increase the cost of construction
The proposed changes will not impact the cost of construction. This proposal coordinates the IBC with the referenced loading standard ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE 7 will be updated from the 2010 edition to the 2016 edition as an Administrative Update to the 2018 I-Codes.

As of the submission date of this code change proposal, the ASCE 7 Standards Committee has completed the committee balloting on technical changes. The document is designated as ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures is expected to be completed, published, and available for purchase prior to the ICC Public Comment Hearings for Group B in October of 2016. Any person interested in obtaining a public comment copy of ASCE 7-16 may do so by contacting James Neckel at ASCE (jneckel “at” asce.org).

S88-16 : 1607.4- GOUPIL12934

Public Hearing Results

Committee Action: Approved as Modified
Modification:
2015 International Building Code

1607.9.3 Elements supporting hoists for façade access and building maintenance equipment. In addition to any other applicable live loads, structural elements that support hoists for façade access and building maintenance equipment shall be designed for a live load consisting of the larger of 2.5 times the rated load of the hoist times 2.5 or the stall load of the hoist, whichever is larger.

Committee Reason: Coordination with the latest edition of the referenced standard, ASCE 7 which was updated in ADM94-16. The modification further updates the proposal for consistency with ASCE 7 due to public comments.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:
Proponent: Gwenyth Searer, representing self requests Approve as Modified by this Public Comment.

Further Modify as Follows:

2015 International Building Code

1607.9.4 Lifeline Fall arrest and lifeline anchorages for façade access equipment. In addition to any other applicable live loads, fall arrest and lifeline anchorages and structural elements that support these anchorages shall be designed for a live load of at least 3,100 pounds (13.8 kN) for each attached lifeline, in every direction that a fall arrest load may be applied.

Commenter’s Reason: During the public comment period of ASCE 7-2016, a commenter suggested editorial changes to the language regarding personal fall arrest anchorages. The Committee accepted the proposed changes, but only after the deadline for submission of code change proposals.

This public comment matches the changes made by ASCE 7.

The original wording of the section reads:

"1607.9.4 Lifeline anchorages for façade access equipment. In addition to any other applicable live loads, lifeline anchorages and structural elements that support lifeline anchorages shall be designed for a live load of at least 3,100 pounds (13.8 kN) for each attached lifeline in every direction that a fall arrest load may be applied."

If this public comment is accepted, the section will be revised to read:

"1607.9.4 Fall arrest and lifeline anchorages. In addition to any other applicable live loads, fall arrest and lifeline anchorages and structural elements that support these anchorages shall be designed for a live load of at least 3,100 pounds (13.8 kN) for each attached lifeline in every direction that a fall arrest load may be applied."

This public comment will bring the language into compliance with the language approved for ASCE 7-16 and matches the changes made to 1607.9.3 in S88-16, which was approved As Modified by the ICC Structural Committee.

S88-16
Proposed Change as Submitted

Proponent: Karl Rubenacker, representing Codes & Standards Committee, Structural Engineer's Association of New York (karl.rubenacker@gmsllp.com)

2015 International Building Code

Add new text as follows:

1607.8.3.1 Columns in parking areas. Unless specially protected, columns in parking areas subject to impact of moving vehicles shall be designed to resist the lateral load due to impact and this load shall be considered a variable load. For passenger vehicles, this lateral load shall be taken as a minimum of 6,000 pounds (26.70 kN) applied at least 1 foot 6 inches (457mm); above the roadway, and acting simultaneously with other design loads. In addition, columns in parking areas shall meet the requirements of Section 1615 for structural integrity.

Reason: Clarification of design load requirement for columns in parking areas, this load case should not be neglected in design of garages.

Cost Impact: Will not increase the cost of construction
Will not increase cost of construction - is clarification of design loading.

Committee Action: Disapproved
Committee Reason: Disapproval was requested by proponent in recognition that further work is needed on the proposal. There was insufficient justification provided for the proposed loads on columns in parking areas.

Assembly Action: None

Individual Consideration Agenda

Proponent: Scott Campbell, representing Portland Cement Association (scampbell@cement.org) requests Approve as Submitted.

Commenter's Reason: The effects of impact loads on columns in parking garages should be considered in the design. The current proposal clarifies the load case.
S93-16
IBC: 1607.12.3.1.

Proposed Change as Submitted

Proponent: Jennifer Goupil, AMERICAN SOCIETY OF CIVIL ENGINEERS, representing SELF (jgoupil@asce.org)

2015 International Building Code

Revise as follows:

1607.12.3.1 Vegetative and landscaped roofs. The weight of all landscaping materials shall be considered as dead load and shall be computed on the basis of saturation of the soil as determined in accordance with ASTM E 2397 - Section 3.1.4 of ASCE 7. The uniform design live load in unoccupied landscaped areas on roofs shall be 20 psf (0.958 kN/m^2). The uniform design live load for occupied landscaped areas on roofs shall be determined in accordance with Table 1607.1.

Reason: This proposed changes to Section 1607 will harmonize the provision in the code with the 2016 edition of the referenced loading standard ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7-16).

Section 1607.12.3.1 Vegetative and landscaped roofs - This modifies the pointer to Chapter 3 of ASCE 7 from ASTM E2397. ASCE 7 is the referenced loading standard and contains provisions for these types of use areas; the 2016 edition of ASCE 7 includes loads associated with landscaped and vegetative roofs maintained by irrigation and subject to rainfall. The provisions clarify which components are considered dead load versus rain load for vegetative roof areas, and clearly outlines the minimum live loads in Table 4-1 of ASCE 7, which now includes a section for occupiable areas of roofs.

Cost Impact: Will not increase the cost of construction

The proposed changes will likely impact the design and construction of these systems because this proposal seeks to reference the recognized loading standard ASCE 7, which now includes loading provisions for this use type. Whether costs increase or decrease or otherwise may impact design costs are undetermined because the current ASTM reference may or may not be similar to the acceptable ASCE 7 loading methodology, which is consistent with all other provisions in ASCE 7. This proposal coordinates the IBC with the referenced loading standard ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE 7 will be updated from the 2010 edition to the 2016 edition as an Administrative Update to the 2018 I-Codes.

As of the submission date of this code change proposal, the ASCE 7 Standards Committee has completed the committee balloting on technical changes. The document is designated ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures is expected to be completed, published, and available for purchase prior to the ICC Public Comment Hearings for Group B in October of 2016. Any person interested in obtaining a public comment copy of ASCE 7-16 may do so by contacting James Neckel at ASCE (jneckel "at" asce.org).

S93-16:
1607.12.3.1-
GOUPIL12937

Public Hearing Results

Committee Action: Approved as Submitted

Committee Reason: This proposal provides a direct reference to the ASCE 7 load provision for landscaped and vegetative roofs. As the reason indicates this will clarify which components are considered dead load versus rain load for vegetative roof areas,

Assembly Action: None

Individual Consideration Agenda

Proponent: Ed Berkel, representing ICC Code Correlation Committee (ccc@icc.org) requests Disapprove.

Commenter's Reason: The Code Correlation Committee requests Disapproval of this code change proposal in order to bring a correlation issue to the attention of the full membership at the Public Comment Hearings and to allow the membership to coordinate action on this code change proposal with action taken on Code Change Proposal ADM94-16.
ADM94-16 is the administrative update to referenced standards in the I-Codes. One of these standards, ASCE7, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, was proposed for update to ASCE7-16. However, a successful assembly motion requests that the referenced ASCE7 remain at ASCE7-10 as it presently is referenced in the 2015 I-Codes. This code change proposal coordinates with and relies upon reference to ASCE7-16.

The Code Correlation Committee is a standing committee of the International Code Council whose objectives, procedures and organization are set forth in Council Policy CP#44-13. The objective of the Code Correlation Committee is to maintain technical and editorial consistency among the International Codes and to assist staff in the evaluation and processing of code change proposals and comments that are exclusively editorial.
Proposed Change as Submitted

Proponent: Joseph Cain, SunEdison, representing Solar Energy Industries Association (SEIA) (joecainpe@aol.com)

2015 International Building Code

Revise as follows:

1607.12.5.1 Roof live load. Roof surfaces assemblies and supporting structures to be covered by solar photovoltaic panels or modules shall be designed for the roof live load, \( L_r \), assuming that for the photovoltaic panels or modules load case where photovoltaic panel systems are not present. The roof photovoltaic live load in need not be applied to roof areas covered by solar photovoltaic panels or modules shall be in addition to the panel loading unless the area covered by each solar photovoltaic panel or module is inaccessible. Areas photovoltaic panels where the clear space vertical height between the underside of the panels and the rooftop is not more than 24 inches (610 mm) shall be considered inaccessible or less. Roof surfaces assemblies and supporting structures not covered by photovoltaic panels shall be designed for the roof live load.

Reason: This proposal includes only editorial changes to clarify the existing requirements. Language is revised to clarify that roof assemblies and supporting structures are designed, not “roof surfaces are designed.” Language is revised to clarify that the live load threshold criteria is related to clear vertical height between the underside of the photovoltaic panels and the roof surface, rather than a determination of “accessible” or “inaccessible.” The term accessible is a defined term in the International Building Code. The definition of accessible is: “A site, building, facility or portion thereof that complies with Chapter 11.” It is inappropriate to use this defined term for the live load threshold. Clear vertical height is much more descriptive and clear.

Cost Impact: Will not increase the cost of construction

The proposal will not change the cost of construction because there is no change to technical requirements. These are editorial changes only.

Committee Reason: This code change improves the current wording of roof live loads at photovoltaic panels. The modification incorporates some additional wording from the referenced standard, ASCE 7, that is preferred.

Public Hearing Results

Modification:

2015 International Building Code

1607.12.5.1 Roof live load. Roof assemblies and supporting structures to be covered by solar panel systems shall be designed for to resist each of the following conditions:

1. The uniform and concentrated roof live load, \( L_r \), for loads with the load case where photovoltaic panel systems are not present photovoltaic panel system dead loads.

EXCEPTION: The roof live load need not be applied to roof areas the area covered by photovoltaic panels where the clear space vertical height between the underside of the panels and the rooftop roof surface is 24 inches in. (610 mm) or less. Roof assemblies.

2. The uniform and supporting structures not covered by photovoltaic panels shall be designed for the concentrated roof live loads without the photovoltaic panel system present.

Committee Reason: This code change improves the current wording of roof live loads at photovoltaic panels. The modification incorporates some additional wording from the referenced standard, ASCE 7, that is preferred.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Jonathan Siu, representing Washington Association of Building Officials Technical Code Development Committee (Jon.Siu@seattle.gov) requests Approve as Modified by this Public Comment.

Further Modify as Follows:
1607.12.5.1 Roof live load. Roof structures that support photovoltaic panel systems shall be designed to resist each of the following conditions:

1. **The All applicable** uniform and concentrated roof live loads with the photovoltaic panel system dead loads.
   
   **Exception:** The roof live load loads need not be applied to the area covered by photovoltaic panels where the clear space between the panels and the roof surface is 24 in. (610 mm) or less.

2. **The All applicable** uniform and concentrated roof live loads without the photovoltaic panel system present.

**Commenter's Reason:** This public comment further clarifies the design load requirements for roof structures supporting PV panel systems. The supporting structure needs to be designed for more than just the panel dead loads and roof live load. Snow, rain (which can lead to ponding), wind, and seismic loads also need to be accounted for. Mentioning only roof live loads can be misleading. Even though Section 1607 is titled Live Loads, there is precedence to point to other types of loading in Section 1607.12.4, which refers to snow and wind loads for awning and canopy design.
Committee Action: Approved as Modified

Proponent: Jennifer Goupil, AMERICAN SOCIETY OF CIVIL ENGINEERS, representing SELF (jgoupil@asce.org)

2015 International Building Code

Add new text as follows:

1607.12.5.2.1 Photovoltaic panels installed on open grid roof structures Structures with open grid framing and no roof deck or sheathing supporting photovoltaic panel systems shall be designed to support the uniform and concentrated roof live loads specified in Section 1607.12.3.1, except that the uniform roof live load shall be permitted to be reduced to 12 psf (0.57kN/m²).

Reason: This proposed changes to Section 1607 will harmonize the provision in the code with the 2016 edition of the referenced loading standard ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7-16).

Reference:

Cost Impact: Will not increase the cost of construction

The proposed changes may or may not impact the cost of construction. This proposal standardizes this evolving industry where requirements did not previously exist so that the loading and design requirements are consistent. This proposal coordinates the IBC with the referenced loading standard ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE 7 will be updated from the 2010 edition to the 2016 edition as an Administrative Update to the 2018 I-Codes.

As of the submission date of this code change proposal, the ASCE 7 Standards Committee has completed the committee balloting on technical changes. The document is designated ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures is expected to be completed, published, and available for purchase prior to the ICC Public Comment Hearings for Group B in October of 2016. Any person interested in obtaining a public comment copy of ASCE 7-16 may do so by contacting James Neckel at ASCE (jneckel "at" asce.org).

Public Hearing Results

Committee Action: Approved as Modified

Modification:

2015 International Building Code

1607.12.5.2.1 Photovoltaic panels installed on open grid roof structures Structures with open grid framing and no roof deck or sheathing supporting photovoltaic panel systems shall be designed to support the uniform and concentrated roof live loads specified in Section 1607.12.3.1, except that the uniform roof live load shall be permitted to be reduced to 12 psf.
Committee Reason: This proposal adds roof load requirements for photovoltaic panels that are taken from the latest edition of the referenced standard, ASCE 7, which was updated in ADM94-16. The modification corrects a mistaken section reference.

Individual Consideration Agenda

Public Comment 1:

Proponent: Jennifer Goupil, American Society of Civil Engineers (ASCE), representing American Society of Civil Engineers (ASCE) (jgoupil@asce.org) requests Approve as Modified by this Public Comment.

Further Modify as Follows:

2015 International Building Code

1607.12.5.2.1 Photovoltaic panels installed on open grid roof structures with open grid framing and no roof deck or sheathing supporting photovoltaic panel systems shall be designed to support the uniform and concentrated roof live loads specified in Section 1607.12.5.1, except that the uniform roof live load shall be permitted to be reduced to 12 psf (0.57 kN/m²).

Commenter’s Reason: This clarifies that the section specifically refers to roof structures. At the request of the committee, this public comment clarifies the intent with the descriptor word “roof” in the text of the provisions since titles are not enforceable.
Committee Action: Disapproved

Assembly Action: None

S101-16:

**Proposed Change as Submitted**

**Proponent:** Ali Fattah, City of San Diego Development Services Department (afattah@sandiego.gov)

**2015 International Building Code**

Add new text as follows:

**706.2 Structural stability** Fire walls shall be designed and constructed to allow collapse of the structure on either side without collapse of the wall under fire conditions. Fire walls designed and constructed in accordance with NFPA 221 and Section 1607.14.2 shall be deemed to comply with this section.

**1607.14.2 Fire walls.** Fire walls and their supports shall be designed in accordance with this chapter. In addition, where fire walls are designed in accordance with NFPA 221, they shall be designed to withstand a minimum out-of-plane Allowable Stress Design load of 5 lbs/ft$^2$ (0.38 kPa), applied from either direction. A minimum out-of-plane uniform Strength Design load, $A_k$, of 8 lbs/ft$^2$ (0.38 kPa), in conjunction with the load combinations for extraordinary events in ASCE 7, is permitted to be used for design.

**Reason:** This is a necessary code change for consistency in the design of fire walls for structural loads. This code change was submitted as public comment to FS28-15 that was disapproved in Group A in Long Beach. This code change addresses the design of fire walls and does not address the loading on fire walls during an E119 fire test.

The code change is proposed in Chapter 16 with a cross reference in Section 706.2, where the other structural loads are defined. The engineers who are going to be required to design these walls for structural loading will be unlikely to find the design criteria in Chapter 7—these engineers live in Chapters 16 through 23. In addition, it is necessary for this code change to be in the IBC since most structural engineers do not purchase a copy of NFPA 221.

Second, the code change clarifies that the 5 psf out-of-plane load (required in Section 4.2 of NFPA 221) is Allowable Stress loading. The second sentence in the new Section 1607.14.2 then gives guidance to the design engineer what to use for Strength Design loading (which is what the vast majority of the loads in the code are calibrated to) and what load combinations to use in ASCE 7. The reference to Section 2.5.2.1 of ASCE 7 load combinations for extraordinary events addresses areas where earthquake loads are not required and where the conditions in the exception in ASCE 7 Section 13.5.8.1 and a design is required.

This in no way conflicts with or changes the requirements in NFPA 221—it is merely a conversion from one type of loading (Allowable Stress) to another (Strength Design), and brings consistency in application for code officials and design engineers. With this proposal, the design requirements in the IBC, NFPA 221, and ASCE 7 will all be coordinated. Without this code change, even if an engineer or code official were to find the reference to NFPA 221 and the appropriate section in that standard, there is no guidance in any of the three documents as to what kind of load it is (allowable stress or strength design), or what load combinations to use. In response to the Committee's statement published in the Report of the Committee Action Hearings regarding the cost of construction, buildings incorporating fire walls tend to be large buildings whose structures are designed by an engineer. The additional cost of designing the fire walls for this load is not significant.

**Cost Impact:** Will not increase the cost of construction

In response to the Committee's statement published in the Report of the Committee Action Hearings for code change FS28-15 in Group A regarding the cost of construction, buildings incorporating fire walls tend to be large buildings whose structures are designed by an engineer. The additional cost of designing the fire walls for this load is not significant.

**Public Hearing Results**

Committee Reason: Committee's action on S55-16 accomplishes the primary intent for fire wall design. A public comment is suggested to add a pointer to the Chapter 16 design requirements in Section 706.2.

Assembly Action: None

**Individual Consideration Agenda**

Public Comment 1:
Modify as Follows:

2015 International Building Code

706.2 Structural stability

Fire walls shall be designed and constructed to allow collapse of the structure on either side without collapse of the wall under fire conditions. Fire walls designed and constructed in accordance with Section 706, or NFPA 221 and, as well as Section 1607.14.2 shall be deemed to comply with this section.

1607.14.2 Fire walls. Fire walls and their supports shall be designed in accordance with this chapter. In addition to withstand allow collapse of the structure on either side without collapse of the wall when subjected to a minimum out-of-plane Allowable Stress Design load of 5 lbs/ft^2 applied from either direction, A or a minimum out-of-plane uniform Strength Design load, \( A_{L_a} \) of 8 lbs/ft^2 (0.38 kPa), in conjunction with the load combinations for extraordinary events in ASCE 7, is permitted to be used for design applied from either direction.

Commenter's Reason: This public comment is submitted in the event that the modifications in the public comment to S55 are not approved. Both S101 and S55 seek to accomplish the same outcome clarify what out of plane load to apply, that it is not an additive load concurrent with other loads, and that the verification needs to apply from either direction.

This loading issue becomes significant for Fire Walls that incorporate a non-load bearing fire resistance rated wall assembly located between wood framed load bearing wood framed walls. The load is necessary for determining the frequency and fastening of horizontal restraining attachment from either side of the free standing fire resistance rated wall assembly.

The committee disapproved S101 and the floor modification because S55 was approved.

We worked with AWC and other supportive groups and feel that S101 is more complete.
S103-16
IBC: 1608.2, 1608.3.

Proposed Change as Submitted

Proponent: Jennifer Goupil, AMERICAN SOCIETY OF CIVIL ENGINEERS, representing SELF (jgoupil@asce.org)

2015 International Building Code
Delete and substitute as follows:

FIGURE 1608.2
GROUND SNOW LOADS, \( p_g \), FOR THE UNITED STATES (psf)

(Existing code figure not shown for clarity)

Revise as follows:

FIGURE 1608.2
GROUND SNOW LOADS, \( p_g \), FOR THE UNITED STATES (psf)

(Existing code figure not shown for clarity)
1608.3 Ponding instability. Susceptible bays of roofs shall be evaluated for ponding instability in accordance with Section 7.1.14 Chapter 7 and Chapter 8 of ASCE 7.

Reason: This proposed changes to Section 1608 will harmonize the provision in the code with the 2016 edition of the referenced loading standard ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7-16).
The current ASCE 7 committee has developed the 2016 edition of the standard to supplement the basic ground snow load map with an extensive database of ground snow loading data for individual cites in regions with highly variable climatic conditions associated with mountains and other factors. This data was assembled over a period of many years through the efforts of regional experts and structural engineering associations with specialized knowledge in local climatic conditions and vetted by the Committee as having followed appropriate and consistent procedures. The revised map indicates which states have supplemental data within the ASCE 7-16 standard.

**Cost Impact:** Will not increase the cost of construction

The proposed changes will not impact the cost of construction. The proposed changes will add snow load data to the code that is already required by the states adding no additional burden to the design. This proposal coordinates the IBC with the referenced loading standard ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE 7 will be updated from the 2010 edition to the 2016 edition as an Administrative Update to the 2018 I-Codes.

As of the submission date of this code change proposal, the ASCE 7 Standards Committee has completed the committee balloting on the technical changes. The document designated ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures is expected to be completed, published, and available for purchase prior to the ICC Public Comment Hearing for Group B in October 2016. Any person interested in obtaining a public comment copy of ASCE 7-16 may do so by contacting James Neckel at ASCE (jneckel "at" asce.org).

Public Hearing Results

**Committee Action:** Approved as Submitted

**Committee Reason:** This proposal updates the IBC snow load provision for consistency with the latest edition of the referenced standard, ASCE 7, which was updated in ADM94-16. The updates incorporate necessary local conditions.

**Assembly Action:** None

**Individual Consideration Agenda**

**Public Comment 1:**

Proponent: Jennifer Goupil, American Society of Civil Engineers (ASCE), representing American Society of Civil Engineers (ASCE) (jgoupil@asce.org) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

**FIGURE 1608.2**

Ground Snow Loads, Pq, for the United States (psf)
Commenter's Reason: This changes is required to coordinate with ASCE 7-16. This was addressed during the ASCE 7-16 public comment and is incorporated into the final version of the standard.

The updated snow load data for the state of New Hampshire is in response to a public comment recieved for ASCE 7-16. This update is consistent with the changes adopted for the western mountainous states. The reference to the data eliminate the case study regions within the state but more importantly coordinates snow load data with the requirements of the state, which
governs the design. Because the map within IBC is duplicative of the snow map in ASCE 7-16, this is a coordinating update. There is no significant cost impact from this update to the IBC because the data referenced is in fact currently required by the state, this is simply a coordination item.

Proponent: Ed Berkel, representing ICC Code Correlation Committee (ccc@iccroles.org) requests Disapprove.

Commenter's Reason: The Code Correlation Committee requests Disapproval of this code change proposal in order to bring a correlation issue to the attention of the full membership at the Public Comment Hearings and to allow the membership to coordinate action on this code change proposal with action taken on Code Change Proposal ADM94-16.

ADM94-16 is the administrative update to referenced standards in the I-Codes. One of these standards, ASCE7, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, was proposed for update to ASCE7-16. However, a successful assembly motion requests that the referenced ASCE7 remain at ASCE7-10 as it presently is referenced in the 2015 I-Codes. This code change proposal coordinates with and relies upon reference to ASCE7-16.

The Code Correlation Committee is a standing committee of the International Code Council whose objectives, procedures and organization are set forth in Council Policy CP#44-13. The objective of the Code Correlation Committee is to maintain technical and editorial consistency among the International Codes and to assist staff in the evaluation and processing of code change proposals and comments that are exclusively editorial.
Proposed Change as Submitted

Proponent: W Lee Shoemaker, Thomas Associates Inc, representing Metal Building Manufacturers Association (lshoemaker@mbma.com)

2015 International Building Code

Revise as follows:

1609.1.1 Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapters 26 to 30 of ASCE 7 or provisions of the alternate all-heights method in Section 1609.6. The type of opening protection required, the ultimate design wind speed, $V_{ult}$, and the exposure category for a site is permitted to be determined in accordance with Section 1609 or ASCE 7. Wind shall be assumed to come from any horizontal direction and wind pressures shall be assumed to act normal to the surface considered.

Exceptions:

1. Subject to the limitations of Section 1609.1.1.1, the provisions of ICC 600 shall be permitted for applicable Group R-2 and R-3 buildings.
2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AWC WFCM.
3. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AISI S230.
5. Designs using TIA-222 for antenna-supporting structures and antennas, provided the horizontal extent of Topographic Category 2 escarpments in Section 2.6.6.2 of TIA-222 shall be 16 times the height of the escarpment.
6. Wind tunnel tests in accordance with ASCE 49 and Sections 31.4 and 31.5 of ASCE 7.
7. The wind load for roof components and cladding need not be greater than 1.3 times the corresponding wind load determined using ASCE 7-10.

The wind speeds in Figures 1609.3(1), 1609.3(2) and 1609.3(3) are ultimate design wind speeds, $V_{ult}$, and shall be converted in accordance with Section 1609.3.1 to nominal design wind speeds, $V_{asd}$, when the provisions of the standards referenced in Exceptions 4 and 5 are used.

Reason: ASCE 7-16, that is under public review at the time of this code change submittal, incorporates the latest information and knowledge with respect to wind loads. There was considerable ASCE 7 committee debate on revisions that drastically affected the roof component and cladding loads for gable and hip roofs with a mean roof height less than 60 feet. The ASCE 7-16 Commentary explains that "the negative roof (GC_r) values given in these figures are significantly greater (in magnitude) than those given in previous versions (2010 and earlier)...".

The GC_r pressure coefficients have been traditionally derived from wind tunnel tests. The most recent wind tunnel testing utilized a finer grid of pressure taps on the models that revealed that the GC_r values were significantly higher in some cases. However, the GC_r pressure coefficients are one of six factors that are combined in the equation to calculate the component and cladding wind loads. The other five factors include wind speed (V), directionality factor (K_d), internal pressure (GC_p), topographic factor (K_t), and velocity pressure exposure coefficient (K_z). The ASCE 7 debate did not focus on whether the roof component and cladding GC_r values were in fact greater based on the latest wind tunnel tests, but on the resulting wind loads. It did not seem realistic that we were underestimating design wind loads on roof components and cladding by a factor of more than 2 in some cases. For example fasteners in the field of a gable roof with a roof slope between 7 and 20 degrees would see the negative GC_r value increase from -0.9 to -2.0. It, in fact, we have been under-designing by a factor of two (well over the typical factor of safety), there would have been more widespread performance issues raised over the years associated with a design wind load deficiency rather than the most common failure issues that are sporadic and most often associated with poor quality of construction.

The question raised was – are there conservatisms in the other factors and systematic biases that have been offsetting the GC_r pressure coefficients? The resounding answer to that question from the ASCE 7 Committee was yes, absolutely. But, the debate was a philosophical one that divided the committee into two camps. Some said we know there are probably other conservatisms and the wind loads aren't as high as they will be with the proposed GC_r revisions, but we will start with GC_r and address the other factors in subsequent revisions. Others said that piecemeal approach would increase construction costs too severely, given the lack of performance issues. It was not a resounding decision – a swing of one vote would have changed the outcome.

It should be pointed out that the wind speed maps (V) are also revised in ASCE 7-16, and that there were some reductions in the interior portions of the United States (non-hurricane regions) that would partially offset the total wind load increase due to the increase in GC_r. However this reduction in wind speed is not manifested in the hurricane regions, where the largest wind loads would occur.
This proposal would cap the wind load on roof components and cladding to 30% higher than those that we are currently designing for. This is still a substantial increase, but it is felt that would be a prudent compromise while the other conservatisms are studied in the next cycle of ASCE 7. Several industry groups, including MBMA and AISI have committed funding to begin these studies. It is felt this would be the reasonable approach and this would also even out the severe swings in the wind loads from one cycle to the next that would ensue.

Cost Impact: Will increase the cost of construction
This proposal specifically addresses construction costs. The cost of a roof, especially in hurricane regions, could easily double without a reasonable cap on the wind loads mandated by ASCE 7-16. The roof covering, connections, substrate, and supporting members, as well as roof accessories such as solar panels, would all be impacted. With the 30% cap over existing design loads, roof costs will still potentially increase, but the impact will be much less and more acceptable considering the lack of performance problems.

Public Hearing Results
Committee Reason: The committee believes that wind load concerns need to be addressed in ASCE 7 committee. The contradictory testimony indicates the proponent should work on incorporating these suggestions in the public comment phase. There were concerns of significant economic impact from multiple industries and if there is such a jump in wind pressures, a gradual increase is warranted. Perhaps smoothing it out over three to six years may be warranted since it would give ASCE and industry a chance to fix the wind provisions or, if they are correct, this would turn out to be an incremental step. The concern is that such a cap not be tied to an older edition of the standard, but instead be a reduction to the pressure computed under ASCE 7-16. Another concern is that pressures increase significantly and they will also affect IEBC wind triggers which is not intended. It was also noted that the proposed exception is appropriately written as an option and a designer could still calculate wind pressures directly from ASCE 7-16. But in some locations there will be large increases in roof component and cladding pressures that is not accompanied by widespread field observation.

Individual Consideration Agenda
Public Comment 1:
Proponent: Mike Ennis, representing SPRI, Inc. (m.ennis@mac.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

1609.1.1 Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapters 26 to 30 of ASCE 7 or provisions of the alternate all-heights method in Section 1609.6. The type of opening protection required, the ultimate design wind speed, \( V_{ult} \), and the exposure category for a site is permitted to be determined in accordance with Section 1609 or ASCE 7. Wind shall be assumed to come from any horizontal direction and wind pressures shall be assumed to act normal to the surface considered.

Exceptions:
1. Subject to the limitations of Section 1609.1.1.1, the provisions of ICC 600 shall be permitted for applicable Group R-2 and R-3 buildings.
2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AWC WFCM.
3. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AISI S230.
5. Designs using TIA-222 for antenna-supporting structures and antennas, provided the horizontal extent of Topographic Category 2 escarpments in Section 2.6.6.2 of TIA-222 shall be 16 times the height of the escarpment.
6. Wind tunnel tests in accordance with ASCE 49 and Sections 31.4 and 31.5 of ASCE 7.

7. The wind load for roof components and cladding need not be greater than 1.3 times the corresponding wind load determined using ASCE 7-10.

7. It shall be permissible to calculate wind load for roof components and cladding using ASCE 7-10.

The wind speeds in Figures 1609.3(1), 1609.3(2) and 1609.3(3) are ultimate design wind speeds, $V_{ud}$, and shall be converted in accordance with Section 1609.3.1 to nominal design wind speeds, $V_{asd}$, when the provisions of the standards referenced in Exceptions 4 and 5 are used.

Commenter's Reason: ASCE7-16 nearly doubles the calculated wind loads in some situations with no field performance verification that this type of increase is justified. This Public Comment would allow for the use of ASCE7-10 to calculate wind loads until additional research can be conducted to evaluate other variables used in the calculation of wind loads. It is expected that this additional research would reduce calculated wind loads.

Proponent: Gary Ehrlich, National Association of Home Builders, representing National Association of Home Builders (gehrlich@nahb.org) requests Approve as Submitted.

Commenter's Reason: The purpose of this public comment is to express NAHB's support for MBMA's sensible, interim solution for addressing the significant increases in roof component and cladding pressures for low-rise buildings in the 2016 edition of ASCE 7. There was significant debate during the ASCE 7 update process, at both the Wind Load Subcommittee and Main Committee levels, over these roof pressures, and the final proposals barely achieved the threshold for consensus. As noted in the proponent's reason statement, there are many factors that affect the actual wind pressure a building sees, but which are either not currently considered by ASCE 7 or considered in an overly conservative way. Among these are wind directionality, the shielding effect from surrounding buildings, the effect of the surrounding terrain, and changes in air density due to temperature. If all these factors were taken into account, it would explain why failures have not occurred even in those isolated areas where near-design or design-level wind speeds have been experienced.

Even members of the ASCE 7 Wind Load Subcommittee who supported the new wind pressures agreed these factors should be researched for and considered in the next edition of ASCE 7. A number of research efforts are already planned, and along with other interested stakeholders NAHB has committed funding towards these efforts.

Capping the increase in component and cladding pressures is a sensible, interim solution that will reduce the amount of money and material wasted from overdesigning buildings for the next 6 years until this additional research can be completed and the 2022 edition of ASCE 7 revised to take advantage of these offsetting effects.

Proponent: Mike Ennis, representing SPRI, Inc. (m.ennis@mac.com) requests Approve as Submitted.

Commenter's Reason: The members of SPRI, Inc. include manufacturers of single ply roof membranes, components used in these systems, design professionals and testing laboratories. We are very concerned about the performance of these systems, including how they perform when exposed to wind loads. SPRI has supported RICOWI, an association that investigates the performance of various types of roof systems after weather events such as hurricanes and hail storms. We have used the knowledge we have gained to develop industry consensus standards to improve the field performance of our members systems. For example, we have developed ANSI national consensus standards to test the edge securement of low slope roof assemblies and to properly design ballasted systems so that aggregate blow-off does not occur. We further worked to get these standards included in the IBC. We have also developed an ANSI consensus gutter securement standard and are working to get it accepted in the IBC.

We support reasonable efforts to improve the performance of roof systems when exposed to wind loads. Our concern with this proposal is that the calculated wind loads are increased by over 50% in some situations greatly increasing the cost of construction without any field performance issues justifying this very large increase. We have participated in four hurricane investigations, Ivan, Charley, Ike and Katrina. We reported at the Committee Action Hearing that while roof system damage was observed it was not due to the roof system being under-designed for the wind load. Loss of edge securement, tears in the roof membrane and other workmanship issues were noted, not being under-designed for the wind load. Testimony was offered in opposition to our testimony that this was because these hurricanes had not reached design wind speeds. We want to point out that Hurricane Charley made landfall on Friday, August 13, 2004, at Port Charlotte and the community of Punta Gorda as a Category 4 hurricane. This made Hurricane Charley the strongest wind event in the United States since Hurricane Andrew in 1992. With wind speeds estimated at over 150 mph for sustained winds and well over 170 mph for gusts, Charley provided the first opportunity for observation of a “code event” in 23 years. We did not observe damage that would indicate that the systems were under-designed.
We believe that S105 provides a reasonable compromise to cap the calculated wind load increases at 30%, still resulting in a large, but manageable calculated wind load increase, while allowing time to complete additional research on other variables used in the ASCE7 calculation, that we believe will actually decrease the calculated wind loads.

Proponent: W Lee Shoemaker, Thomas Associates, Representing the Metal Building Manufacturers Association, representing Metal Building Manufacturers Association (lishoemaker@mbma.com) requests Approve as Submitted.

Commenter's Reason: This proposal limits the roof component and cladding wind load to 30% more than ASCE 7-10 - a 30% increase over the current design wind load. This reasonable limit is necessary because ASCE 7-16 is incomplete with regard to the introduction of new wind tunnel data that produces overly conservative loads in hurricane affected areas. This limit would be optional, i.e. roof component and cladding may be based on ASCE 7-16 without checking this upper limit. However, it would permit reasonable, economic roof designs in hurricane areas while additional research is carried out to eliminate the conservatism that were introduced into ASCE 7-16.

There are normally improvements and additions to ASCE 7 with each cycle, based on new research and information. These revisions are usually moderate adjustments. What happened with the roof component and cladding loads in ASCE 7-16 was not a moderate adjustment - uplift forces can more than double in the hurricane affected areas. A change of this magnitude should not happen unless warranted by a significant event or performance issue.

As the representative of a major industry that designs and supplies roofing (MBMA members produced approximately 2.25 billion square feet of low rise buildings in the past 10 years), we would be leading the effort to make corrections if we felt that there was a problem or that the current roof uplift loads were as inadequately low as ASCE 7-16 would have you believe. We also have an obligation to provide cost effective solutions that perform well. We think that it is punitive to the cost of construction without field evidence that would say that we have been underestimating roof uplift loads by more than a factor of 2 in some cases. Additional research is required on both the load side and the resistance side of the equation to see if other load phenomena are at play or if the uniform static load tests on roofing materials are overly conservative to explain this discrepancy.

This particular change in ASCE 7-16 that would be affected by the proposed limit was narrowly passed - in fact a swing of one vote would have changed the outcome. The IBC Structural Committee disapproved S105 by a vote of 8-7, again, a one vote swing would have changed the outcome. Now it is time for government representatives to cast their vote. If you feel that roof wind loads should be designed for a load more than 30% over what we are currently doing because you have seen problems that would justify this increase, then you should vote against S105, but if you have not seen problems, please vote for S105 and keep construction cost for roofs reasonable until further research can be carried out.
Proposed Change as Submitted

Proponent: Mike Ennis, representing SPRI Inc. (m.ennis@mac.com)

2015 International Building Code

Revise as follows:

1609.5.2 Roof coverings. Roof coverings shall comply with Section 1609.5.1.

Exception: Rigid tile roof coverings that are air permeable and installed over a roof deck complying with Section 1609.5.1 are permitted to be designed in accordance with Section 1609.5.3.

Asphalt shingles installed over a roof deck complying with Section 1609.5.1 shall comply with the wind-resistance requirements of Section 1504.1.1. Non-ballasted roof coverings installed over a roof deck complying with Section 1609.5.1 shall comply with the wind resistance requirements of Section 1504.3 and shall be permitted to use a wind load of 0.5W instead of 0.6W in accordance with Equation 16-15 in Section 1605.3.1.

Reason: A load factor of 0.5, instead of 0.6 should be used for allowable stress design of non-life-safety building components such as roof coverings. There is precedence for this approach in Table 1604.3, note f, for non-life safety consideration of deflections using service loads instead of life-safety loads. This proposal is more conservative than note f because we are not targeting a 10-year service load as used for deflection design purposes, instead a 30 year service load is proposed for design of the roof covering component (and not the structural roof deck) given that the roof covering performance is not a matter of life safety (as is the case for deflections) but does have economic implications that must be practically balanced with life-expectancy of the component, first costs, and cost to replace. Designing for a 700-year return period wind event with a component that may only have a 30-year service life-expectancy and must be periodically replaced to maintain reliable performance is not practical. Using a wind load factor of 0.5 instead will better ensure risk-consistent designs and encourage timely and economical roof replacements that should help improve overall roof covering performance. The difference in wind speed between the 700-yr (Risk II map) and the 300-yr (Risk I map) is equivalent to a factor of approximately 1.2 on wind load. This yields a corrected wind load factor of \((1/1.2)(0.6) = 0.5\).

Cost Impact: Will not increase the cost of construction

While this proposal is justified on its own merits, it will also help offset expected cost increases anticipated in changes to ASCE 7-16 roof component and cladding wind loads that have failed to consider offsetting wind load effects in a standard that focuses primarily on structural safety applications, not serviceability and economic design considerations.

Committee Action: Disapproved

Committee Reason: See S79-16 reason.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Mike Ennis, representing SPRI, Inc. (m.ennis@mac.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

1609.5.2 Roof coverings. Roof coverings shall comply with Section 1609.5.1.

Exception: Rigid tile roof coverings that are air permeable and installed over a roof deck complying with Section 1609.5.1 are permitted to be designed in accordance with Section 1609.5.3.

Asphalt shingles installed over a roof deck complying with Section 1609.5.1 shall comply with the wind-resistance requirements of Section 1504.1.1. Non-ballasted roof coverings installed over a roof deck complying with Section 1609.5.1 shall comply with the wind resistance requirements of Section 1504.3 and shall be permitted to use a wind load instead of 0.6W. This can be calculated using ASCE7-10 to determine the appropriate wind load factor.
Commenter's Reason: ASCE7-16 nearly doubles the calculated wind loads in some situations with no field performance verification that this type of increase is justified. This Public Comment would allow for the use of ASCE7-10 to calculate wind loads until additional research can be conducted to evaluate other variables used in the calculation of wind loads. It is expected that this additional research would reduce calculated wind loads.
Proposed Change as Submitted

Proponent: Ronald Hamburger, SIMPSON GUMPertz & HEGER, representing SELF (rohamburger@sgh.com)

2015 International Building Code

Delete without substitution:

1609.6 - Alternate all-heights method. The alternate wind design provisions in this section are simplifications of the ASCE 7 Directional Procedure.

1609.6.1 - Scope. As an alternative to ASCE 7 Chapters 27 and 30, the following provisions are permitted to be used to determine the wind effects on regularly shaped buildings, or other structures that are regularly shaped, that meet all of the following conditions:

1. The building or other structure is less than or equal to 75 feet (22 860 mm) in height with a height-to-least-width ratio of 4 or less, or the building or other structure has a fundamental frequency greater than or equal to 1 hertz.
2. The building or other structure is not sensitive to dynamic effects.
3. The building or other structure is not located on a site for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.
4. The building shall meet the requirements of a simple diaphragm building as defined in ASCE 7 Section 26.2, where wind loads are only transmitted to the main windforce-resisting system (MWFRS) at the diaphragm.
5. For open buildings, multispan gable roofs, stepped roofs, sawtooth roofs, domed roofs, roofs with slopes greater than 45 degrees (0.79 rad), solid free-standing walls and solid signs, and rooftop equipment, apply ASCE 7 provisions.

1609.6.1.1 - Modifications. The following modifications shall be made to certain subsections in ASCE 7; in Section 1609.6.2, symbols and notations that are specific to this section are used in conjunction with the symbols and notations in ASCE 7 Section 26.3.

1609.6.2 - Symbols and notations. Coefficients and variables used in the alternative all-heights method equations are as follows:

\[ C_{Net} = K_d \left| G \right| \left( C_p - \left( G C_p \right) \right), \text{ in accordance with Table 1609.6.2.} \]

\[ G = \text{ gust effect factor for rigid structures in accordance with ASCE 7 Section 26.9.1.} \]

\[ K_d = \text{ wind directionality factor in accordance with ASCE 7 Table 26.6.} \]

\[ P_{Net} = \text{ design wind pressure to be used in determination of wind loads on buildings or other structures or their components and cladding, in psf (kN/m^2).} \]

1609.6.3 - Design equations. When using the alternative all-heights method, the MWFRS, and components and cladding of every structure shall be designed to resist the effects of wind pressures on the building envelope in accordance with Equation 16-35.

\[ P_{Net} = 0.00256 V^2 K_d C_{Ref} K_p \text{ (Equation 16-35)} \]

Design wind forces for the MWFRS shall be not less than 16 psf (0.77 kN/m^2), multiplied by the area of the structure projected on a plane normal to the assumed wind direction (see ASCE 7 Section 27.4.7 for criteria). Design net wind pressure for components and cladding shall be not less than 16 psf (0.77 kN/m^2) acting in either direction normal to the surface.

1609.6.4 - Design procedure. The MWFRS and the components and cladding of every building or other structure shall be designed for the pressures calculated using Equation 16-35.

1609.6.4.1 - Main windforce-resisting systems. The MWFRS shall be investigated for the torsional effects identified in ASCE 7 Figure 27.4-8.

1609.6.4.2 - Determination of \( K_x \) and \( K_{zt} \). Velocity pressure exposure coefficient, \( K_x \), shall be determined in accordance with ASCE 7 Section 27.3.1 and the topographic factor, \( K_{zt} \), shall be determined in accordance with ASCE 7 Section 26.8.

1. For the windward side of a structure, \( K_x \) and \( K_{zt} \) shall be based on height \( z \).
2. For leeward and sidewalls, and for windward and leeward roofs, \( K_x \) and \( K_{zt} \) shall be based on mean roof height \( h \).

1609.6.4.3 - Determination of net pressure coefficients, \( C_{Ref} \). For the design of the MWFRS and for components and cladding, the sum of the internal and external net pressure shall be based on the net pressure coefficient, \( C_{Ref} \).
1. The pressure coefficient, $C_{net}$, for walls and roofs shall be determined from Table 1609.6.2.
2. Where $C_{net}$ has more than one value, the more severe wind load condition shall be used for design.

1609.6.4.4 Application of wind pressures. When using the alternative all-heights method, wind pressures shall be applied simultaneously on, and in a direction normal to, all building envelope wall and roof surfaces.

1609.6.4.4.1 Components and cladding. Wind pressure for each component or cladding element is applied as follows using $C_{net}$ values based on the effective wind area, $A$, contained within the zones in areas of discontinuity of width and/or length “$a$,” “$2a$” or “$4a$” at corners of roofs and walls, edge strips for ridges, rakes and eaves; or field areas on walls or roofs as indicated in figures in tables in ASCE 7 as referenced in Table 1609.6.2 in accordance with the following:

1. Calculated pressures at local discontinuities acting over specific edge strips or corner boundary areas.
2. Include “field” (Zone 1, 2 or 4, as applicable) pressures applied to areas beyond the boundaries of the areas of discontinuity.
3. Where applicable, the calculated pressures at discontinuities (Zone 2 or 3) shall be combined with design pressures that apply specifically on rakes or eave overhangs.

**TABLE 1609.6.2**

<table>
<thead>
<tr>
<th>$C_{net}^{a-b}$</th>
</tr>
</thead>
</table>

**Reason:** Since 2006, the IBC has permitted determination of wind loads using either the procedures contained in ASCE 7 or a series of simplified procedures known as the "Alternate all-heights method" contained in Section 1609.6 of the IBC. The "Alternate all-heights method" was originally developed by the Western States Structural Engineering Associations, a consortium of the Structural Engineers Associations (SEAs) of California, Washington and a few other states. The Western States SEAs developed this procedure because those members felt that the procedures contained within ASCE 7 were excessively complex and difficult to apply to the design of buildings. These engineers wanted simplified procedures, similar to those which had formerly appeared in the legacy Uniform Building Code. It is worth noting that the two other legacy codes essentially transcribed the ASCE 7 provisions and that engineers in the eastern United States did not have problems using the ASCE 7 provisions.

In the time since the Alternate all-heights method was developed, wind engineering has advanced substantially and the pressure coefficients specified by the ASCE 7 procedures have been revised several times because research showed that the older coefficients were unconservative. As a result, the Alternate all-heights method contained in the code no longer provides similar levels of structural performance as the procedures contained in ASCE 7. The Western States Structural Engineers Associations that originally developed the alternative method have not updated it to keep pace with ASCE 7, and instead, have largely participated in the development of the wind provisions within the ASCE 7 Standard. Further, in 2010 in response to western engineers’ concerns that the procedures embodied in ASCE 7 were excessively complex, the ASCE 7 Standard adopted an alternative simplified procedure similar to the Alternate all-heights method. Recent surveys of engineering practice by the National Council of Structural Engineers Associations Wind Loads Committee has determined that most engineers do not use the Alternate all-heights method, and instead use one of the several methods available in ASCE 7.

Removal of the Alternate all-heights method will ensure that all buildings and structures in the United States are designed with consistent levels of safety and serviceability for wind loading.

**Cost Impact:** Will not increase the cost of construction

The proposed changes will not impact the cost of construction. This proposal is a re-organization of the pointers in the IBC to refer to the wind provisions in the referenced loading standard ASCE 7. ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures will be updated from the 2010 edition to the 2016 edition as an Administrative Update to the 2018 I-Codes.

As of the submission date of this code change proposal, the ASCE 7 Standards Committee has completed the committee balloting on the technical changes. The document designated ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures is expected to be completed, published, and available for purchase prior to the ICC Public Comment Hearings for Group B in October 2016. Any person interested in obtaining a public comment copy of ASCE 7-16 may do so by contacting JamesNeckel at ASCE (jneckel "at" asce.org ).

Public Hearing Results
Committee Action: Approved as Submitted

Committee Reason: Agreement with proponent's reason which indicates the Alternate all heights method in the IBC has not kept pace with the wind updates in the ASCE 7 referenced standard and therefore it does not provide the same level of protection.

Assembly Action: None

Individual Consideration Agenda

Proponent: Alan Robinson, SEAOC Code Committee, representing Structural Engineers Association of California Code Committee requests Disapprove.

Commenter's Reason: The Structural Engineers Association of California (SEAOC) disagrees with the proponent's reason statement that the IBC Alternate all-height methods (2015 IBC Section 1609.6) does not provide the same level of safety and serviceability for wind loading. S109-16 has been approved at CAH by the Committee with the net pressure ($C_{net}$) values modified to correspond to changes in ASCE 7-16 for components and cladding. The IBC Alternate all-heights method yields the same or more conservative results once Table 1609.6.2 is updated. While practicing engineers may option to use the ASCE 7 provisions, the Alternate all-heights method in IBC serves to simplify the calculation process with less chance of error in reading ASCE 7 figures. Additionally, the tabulated net pressure coefficients for both the main windforce resisting system and components and claddings serve as a good tool for Plan Check officials to spot check calculation submittals.

S108-16
TABLE 1609.6.2

NET PRESSURE COEFFICIENTS, $C_{\text{net}}^{a, b}$

<table>
<thead>
<tr>
<th>STRUCTURE OR PART THEREOF</th>
<th>DESCRIPTION</th>
<th>$C_{\text{net}}$ FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Enclosed</td>
</tr>
<tr>
<td></td>
<td>+ Internal pressure</td>
<td>- Internal pressure</td>
</tr>
<tr>
<td>Walls:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Windward wall</td>
<td>0.43</td>
<td>0.73</td>
</tr>
<tr>
<td>Leeward wall</td>
<td>-0.51</td>
<td>-0.21</td>
</tr>
<tr>
<td>Sidewall</td>
<td>-0.66</td>
<td>-0.35</td>
</tr>
<tr>
<td>Parapet wall</td>
<td>Windward</td>
<td>1.28</td>
</tr>
<tr>
<td></td>
<td>Leeward</td>
<td>-0.85</td>
</tr>
<tr>
<td>Roofs:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wind perpendicular to ridge</td>
<td>+ Internal</td>
<td>- Internal</td>
</tr>
<tr>
<td>Leeward roof or flat roof</td>
<td>-0.66</td>
<td>-0.35</td>
</tr>
<tr>
<td>Windward roof slopes:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slope</td>
<td>Condition 1</td>
<td>-1.09</td>
</tr>
<tr>
<td>Slope = 4:12 (18°)</td>
<td>Condition 2</td>
<td>-0.73</td>
</tr>
<tr>
<td>Slope</td>
<td>Condition 2</td>
<td>-0.05</td>
</tr>
<tr>
<td>Slope = 5:12 (23°)</td>
<td>Condition 2</td>
<td>-0.58</td>
</tr>
<tr>
<td>Slope = 6:12 (27°)</td>
<td>Condition 2</td>
<td>0.03</td>
</tr>
<tr>
<td>Slope = 7:12 (30°)</td>
<td>Condition 1</td>
<td>0.47</td>
</tr>
<tr>
<td>Slope = 9:12 (37°)</td>
<td>Condition 2</td>
<td>-0.27</td>
</tr>
<tr>
<td>Slope = 12:12 (45°)</td>
<td></td>
<td>0.14</td>
</tr>
<tr>
<td>Wind parallel to ridge and flat roofs</td>
<td>-1.09</td>
<td>-0.79</td>
</tr>
<tr>
<td>Nonbuilding Structures: Chimneys, Tanks and Similar Structures:</td>
<td>1, 7, 25</td>
<td></td>
</tr>
<tr>
<td>h/D</td>
<td>Square (Wind normal to face)</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td>Square (Wind on diagonal)</td>
<td>0.77</td>
</tr>
<tr>
<td></td>
<td>Hexagonal or octagonal</td>
<td>0.81</td>
</tr>
<tr>
<td></td>
<td>Round</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td>Open signs and lattice frameworks</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Flat</td>
<td>1.45</td>
</tr>
<tr>
<td></td>
<td>Round</td>
<td>0.87</td>
</tr>
</tbody>
</table>

1. Main windforce-resisting frames and systems

<table>
<thead>
<tr>
<th>STRUCTURE OR PART THEREOF</th>
<th>DESCRIPTION</th>
<th>$C_{\text{net}}$ FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Enclosed</td>
</tr>
<tr>
<td>Roof elements and slopes</td>
<td>Enclosed</td>
<td></td>
</tr>
<tr>
<td>Gable of hipped configurations (Zone 1)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flat</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Positive</td>
<td>10 square feet or less</td>
<td>0.580.75</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>0.41</td>
</tr>
<tr>
<td>STRUCTURE OR PART THEREOF</td>
<td>DESCRIPTION</td>
<td>$C_{net}$</td>
</tr>
<tr>
<td>---------------------------</td>
<td>-------------</td>
<td>----------</td>
</tr>
<tr>
<td>Roof elements and slopes</td>
<td>Enclosed</td>
<td>Partially enclosed</td>
</tr>
<tr>
<td>Monosloped configurations at ridges, eaves and rakes (Zone 2)</td>
<td>Flat</td>
<td></td>
</tr>
<tr>
<td>Positive</td>
<td>10 square feet or less</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>0.41</td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.51</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-1.43</td>
</tr>
<tr>
<td>Tall flat topped roofs $h &gt; 60$ feet</td>
<td>Enclosed</td>
<td>Partially enclosed</td>
</tr>
<tr>
<td>Flat</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-2.11</td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>-1.51</td>
</tr>
<tr>
<td>Gable or hipped configurations at corners (Zone 3)</td>
<td>See ASCE 7 Figure 30.4-2B / Zone 3</td>
<td></td>
</tr>
<tr>
<td>Flat</td>
<td>Enclosed</td>
<td>Partially enclosed</td>
</tr>
<tr>
<td>Positive</td>
<td>10 square feet or less</td>
<td>0.580.75</td>
</tr>
</tbody>
</table>

### Table 2. Components and cladding not in areas of discontinuity—roofs and overhangs

<table>
<thead>
<tr>
<th>COMPONENTS AND CLADDING</th>
<th>DESCRIPTION</th>
<th>$C_{net}$</th>
<th>FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.00</td>
<td>-1.85</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-0.92</td>
<td>-1.68</td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>-0.94</td>
<td>-1.68</td>
</tr>
<tr>
<td>Overhang: Flat</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.45</td>
<td>-2.28</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-1.36</td>
<td>-1.77</td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>-0.94</td>
<td>-1.68</td>
</tr>
<tr>
<td>Overhang for Slope Flat</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.00</td>
<td>-1.68</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-0.83</td>
<td>-1.15</td>
</tr>
<tr>
<td>Overhang for 6:12 (27°)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.34</td>
<td>-1.66</td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>-0.92</td>
<td>-1.23</td>
</tr>
<tr>
<td>Tall flat topped roofs $h &gt; 60$ feet</td>
<td>Enclosed</td>
<td>Partially enclosed</td>
<td></td>
</tr>
<tr>
<td>Flat</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.70</td>
<td>-3.13</td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>-1.53</td>
<td>-1.68</td>
</tr>
</tbody>
</table>

### Table 3. Components and cladding in areas of discontinuity—roofs and overhangs

<table>
<thead>
<tr>
<th>COMPONENTS AND CLADDING</th>
<th>DESCRIPTION</th>
<th>$C_{net}$</th>
<th>FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.32</td>
<td>-2.00</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-0.83</td>
<td>-1.15</td>
</tr>
<tr>
<td>Monosloped configurations (Zone 1)</td>
<td>Enclosed</td>
<td>Partially enclosed</td>
<td></td>
</tr>
<tr>
<td>Flat</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Positive</td>
<td>10 square feet or less</td>
<td>0.49</td>
<td>0.81</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>0.41</td>
<td>0.72</td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.26</td>
<td>-1.57</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-1.09</td>
<td>-1.40</td>
</tr>
<tr>
<td>Tall flat topped roofs $h &gt; 60$ feet</td>
<td>Enclosed</td>
<td>Partially enclosed</td>
<td></td>
</tr>
<tr>
<td>Flat</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.34</td>
<td>-1.66</td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>-0.92</td>
<td>-1.23</td>
</tr>
</tbody>
</table>

### Table 4. Components and cladding in areas of discontinuity—roofs and overhangs (continued)

<table>
<thead>
<tr>
<th>COMPONENTS AND CLADDING</th>
<th>DESCRIPTION</th>
<th>$C_{net}$</th>
<th>FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.45</td>
<td>-2.28</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-1.36</td>
<td>-1.77</td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>-0.94</td>
<td>-1.68</td>
</tr>
<tr>
<td>Overhang for Slope Flat</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.00</td>
<td>-1.68</td>
</tr>
<tr>
<td></td>
<td>100 square feet or more</td>
<td>-0.83</td>
<td>-1.15</td>
</tr>
<tr>
<td>Overhang for 6:12 (27°)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative</td>
<td>10 square feet or less</td>
<td>-1.70</td>
<td>-3.13</td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>-1.53</td>
<td>-1.68</td>
</tr>
<tr>
<td>Gable or hipped configurations at corners (Zone 3)</td>
<td>See ASCE 7 Figure 30.4-2B / Zone 3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flat</td>
<td>Enclosed</td>
<td>Partially enclosed</td>
<td></td>
</tr>
<tr>
<td>Positive</td>
<td>10 square feet or less</td>
<td>0.580.75</td>
<td>0.891.06</td>
</tr>
</tbody>
</table>
### 3. Components and cladding in areas of discontinuity—roofs and overhangs

<table>
<thead>
<tr>
<th>Positive</th>
<th>Negative</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 square feet or more</td>
<td>0.41</td>
</tr>
<tr>
<td>10 square feet or less</td>
<td>-2.53</td>
</tr>
<tr>
<td>100 square feet or more</td>
<td>-1.85</td>
</tr>
</tbody>
</table>

**Overhang for Slope Flat**

<table>
<thead>
<tr>
<th>Positive</th>
<th>Negative</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 square feet or less</td>
<td>-3.15</td>
</tr>
<tr>
<td>100 square feet or more</td>
<td>-2.13</td>
</tr>
</tbody>
</table>

**6:12 (27°)**

<table>
<thead>
<tr>
<th>Positive</th>
<th>Negative</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 square feet or less</td>
<td>0.92</td>
</tr>
<tr>
<td>100 square feet or more</td>
<td>0.83</td>
</tr>
</tbody>
</table>

**Overhang for 6:12 (27°)**

<table>
<thead>
<tr>
<th>Enclosed</th>
<th>Partially enclosed</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 square feet or less</td>
<td>-0.92</td>
</tr>
<tr>
<td>500 square feet or more</td>
<td>-0.72</td>
</tr>
</tbody>
</table>

### 4. Components and cladding not in areas of discontinuity—walls and parapets (continued)

#### Wall Elements: \( h \leq 60 \) feet (Zone 4) ASCE 7 Figure 30.4-1

<table>
<thead>
<tr>
<th>Positive</th>
<th>Negative</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 square feet or less</td>
<td>1.00</td>
</tr>
<tr>
<td>500 square feet or more</td>
<td>0.75</td>
</tr>
</tbody>
</table>

**Wall Elements: \( h > 60 \) feet (Zone 4) ASCE 7 Figure 30.6-1 Zone 4**

<table>
<thead>
<tr>
<th>Positive</th>
<th>Negative</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 square feet or less</td>
<td>0.92</td>
</tr>
<tr>
<td>500 square feet or more</td>
<td>0.66</td>
</tr>
</tbody>
</table>

### STRUCTURE OR PART THEREOF

<table>
<thead>
<tr>
<th>DESCRIPTION</th>
<th>( C_{net} ) FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Negative 20 square feet or less</td>
<td>-0.92</td>
</tr>
<tr>
<td>500 square feet or more</td>
<td>-0.75</td>
</tr>
<tr>
<td>Parapet Walls Positive</td>
<td>2.87</td>
</tr>
<tr>
<td>Negative</td>
<td>-1.68</td>
</tr>
</tbody>
</table>

**Wall elements: \( h \leq 60 \) feet (Zone 5) ASCE 7 Figure 30.4-1**

<table>
<thead>
<tr>
<th>Enclosed</th>
<th>Partially enclosed</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 square feet or less</td>
<td>1.00</td>
</tr>
<tr>
<td>500 square feet or more</td>
<td>0.75</td>
</tr>
</tbody>
</table>

**Wall elements: \( h > 60 \) feet (Zone 5) ASCE 7 Figure 30.6-1 Zone 4**

<table>
<thead>
<tr>
<th>Positive</th>
<th>Negative</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 square feet or less</td>
<td>0.92</td>
</tr>
<tr>
<td>500 square feet or more</td>
<td>0.66</td>
</tr>
</tbody>
</table>
Committee Action: Approved as Submitted

Assembly Action: None

<table>
<thead>
<tr>
<th></th>
<th>500 square feet or more</th>
<th>0.66</th>
<th>0.98</th>
</tr>
</thead>
<tbody>
<tr>
<td>Negative</td>
<td>20 square feet or less</td>
<td>-1.68</td>
<td>-2.00</td>
</tr>
<tr>
<td></td>
<td>500 square feet or more</td>
<td>-1.00</td>
<td>-1.32</td>
</tr>
</tbody>
</table>

Parapet walls

| Positive             | 3.64  | 3.95  |
| Negative             | -2.45 | -2.76 |

For SI: 1 foot = 304.8 mm, 1 square foot = 0.0929m², 1 degree = 0.0175 rad.
a. Linear interpolation between values in the table is permitted.
b. Some Cnet values have been grouped together. Less conservative results may be obtained by applying ASCE 7 provisions.

Reason: Full scale testing and wind tunnel testing has shown that for low-rise buildings the roof pressures coefficients have been non-conservative for the past editions of the code. The values in ASCE 7-16 have been corrected and thus the values in Table 1609.6.2 need to be adjusted to match the ASCE 7-16 values.

Cost Impact: Will increase the cost of construction

Initial roof construction and roofing costs will increase but overall repair costs following major wind events will be decreased. The layout of this portion of the table has been formatted to match the remaining portions of the table from the 2015 IBC. This formatting leads to very conservative design values being utilized as compared to the ASCE 7-16 figures for the various wind zones specified in ASCE 7-16.

S109-16: 1609.6.2-
SCOTT12744

Public Hearing Results

Committee Reason: This proposal updates the IBC alternative all heights provisions for coordination with the latest edition of the referenced standard, ASCE 7, which was updated in ADM84-16

Assembly Action: None

Individual Consideration Agenda

Proponent: Ed Berkel, representing ICC Code Correlation Committee (ccc@iccwest.org) requests Disapprove.

Commenter's Reason: The Code Correlation Committee requests Disapproval of this code change proposal in order to bring a correlation issue to the attention of the full membership at the Public Comment Hearings and to allow the membership to coordinate action on this code change proposal with action taken on Code Change Proposal ADM94-16.

ADM94-16 is the administrative update to referenced standards in the I-Codes. One of these standards, ASCE 7, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, was proposed for update to ASCE 7-16. However, a successful assembly motion requests that the referenced ASCE 7 remain at ASCE 7-10 as it presently is referenced in the 2015 I-Codes. This code change proposal coordinates with and relies upon reference to ASCE 7-16.

The Code Correlation Committee is a standing committee of the International Code Council whose objectives, procedures and organization are set forth in Council Policy CP#44-13. The objective of the Code Correlation Committee is to maintain technical and editorial consistency among the International Codes and to assist staff in the evaluation and processing of code change proposals and comments that are exclusively editorial.
Proposed Change as Submitted

Proponent: Jennifer Goupil, AMERICAN SOCIETY OF CIVIL ENGINEERS, representing SELF (jgoupil@asce.org)

2015 International Building Code

Revise as follows:

1611.1 Design rain loads. Each portion of a roof shall be designed to sustain the load of rainwater that will accumulate on it if as per requirements of the secondary drainage system at its design flow Chapter 8 of ASCE 7. The design rainfall shall be based on the 100-year 15-minute duration event, which is twice the 100-year hourly rainfall rate indicated in Figure 1611.1, or on other rainfall rates determined from approved local weather data.

\[ R = 5.2 \left( d_s + d_h \right) \]  
(Equation 16-36)

For SI: \( R = 0.0098(d_s + d_h) \)

where:

- \( d_s \) = Additional depth of water on the undeflected roof above the inlet of secondary drainage system at its design flow (i.e., the hydraulic head), in inches (mm).
- \( d_h \) = Depth of water on the undeflected roof up to the inlet of secondary drainage system when the primary drainage system is blocked (i.e., the static head), in inches (mm).
- \( R \) = Rain load on the undeflected roof, in psf (kN/m²). When the phrase "undeflected roof" is used, deflections from loads (including dead loads) shall not be considered when determining the amount of rain on the roof.

1611.2 Ponding instability. Susceptible bays of roofs shall be evaluated for ponding instability in accordance with Section 8.4-Chapter 7 and Chapter 8 of ASCE 7.

Reason: This proposed changes to Section 1611 will harmonize the provision in the code with the 2016 edition of the referenced loading standard ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7-16).

The proposed changes to secondary (overflow) system design harmonizes the roof load design for the structure with the expectations for the design of the roof drainage system. This proposal coordinate the IBC with the referenced loading standard ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures, which was updated to be consistent with the International Plumbing Code provisions. The changes provide a basis for the design mean reoccurrence interval and duration for determining the Hydraulic Head (\( d_h \)). Currently the IBC requires the calculation of \( d_h \); however, the code does not state the design storm (mean reoccurrence interval and duration) for determining the design rain load (depth of water on the undeflected roof) and it has led some confusion. Typical design values for plumbing systems have been between 15 minute and 60 minutes; the 1995 International Plumbing Code (IPC) first used the 100-year / 60-minute duration for the design of the primary drainage system and 2 times the flow rate from the 100-year / 60-minute duration storm for the design of the secondary drainage system.

The use of twice the flow rate of the 1-hour duration is close to the design intensity for the 15 minute duration storm. The IPC also used a 15-minute duration rainfall event for the design of roof drainage systems. The basis for the use of a 1-hour duration storm is unclear – the critical duration for most roof geometries is closer to 15 minutes. Graber (2009) provides guidance for determining the critical duration and the paper advises against the use of the 1-hour duration storm for the design of the primary and secondary drainage system in hopes of handling the critical short-duration rainfall event.

ASCE 7 does not provide rainfall data or maps for determining the rainfall rate. The best source currently is the National Oceanic and Atmospheric Administration (NOAA's) National Weather Service Precipitation Frequency Data Server - Hydrometeorological Design Studies Center (http://hdsc.nws.noaa.gov/hdsc/pfds/index.html) for precipitation intensity (inches per hour) based on the 100-year mean reoccurrence interval.
Cost Impact: Will increase the cost of construction
The proposed changes may impact the design of roofs where the secondary (overflow) system was previously based on an unconservative hydraulic head from a lower rainfall intensity. The changes harmonizes the roof load design for the structure with the expectations for the design of the roof drainage system. This proposal coordinate the IBC with the referenced loading standard ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures, which was updated to be consistent with the International Plumbing Code provisions. ASCE 7 will be updated from the 2010 edition to the 2016 edition as an Administrative Update to the 2018 I-Codes.
As of the submission date of this code change proposal, the ASCE 7 Standards Committee has completed the committee balloting on the technical changes. The document designated ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures is expected to be completed, published, and available for purchase prior to the ICC Public Comment Hearings for Group B in October of 2016. Any person interested in obtaining a public comment copy of ASCE 7-16 may do so by contacting James Neckel at ASCE (jneckel "at" asce.org.)

Public Hearing Results

Committee Action: Approved as Submitted
Committee Reason: This code change updates the IBC rain load criteria for consistency with the latest edition of the referenced standard, ASCE 7, which was updated in ADM94-16.

Assembly Action: None

Individual Consideration Agenda

Proponent: Jennifer Goupil, American Society of Civil Engineers (ASCE), representing American Society of Civil Engineers (ASCE) (jgoupil@asce.org) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

1611.1 Design rain loads. Each portion of a roof shall be designed to sustain the load of rainwater as per the requirements of Chapter 8 of ASCE 7. The design rainfall shall be based on the 100-year 15-minute duration event, which is twice the 100-year hourly rainfall rate indicated in Figure 1611.1, or on other rainfall rates determined from approved local weather data. Alternatively, a design rainfall of twice the 100-year hourly rainfall rate indicated in Figure 1611.1 shall be permitted.

\[ R = 5.2(d_s + d_h) \]  \( \text{(Equation 16-36)} \)
For SI: \[ R = 0.0098(d_s + d_h) \]

where:
- \( d_h \) = secondary drainage system at its design flow (i.e., the hydraulic head), in inches (mm).
- \( d_s \) = depth of water on the undeflected roof up to the inlet of secondary drainage system when the primary drainage system is blocked (i.e., the static head), in inches (mm).
Rain load on the undeflected roof, in psf (kN/m²). When the phrase “undeflected roof” is used, deflections from loads (including dead loads) shall not be considered when determining the amount of rain on the roof.

**Commenter's Reason:** This clarification is made at request of the structural committee. The reason for the proposed change is that the phrase “which is twice the 100-year hourly rainfall rate” that was added in the original proposal is explanatory information as stated and not written in appropriate code language. By removing the phase and adding it as an alternative in a separate sentence, the data within Figure 1611.1 is permitted to be used without compromising the code requirement for design rainfall.
Proposed Change as Submitted

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com)

2015 International Building Code

Delete without substitution:

TABLE 1613.3.3 (2)
VALUES OF SITE COEFFICIENT $F^a$

<table>
<thead>
<tr>
<th>SITE CLASS</th>
<th>MAPPED SPECTRAL RESPONSE ACCELERATION AT 1 SECOND PERIOD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$S_1 = 0.1$</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.7</td>
</tr>
<tr>
<td>D</td>
<td>2.4</td>
</tr>
<tr>
<td>E</td>
<td>3.6</td>
</tr>
<tr>
<td>F</td>
<td>Note b</td>
</tr>
</tbody>
</table>

Notes:
- Use straight line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, $S_1$.
- Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

TABLE 1613.3.3 (1)
VALUES OF SITE COEFFICIENT $F_a^a$

<table>
<thead>
<tr>
<th>SITE CLASS</th>
<th>MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIOD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$S_g ≤ 0.25$</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.2</td>
</tr>
<tr>
<td>D</td>
<td>1.6</td>
</tr>
<tr>
<td>E</td>
<td>2.5</td>
</tr>
<tr>
<td>F</td>
<td>Note b</td>
</tr>
</tbody>
</table>

Notes:
- Use straight line interpolation for intermediate values of mapped spectral response acceleration at short period, $S_g$.
- Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

Reason: Tables 1613.3.3(1)(2) are no longer applicable, because "MAPPED SPECTRAL RESPONSE ACCELERATIONS AT SHORT PERIOD / AT 1-SECOND PERIOD are no longer a part of this code. Soil Site Factors will be handled separately, in the manner consistent with previous practice, as in the 1997 Uniform Building Code. This will create a more physical and logical basis for an important consideration in the engineering design process.

References:
- 1990 SEAOC BLUE BOOK
- 1997 Uniform Building Code
- Robert E. Bachman and David R. Bonneville (2000)

Bibliography: See also BIBLIOGRAPHY in Proposal: Figure 1613.3.1 RISK-TARGETED MCER
Cost Breakdown of Nonstructural Building Elements
Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake.
http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032

Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Permalink: http://dx.doi.org/10.1061/(ASCE)CF.1943-5509.0000630
Read More: http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630

Homeowner's Guide to Earthquake Safety

Retrofitting Questions and Answers
Earthquake Safety, Inc., 2015 (web based)
http://www.earthquakesafety.com/earthquake-retrofitting-faq.html

Cost and Seismic Design
https://www.google.com/#q=cost+and+seismic+design+christopher+arnold+pdf

Earthquake Architecture website
http://www.iitk.ac.in/nicee/wcee/article/14_05-06-0185.PDF

Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee

Cost Impact: Will increase the cost of construction
Will increase the cost of construction
Will, in some cases, not increase the cost of construction
Will, in some cases, increase the cost of construction
Since the mapped maximum considered earthquake spectral response accelerations can fluctuate up-or-down between map editions, this could effect cost accordingly, but I estimate that this would be minor.
This proposal may or may not affect the cost of construction. This is (1) because detached one- and two-family dwellings must be already built to withstand the lateral forces due to wind; and (2) must include basements, "safe rooms"), or other afforded protections to protect occupants against the deadly impacts of hurricanes and tornadoes.
The point is; Detached one- and two-family need to consider the maximum Magnitude of realistic scenario earthquakes that they could, in fact, experience.
And not be constructed vulnerable to earthquakes, because a flawed numerical hazard model "guesses" incorrectly as to the likelihood or possibility of earthquakes. This should remain a rational and a scientific decision based upon protecting both public safety and property. A second point is that "cost" due to structural elements is almost always less than 80% of the cost of a building!
"In general, better seismic performance is achieved through increased lateral design forces (i.e., base shear), and detailing requirements that improve structural connection strength or structural member behavior in the inelastic range of response. Requirements for seismic bracing and anchorage of nonstructural components reduce potential for nonstructural damage and loss of building (or system) functionality."*
* viii, Executive Summary, NIST GCR 14-917-26

In general, where costs might be increased, cost premiums above requirements for wind tend to fall within a range of +1-3%. For cases where seismic requirements would be now additional to what previous codes either applied/neglected/failed to enforce, estimates probably would fall within the range of 0.25 - 1%.
**Public Hearing Results**

**Committee Action:** Disapproved

**Committee Reason:** S₅ and S₁ are not shown as deleted so it be wrong to delete the parameters Fₐ and Fᵥ as proposed.

**Assembly Action:** None

**Individual Consideration Agenda**

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com) requests Approve as Submitted.

Commenter’s Reason: “It is better to be right than to be consistent.”
- Winston Churchill

“If your definition is wrong, you’ll look for the wrong thing.”
- Carol Cleland

“Anyone who conducts an argument by appealing to authority is not using his intelligence; he is just using his memory.”
- Leonardo da Vinci

“Just because it comes from a consensus standard doesn't mean it isn't without problems.”
- Jay Crandell

The Committee Reason for Disapproval is not an argument, but simply a statement of the Logical Fallacy of STRAW MAN.

"The Straw Man fallacy is committed when a person simply ignores a person's actual position and substitutes a distorted, exaggerated or misrepresented version of that position. This sort of "reasoning" has the following pattern:

1. Person A has position X.

2. Person B [Committee Reason] presents position Y (which is a distorted version of X).

3. Person B [Committee Reason] attacks position Y.

4. Therefore X is false/incorrect/flawed.

This sort of ‘reasoning’ is fallacious because attacking a distorted version of a position simply does not constitute an attack on the position itself. One might as well expect an attack on a poor drawing of a person to hurt the person.”
Per Reason Statement of S112-16 "Tables 1613.3.3(1)(2) are no longer applicable, because "MAPPED SPECTRAL RESPONSE ACCELERATIONS AT SHORT PERIOD [$S_s$] / AT 1-SECOND PERIOD [$S_2$] are no longer a part of this code." This is fully explained in S118-16, wherein the RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS Maps are deleted, in favor of newly substituted alternative maps for seismic design.

This recently published article Reality Check: Seismic Hazard Models You Can Trust provides the comprehensive understanding (based on empirical evidence) of why these and other associated proposed changes are needed to provide a more reasonable and more practical approach to seismic safety for the general public that is exposed to major earthquake risk – namely because the "current probabilistic methods to quantify earthquake hazards have serious problems."


Bibliography: ASCE 7 and SEI Standards
click on ASCE 7 cover icon for a preview, including liability disclaimer statement

Response Assessment [Cost Breakdown] of Nonstructural Building Elements

Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake
Eduardo Miranda, Gilberto Mosqueda, Rodrigo Retamales, and Gokhan Pekcan (2012). Performance of Nonstructural
Components during the 27 February 2010 Chile Earthquake, Earthquake Spectra: June 2012, Vol. 28, No. S1, pp. S453-S471. doi: http://dx.doi.org/10.1193/1.4000032 (http://dx.doi.org/10.1193/1.4000032)
http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032
(http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032)

Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Read More: http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630
(http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630)

Homeowner's Guide to Earthquake Safety

Retrofitting Questions and Answers
Earthquake Safety, Inc., 2015 (web based)

Cost and Seismic Design
https://www.google.com/#q=cost+and+seismic+design+christopher+arnold+pdf
(https://www.google.com/#q=cost+and+seismic+design+christopher+arnold+pdf)

Earthquake Architecture website
http://www.iitk.ac.in/nicee/wce/article/14_05-06-0185.PDF (http://www.iitk.ac.in/nicee/w%20ce/article/14_05-06-0185.PDF)

1988 Uniform Building Code

1990 SEAOC BLUE BOOK Recommended Lateral Force Requirements and Commentary

1999 SEAOC BLUE BOOK Recommended Lateral Force Requirements and Commentary

SEAOC Blue Book - Seismic Design Recommendations Preface to the Online Edition
(http://www.seaoc.org/bluebook/index.html)
(http://seaoc.org/system/files/product/efiles/001010Preface.pdf)

Building Codes, Standards and Resource Documents: A Status Report
1997 Uniform Building Code

Proposed Change as Submitted

Proponent: James Bela, self, representing Oregon Earthquake Awareness (sasquake@gmail.com)

2015 International Building Code

Revise as follows:

1613.1 Scope. Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7, excluding Chapter 14 and Appendix 11A. The seismic design category for a structure is permitted to be determined in accordance with Section 1613 or ASCE 7.

Exceptions:

1. Detached one- and two-family dwellings, assigned to Seismic Design Category A, B or C, or located where the mapped short-period spectral response acceleration, $S_0$, is less than 0.4 g.

2. The seismic force-resisting system of wood-frame buildings that conform to the provisions of Section 2308 are not required to be analyzed as specified in this section.

3. Agricultural storage structures intended only for incidental human occupancy.

4. Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances and nuclear reactors.

Reason: SDCs do not realistically reflect the Magnitudes of earthquakes that may impact said “Detached one- and two-family dwellings,” nor their associated real intensities of shaking (accelerations and velocities, including pga and pgv); (2) the contour seismic hazard-model maps, upon which the assigned SDCs are determined, are (a) numerical creations without physical reality; (b) mathematically flawed and incorrect (because a dimensionless number, the probability in one year, is arbitrarily assigned dimensional terms of “per yr.” or annual frequency — leading to the improperly applied notion of a so-called earthquake “return period” as the basis on assigning earthquake design loads; and (c) non-stable between iterative cycles of creations (sometimes varying 25-30% between issues; and (d) SS or Spectral Response Acceleration is both confusing, misunderstood, and most certainly incorrectly interpreted or understood by all of the vast entities (state decision makers, code officials, design professionals, contractors and probably even the preponderance of ICC Committee members as well as Hearings attendees!

For example, see TAKE ME HOME SEISMIC LOADS
Before Earthquake

Damage from M 6.7 Earthquake
Additional damage from aftershocks

Cost breakdown of office buildings, hotels and hospitals

Bibliography: Cost Breakdown of Nonstructural Building Elements

Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake.
doi: http://dx.doi.org/10.1193/1.4000032 (http://dx.doi.org/10.1193/1.4000032)
http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032
Cost Impact: Will not increase the cost of construction

This proposal may or may not affect the cost of construction. This is (1) because detached one- and two-family dwellings must be already built to withstand the lateral forces due to wind; and (2) must include basements, “safe rooms”), or other afforded protections to protect occupants against the deadly impacts of hurricanes and tornadoes.

The point is; Detached one- and two-family need to consider the maximum Magnitude of realistic scenario earthquakes that they could, in fact, experience.

And not be constructed vulnerable to earthquakes, because a flawed numerical hazard model "guesses" incorrectly as to the likelihood or possibility of earthquakes. This should remain a rational and a scientific decision based upon protecting both public safety and property. A second point is that “cost” due to structural elements is almost always less than 80% of the cost of a building!

"In general, better seismic performance is achieved through increased lateral design forces (i.e., base shear), and detailing requirements that improve structural connection strength or structural member behavior in the inelastic range of response. Requirements for seismic bracing and anchorage of nonstructural components reduce potential for nonstructural damage and loss of building (or system) functionality."

* viii, Executive Summary, NIST GCR 14-917-26


NEHRP Consultants Joint Venture A partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering.

1997 Uniform Building Code

Robert E. Bachman and David R. Bonneville (2000)
In general, where costs might be increased, cost premiums above requirements for wind tend to fall within a range of +1-3%. For cases where seismic requirements would be now additional to what previous codes either applied/neglected/failed to enforce, estimates probably would fall within the range of 0.25 - 1%.

---

**Public Hearing Results**

**Committee Action:** Disapproved

**Committee Reason:** The proposed deletion of the exception for detached one- and two-family dwellings was not substantiated.

**Assembly Action:** None

---

**Individual Consideration Agenda**

Proponent : James Bela, representing Oregon Earthquakie Awareness (sasquake@gmail.com) requests Approve as Submitted.

Commenter's Reason: *"It is better to be right than to be consistent."
- Winston Churchill*

*"If your definition is wrong, you'll look for the wrong thing."
- Carol Cleland*

*"Anyone who conducts an argument by appealing to authority is not using his intelligence; he is just using his memory."
- Leonardo da Vinci*

*"Just because it comes from a consensus standard doesn't mean it isn't without problems."
- Jay Crandell*

The Committee Reason for Disapproval is not an argument, but simply a statement of the *Logical Fallacy* of **STRAW MAN**. The **Straw Man** fallacy is committed when a person simply ignores a person's actual position and substitutes a distorted, exaggerated or misrepresented version of that position. This sort of "reasoning" has the following pattern:

1. Person A has position X.
2. Person B [Committee Reason] presents position Y (which is a distorted version of X).
3. Person B [Committee Reason] attacks position Y.
4. Therefore X is false/incorrect/flawed.

This sort of 'reasoning' is fallacious because attacking a distorted version of a position simply does not constitute an attack on the position itself. One might as well expect an attack on a poor drawing of a person to hurt the person."
What is a Straw Man fallacy?

The Straw Man fallacy is committed when a person simply ignores a person's actual position and substitutes a distorted, exaggerated or misrepresented version of that position. This sort of "reasoning" has the following pattern:
- Person A has position X.
- Person B presents position Y (which is a distorted version of X).
- Person B attacks position Y.
- Therefore X is false/incorrect/flawed.

Since "SDCs [seismic design categories] do not realistically reflect the Magnitudes of earthquakes that may impact said "Detached one- and two-family dwellings," nor the associated real intensities of shaking (accelerations and velocities, including \( pga \) and \( pgv \)), per Reason Statement; and further, since \( S_S \) is being deleted, per S118-16 – therefore, exception 1 is no longer applicable. Further substantiation was provided by the reference to Take Me Home . . . Seismic Loads! - shown here below:

TAKE ME HOME . . . SEISMIC LOADS!

[The M 5.8 2011 Virginia earthquake occurred on August 23 at 1:51:04 p.m. local time in the Piedmont region of the US state of Virginia. The epicenter, in Louisa County, was 61 km (38 mi) northwest of Richmond and 8 km (5 mi) south-southwest of the town of Mineral.]

I haven't seen anything regarding Site Class, for Mineral or Louisa VA, as well as the estimated epicentral region of Central Virginia's Piedmont? Cuckoo seems to be the closest built environment to the epicenter (with still an uncertainty: horizontal +/- 2.3 km (1.4 miles); depth +/- 3.1 km (1.9 miles)). No one has officially designated this as the CUCKOO Earthquake. But read below and see if, that term might be better reserved for USGS seismic hazard mapping and U.S. Building Code requirements in both the Central Virginia Seismic Zone and in other known and active seismic zones throughout the Central and Eastern U.S. (CEUS)?

Also, the MMI intensity of earthquake ground shaking (VII - VIII at the estimated epicentral location) was more correctly indicative of SDC D. [ http://www.nibs.org/client/assets/files/bssc/P749/P-749_Chapter5.pdf (http://www.nibs.org/client/assets/files/bssc/P749/P-749_Chapter5.pdf)]

Since 2000, the USGS Seismic Hazard Maps have continued to lower the hazard [SDS = SDS design earthquake spectral response accelerations]:

SS = 0.31g (1997) (2000); 0.26g (2003); 0.22g (2009)
SCB: SDS = 0.21g (1997) (2000); 0.17g (2003); 0.15g (2009).
SCC: SDS = 0.25g (1997) (2000); 0.20g (2003); 0.17g (2009).
SCD: SDS = 0.32g (1997) (2000); 0.27g (2003); 0.23g (2009).]

making building code earthquake provisions less safe regarding both public safety and economic well-being.

These numbers translate to about a 30% decline in design strength (from a low number to an even lower number) in just the last decade! ( for the SDS "Design Earthquake Spectral Response Acceleration Parameter"). A 33% increase in design strength used to be the difference between Seismic Zone 3 and Seismic Zone 4 requirements!
For Site Class B, this now makes the epicentral region of this M 5.8 Virginia (Cuckoo) earthquake Seismic Design Category A (SDC A) - the same as Florida and Michigan (which have no active seismic zones or geologic evidence of mountain building).


This “minor” earthquake now seems to be amongst the most widely felt earthquakes in U.S. history. (i.e., “ever!”) -- “Felt strongly in much of central Virginia and southern Maryland. Felt throughout the eastern US from central Georgia to central Maine and west to Detroit, Michigan and Chicago, Illinois. Felt in many parts of southeastern Canada from Montreal to Windsor.” Source USGS

Clearly we are no longer in Florida, Michigan... or even in Kansas any more!

Too many (a) unsafe conditions and (b) brittle-failure-mode susceptible building products are allowed in the low SDC's A, B, and C - and it defies both logic, engineering judgment, common sense, as well as the professional responsibility of our combined professions. I doubt if any of the brick veneer that separated during this M 5.8 Virginia earthquake would have even been required to be adequately attached for earthquake (lateral force) resistance in these SDC's of A, B and C?

Remember: “The buck stops shear!”

West Virginia, Mountain Mama... Take Me Home... Seismic Loads!... because
http://www.youtube.com/watch?v=oN86d0CdgHQ

"We have nothing to fear but veneer itself!"

"All sciences are vain and full of errors that are not born of Experience, the mother of all Knowledge."

- Leonardo DaVinci

This recently published article Reality Check: Seismic Hazard Models You Can Trust provides the comprehensive understanding (based on empirical evidence) of why these and other associated proposed changes are needed to provide a more reasonable and more practical approach to seismic safety for the general public that is exposed to major earthquake risk -- namely because the “current probabilistic methods to quantify earthquake hazards have serious problems.”

https://eos.org/opinions/reality-check-seismic-hazard-models-you-can-trust

Bibliography: ASCE 7 and SEI Standards
http://www.asce.org/structural-engineering/asce-7-and-sei-standards/
Response Assessment [Cost Breakdown] of Nonstructural Building Elements
(http://peer.berkeley.edu/publications/peer_reports/reports_2003/0305.pdf)

Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake
doi: http://dx.doi.org/10.1193/1.4000032 (http://dx.doi.org/10.1193/1.4000032)
http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032
(http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032)

Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Permalink: http://dx.doi.org/10.1061/(ASCE)CF.1943-5509.0000630 (http://dx.doi.org/10.1061/(ASCE)CF.1943-5509.0000630)
Read More: http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630
(http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630)

Homeowner's Guide to Earthquake Safety

Retrofitting Questions and Answers
Earthquake Safety, Inc., 2015 (web based)

Cost and Seismic Design
https://www.google.com/#q=cost+and+seismic+design+christopher+arnold+pdf
(https://www.google.com/#q=cost+and+seismic+design+christopher+arnold+pdf)

Earthquake Architecture website
http://www.itik.ac.in/nicee/w cee/article/14_05-06-0185.PDF
(http://www.itik.ac.in/nicee/w%20cee/article/14_05-06-0185.PDF)

Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee

1988 Uniform Building Code

1990 SEAOC BLUE BOOK Recommended Lateral Force Requirements and Commentary

1999 SEAOC BLUE BOOK Recommended Lateral Force Requirements and Commentary
SEAOC Seismology Committee. Recommended Lateral Force Requirements and Commentary (Blue Book), Structural
http://www.buildersbook.com/000S99.html

SEAOC BLUE BOOK - Seismic Design Recommendations Preface to the Online Edition
http://www.seaoc.org/bluebook/index.html

Building Codes, Standards and Resource Documents: A Status Report
http://skghoshassociates.com/sk_publication/PCI_March02_bldg_codes_stand.pdf

1997 Uniform Building Code


Seismic Design Categories
Proposed Change as Submitted

Proponent: Jennifer Goupil, AMERICAN SOCIETY OF CIVIL ENGINEERS, representing SELF (jgoupil@asce.org)

2015 International Building Code

Revise as follows:

1613.1 Scope. Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7, excluding Chapter 14, 11, 12, 13, 15, 17, and Appendix 11A 18, as applicable. The seismic design category for a structure is permitted to be determined in accordance with Section 1613 or ASCE 7.

Exceptions:

1. Detached one- and two-family dwellings, assigned to Seismic Design Category A, B or C, or located where the mapped short-period spectral response acceleration, $S_S$, is less than 0.4 g.
2. The seismic force-resisting system of wood-frame buildings that conform to the provisions of Section 2308 are not required to be analyzed as specified in this section.
3. Agricultural storage structures intended only for incidental human occupancy.
4. Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances and nuclear reactors.

1613.3.2 Site class definitions. Based on the site soil properties, the site shall be classified as Site Class A, B, C, D, E or F in accordance with Chapter 20 of ASCE 7.

Where the soil properties are not known in sufficient detail to determine the site class, Site Class D, subjected to the requirements of Section 1613.3.3, shall be used unless the building official or geotechnical data determines that Site Class E or F soils are present at the site.

For situations in which site investigations, performed in accordance with Chapter 20 of ASCE 7, reveal rock conditions consistent with Site Class B, but site-specific velocity measurements are not made, the site coefficients $F_a$ and $F_v$ shall be taken at unity (1.0).

1613.3.3 Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. The maximum considered earthquake spectral response acceleration for short periods, $S_{MS}$, and at 1-second period, $S_{M1}$, adjusted for site class effects shall be determined by Equations 16-37 and 16-38, respectively:

$$S_{MS} = F_a S_S \quad \text{(Equation 16-37)}$$
$$S_{M1} = F_v S_1 \quad \text{(Equation 16-38)}$$

but $S_{MS}$ shall not be taken less than $S_{M1}$ except when determining Seismic Design Category in accordance with Section 1613.3.5.

where:

- $F_a$ = Site coefficient defined in Table 1613.3.3(1).
- $F_v$ = Site coefficient defined in Table 1613.3.3(2).
- $S_S$ = The mapped spectral accelerations for short periods as determined in Section 1613.3.1.
- $S_1$ = The mapped spectral accelerations for a 1-second period as determined in Section 1613.3.1.

Where Site Class D is selected as the default site class per Section 1613.3.2, the value of $F_a$ shall not be less than 1.2.

Where the simplified design procedure of ASCE 7 Section 12.14 is used, the value of $F_a$ shall be determined in accordance with ASCE 7 Section 12.14.8.1, and the values of $F_v$, $S_{MS}$, and $S_{M1}$ need not be determined.
The mapped spectral accelerations for short periods as determined in Section 1613.3.1.

The mapped spectral accelerations for a 1-second period as determined in Section 1613.3.1.

### TABLE 1613.3.3 (2)
VALUES OF SITE COEFFICIENT $F_a$

<table>
<thead>
<tr>
<th>SITE CLASS</th>
<th>MAPPED RISK TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE$_R$) SPECTRAL RESPONSE ACCELERATION PARAMETER AT 1-SECOND PERIOD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$S_1 \leq 0.1$</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>0.84</td>
</tr>
<tr>
<td>C</td>
<td>1.54</td>
</tr>
<tr>
<td>D</td>
<td>2.4</td>
</tr>
<tr>
<td>F</td>
<td>Note b</td>
</tr>
</tbody>
</table>

- Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, $S_1$.
- Values shall be determined in accordance with Section 11.4.7 of ASCE 7.
- See requirements for site-specific ground motions in Section 11.4.7 of ASCE 7.

### TABLE 1613.3.3 (1)
VALUES OF SITE COEFFICIENT $F_a$

<table>
<thead>
<tr>
<th>SITE CLASS</th>
<th>MAPPED RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE$_R$) SPECTRAL RESPONSE ACCELERATION PARAMETER AT SHORT PERIOD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$S_5 \leq 0.25$</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>0.94</td>
</tr>
<tr>
<td>C</td>
<td>1.34</td>
</tr>
<tr>
<td>D</td>
<td>1.6</td>
</tr>
<tr>
<td>E</td>
<td>2.43</td>
</tr>
<tr>
<td>F</td>
<td>Note b</td>
</tr>
</tbody>
</table>

- Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, $S_5$.
- Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

Delete without substitution:

#### 1613.4 - Alternatives to ASCE 7
The provisions of Section 1613.4 shall be permitted as alternatives to the relevant provisions of ASCE 7.

#### 1613.4.1 - Additional seismic force-resisting systems for seismically isolated structures
Add the following exception to the end of Section 17.5.4.2 of ASCE 7:

- **Exception:** For isolated structures designed in accordance with this standard, the structural system limitations including structural height limits, in Table 12.2.1 for ordinary steel concentrically braced frames (OCBFs) as defined in Chapter 11 and ordinary moment frames (OMFs) as defined in Chapter 11 are permitted to be taken as 160 feet (48 768 mm) for structures assigned to Seismic Design Category D, E or F, provided that the following conditions are satisfied:
  1. The value of $R_i$ as defined in Chapter 17 is taken as 1.
  2. For OMFs and OCBFs, design is in accordance with AISC 341.

#### 1613.5 - Amendments to ASCE 7
The provisions of Section 1613.5 shall be permitted as an amendment to the relevant provisions of ASCE 7.

#### 1613.5.1 - Transfer of anchorage forces into diaphragm
Modify ASCE 7 Section 12.11.2.2.1 as follows:

12.11.2.2.1 Transfer of anchorage forces into diaphragm. Diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorage forces into the diaphragms. Diaphragm connections shall be positive.
mechanical or welded. Added chords are permitted to be used to form subdiaphragms to transmit the anchorage forces to the main continuous cross ties. The maximum length-to-width ratio of a wood, wood structural panel or untopped steel deck sheathed structural subdiaphragm that serves as part of the continuous tie system shall be 2.5 to 1. Connections and anchorages capable of resisting the prescribed forces shall be provided between the diaphragm and the attached components. Connections shall extend into the diaphragm a sufficient distance to develop the force transferred into the diaphragm.

1613.6 Ballasted photovoltaic panel systems. Ballasted, roof-mounted photovoltaic panel systems need not be rigidly attached to the roof or supporting structure. Ballasted non-penetrating systems shall be designed and installed only on roofs with slopes not more than one unit vertical in 12 units horizontal. Ballasted nonpenetrating systems shall be designed to resist sliding and uplift resulting from lateral and vertical forces as required by Section 1605, using a coefficient of friction determined by acceptable engineering principles. In structures assigned to Seismic Design Category C, D, E or F, ballasted non-penetrating systems shall be designed to accommodate seismic displacement determined by nonlinear response history analysis or shake-table testing, using input motions consistent with ASCE 7 lateral and vertical seismic forces for nonstructural components on roofs.

Reason:
This proposal is a coordination proposal to bring the 2018 IBC up to date with the provisions of the 2016 edition of ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7-16).

Section 1613.1 Scope - These proposed changes reflect the current provisions within ASCE 7-16, Appendix 11A was removed and instead of excluding any particular chapters, this proposed change call out the primary ASCE 7 chapters that charge specific parts of the design process. These chapters, in turn, reference other ASCE 7 Sections, other ASCE 7 Chapters,m and other standards for portions of the requirements. All needed provisions of ASCE 7 are incorporated, including ground motions.

Section 1613.3.3.3 Site coefficient and adjusted maximum consider earthquake spectral response acceleration parameters - The site coefficients contained in the IBC date back to studies performed in the early 1990. These site coefficients were inherently tied to the attenuation relationships that we used by USGS to develop the MCE spectral acceleration maps used by the code in that era. The USGS maps contained in the 2010 edition of the ASCE 7 as well as IBC 2012 and th 2015 based on an updated set of attenuation relationships known as the NGA equations. The old site class coefficients are not appropriate for use with ground motions derived using the NGA equations. Note that a separate proposal has been submitted by the Building Seismic Safety Council’s (BSSC) Code Resource Support Committee (CRSC) to update the maps, based on those contained in the 2014 National Hazard Reduction Program (NEHRP) Provision and also ASCE 7-16. These updated maps also based on the NGA equations.

The BSSC Provisions Update Committee (PUC) performed extensive study of the appropriate site class coefficients to use with the NGA-derived ground motions and adopted the values introduced into Tables 1613.3.3(1) and 1613.3.3(2) in this proposal into the 2014 NEHRP Provisions. ASCE 7-16 subsequently adopted these updated values. This proposal brings the IBC into uniformity with the NEHRP Provisions and ASCE 7 and deletes the incorrect coefficients that are contained in IBC 2012 and 2015.

Section 1613.3.4 Design spectral response acceleration parameters - In developing the updated site class coefficients contained in the updated Tables 1613.3.3(1) and 1613.3.3(2), BSSC discovered that the standard spectral shape derived using the $S_{PS}$ and $S_{D1}$ parameters is unconservative for the design of long period buildings ($T > 1$ second) located on Site Class D or softer sites, when the seismic hazard is dominated by large magnitude earthquakes. This proposal adopts language developed by the BSSC PUC for the 2014 NEHRP Provisions and adopted by ASCE 7-16 that requires the use of site-specific spectra to represent ground motions for such buildings.

Section 1613.4 Alternatives to ASCE 7 - The proposed changes deletes the provisions in IBC because this material is now included in the 2016 edition of ASCE 7.

Section 1613.5 Amendments to ASCE 7 - The proposed changes deletes the provisions in the IBC because this material is now included in the 2016 edition of ASCE 7.

Cost Impact: Will not increase the cost of construction.

The proposed changes will not impact the cost of construction. This proposal coordinates the IBC with the referenced loading standard ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE 7 will be updated from the 2010 edition to the 2016 edition as an Administrative Update to the 2018 I-Codes.

As of the submission date of this code change proposal, the ASCE 7 Standards Committee has completed the committee balloting on the technical changes. The document designated ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures is expected to be completed, published, and available for purchase prior to the ICC Public Comment Hearings for Group B in October of 2016. Any person interested in obtaining a public comment copy of ASCE 7-16 may do so by contacting James Neckel at ASCE (jneckel "at" asce.org).
Committee Action: Approved as Modified

Modification:

2015 International Building Code

1613.1 Scope. Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7 Chapter 11, 12, 13, 15, 17, and 18, as applicable. The seismic design category for a structure is permitted to be determined in accordance with Section 1613 or ASCE 7.

- Exceptions:
  1. Detached one- and two-family dwellings, assigned to Seismic Design Category A, B or C, or located where the mapped short-period spectral response acceleration, $S_S$, is less than 0.4 g.
  2. The seismic force-resisting system of wood-frame buildings that conform to the provisions of Section 2308 are not required to be analyzed as specified in this section.
  3. Agricultural storage structures intended only for incidental human occupancy.
  4. Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances and nuclear reactors.
  5. References within ASCE 7 to Chapter 14 shall not apply, except as specifically required herin.

1613.6 Ballasted photovoltaic panel systems. Ballasted, roof-mounted photovoltaic panel systems need not be rigidly attached to the roof or supporting structure. Ballasted non-penetrating systems shall be designed and installed only on roofs with slopes not more than one unit vertical in 12 units horizontal. Ballasted nonpenetrating systems shall be designed to resist sliding and uplift resulting from lateral and vertical forces as required by Section 1605, using a coefficient of friction determined by acceptable engineering principles. In structures assigned to Seismic Design Category C, D, E or F, ballasted nonpenetrating systems shall be designed to accommodate seismic displacement determined by nonlinear response-history analysis or shake-table testing, using input motions consistent with ASCE 7 lateral and vertical seismic forces for nonstructural components on roofs.

Committee Reason: This code change updates the IBC seismic load provisions for consistency with the latest edition of the referenced standard, ASCE 7, which was updated in ADM94-16. The modification retains the prior exclusion of Chapter 14 in ASCE 7 and also retains the IBC requirements for ballasted photovoltaic panel since no evidence was given indicating that they are incorrect.

Assembly Action: None

Public Comment 1:

Proponent: Jennifer Goupil, American Society of Civil Engineers (ASCE), representing American Society of Civil Engineers (ASCE) (jgoupil@asce.org) requests Approve as Modified by this Public Comment.

Further Modify as follows:

2015 International Building Code

1613.3.3 Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. The maximum considered earthquake spectral response acceleration for short periods, $S_{MS}$, and at 1-second period, $S_{M1}$, adjusted for site class effects shall be determined by Equations 16-37 and 16-38, respectively:

$S_{MS} = F_a S_{MS}$ (Equation 16-37)

$S_{M1} = F_a S_{M1}$ (Equation 16-38)

but $S_{MS}$ shall not be taken less than $S_{MS}$ except when determining Seismic Design Category in accordance with Section 1613.3.5.

where:
\[ F_a = \text{Site coefficient defined in Table 1613.3.3(1).} \]

\[ F_{Vs} = \text{Site coefficient defined in Table 1613.3.3(2).} \]

\[ S_S = \text{The mapped spectral accelerations for short periods as determined in Section 1613.3.1.} \]

\[ S_1 = \text{The mapped spectral accelerations for a 1-second period as determined in Section 1613.3.1.} \]

Where Site Class D is selected as the default site class per Section 1613.3.2, the value of \( F_a \) shall not be less than 1.2.

Where the simplified design procedure of ASCE 7 Section 12.14 is used, the value of \( F_a \) shall be determined in accordance with ASCE 7 Section 12.14.8.1, and the values of \( F_{Vs} \), \( S_{MS} \), and \( S_{M1} \) need not be determined.

<table>
<thead>
<tr>
<th>SITE CLASS</th>
<th>MAPPED RISK TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE) SPECTRAL RESPONSE ACCELERATION PARAMETER AT 1-SECOND PERIOD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( S \leq 0.1 )</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>0.8</td>
</tr>
<tr>
<td>C</td>
<td>1.5</td>
</tr>
<tr>
<td>D</td>
<td>2.4</td>
</tr>
<tr>
<td>E</td>
<td>4.2</td>
</tr>
<tr>
<td></td>
<td>( b2)(^b)(^c)</td>
</tr>
<tr>
<td>F</td>
<td>Note ( b )</td>
</tr>
</tbody>
</table>

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, \( S \).

b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

c. See requirements for site-specific ground motions in Section 11.4.7 of ASCE 7.

**Commenter's Reason:** This change is required to coordinate with ASCE 7-16. This clarification was addressed during the ASCE 7-16 public comment and is incorporated into the final version of the standard.

The deleted clause in Section 1613.3.3 will never govern for Site Classes A, B, and C, but if not deleted it will conflict with the proposed revision to ASCE 7-16 Section 12.1.2 and 21.3. To eliminate any confusion, the \( F_a \) site coefficients in ASCE 7-16 Table 11.4-2 were removed for Site Class E site conditions since these values are not used in the standard. The table value and the reference to Note c is replaced with Note b for a direct reference to ASCE 7-16 Section 11.4.7. And because Table 1613.3.3 (2) is duplicative and identical to the Table 11.4-2 in ASCE 7-16, these values need to be adjusted in IBC as well. The values for Site Class D site conditions are retained in the table since they are needed for the exceptions specified in Section ASCE 7-16 11.4.7.

**Proponent:** Ed Berkel, representing ICC Code Correlation Committee (ccc@iccsafe.org) requests Disapprove.

**Commenter’s Reason:** The Code Correlation Committee requests Disapproval of this code change proposal in order to bring a correlation issue to the attention of the full membership at the Public Comment Hearings and to allow the membership to coordinate action on this code change proposal with action taken on Code Change Proposal ADM94-16.

ADM94-16 is the administrative update to referenced standards in the I-Codes. One of these standards, ASCE7, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, was proposed for update to ASCE7-16. However, a successful assembly motion requests that the referenced ASCE7 remain at ASCE7-10 as it presently is referenced in the 2015 I-Codes. This code change proposal coordinates with and relies upon reference to ASCE7-16.
The Code Correlation Committee is a standing committee of the International Code Council whose objectives, procedures and organization are set forth in Council Policy CP#44-13. The objective of the Code Correlation Committee is to maintain technical and editorial consistency among the International Codes and to assist staff in the evaluation and processing of code change proposals and comments that are exclusively editorial.
Proposed Change as Submitted

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com)

2015 International Building Code

Revise as follows:

1613.3 Seismic ground motion Earthquake base shear coefficient values. Seismic ground motion Earthquake base shear coefficient values shall be determined in accordance with this section.

Reason:
To better clarify that the code is anchoring the lateral strength resistance (or base shear) requirement for design to a more scientific and systematically consistent scenario earthquake magnitude criteria; and, is therefore, also not downgrading the lateral design strength or connection detailing requirements based upon a hazard model, which is largely a numerical creation heavily dependent on the perceived likelihoods of earthquake occurrences, as a controlling factor.

PARADIGMS LOST: When Hazard Models are predicated on logical fallacies like “Texas Sharpshooting” and not on scientific facts

“. . . BUILDINGS AND OTHER CRITICAL STRUCTURES SHOULD BE DESIGNED CAPABLE TO RESIST FUTURE EARTHQUAKES. When an earthquake with a given magnitude M occurs, it causes a specific ground shaking that certainly does not take into account whether the event is rare or not; thus ground motion parameters for seismic design should not be scaled depending on earthquake recurrence. Accordingly, in a cost-effective prevention perspective, when considering two sites prone to earthquakes with the same magnitude M, given that all the remaining conditions are the same, the site where large earthquakes are more sporadic appears naturally preferable for new settlements. Nevertheless parameters in seismic design must be equal at the two sites, since the expected magnitude is the same. In fact, although the shaking is more likely at one site, an element that should be factored in insurance, we favor building to the same standard to promote public safety [and community resilience], since there are no evidences that what happened in the past cannot repeat in the near future, or that it will occur only after a very long time.”

- Peresan and Panza (2012)

"The Texas sharpshooter fallacy takes its name from a gunman who shoots at a side of a barn, only later to draw targets around a cluster of points that were hit. The gunman didn’t aim for the target specifically (instead aiming for the barn), but outsiders might believe that he meant to hit the target."

http://www.investopedia.com/terms/t/texas-sharpshooter-fallacy.asp

Bibliography:
FORUM: Improving Earthquake Hazard Assessments in Italy: An Alternative to “Texas Sharpshooting.”
Eos Vol. 93, No. 51 18 December 2012

“Texas Sharpshooter” Fallacy
http://www.investopedia.com/terms/t/texas-sharpshooter-fallacy.asp

Earthquake Magnitude Scale and Class
http://www.geo.mtu.edu/UPSeis/magnitude.html

M 5.8 Aug 11, 2011 Mineral VA Earthquake
The Mw 5.8 Virginia Earthquake of August 23, 2011

Residential and Building Damage near epicenter: M 5.8 Mineral, Virginia EQ MMI VII – VIII
Louisa County High School Building Damage
Damage from M 6.0 Wells, Nevada EQ 2008
https://disastersafety.org/ibhs-risks-earthquake/
Cost Breakdown of Nonstructural Building Elements

Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake.
doi:http://dx.doi.org/10.1193/1.4000032
http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032

Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Permalink: http://dx.doi.org/10.1061/(ASCE)CF.1943-5509.0000630
Read More: http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630

Homeowner's Guide to Earthquake Safety

Retrofitting Questions and Answers
Earthquake Safety, Inc., 2015 (web based)
http://www.earthquakesafety.com/earthquake-retrofitting-faq.html

Cost and Seismic Design
https://www.google.com/#q=cost+and+seismic+design+christopher+arnold+pdf

Earthquake Architecture website
http://www.iitk.ac.in/nicee/wcee/article/14_05-06-0185.PDF

Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee

Cost Impact: Will not increase the cost of construction
These are changes in terminology, for the purpose of clarifying both the intent of the code and the practice of earthquake engineering.
This proposal may or may not affect the cost of construction. This is (1) because detached one- and two-family dwellings must be already built to withstand the lateral forces due to wind; and (2) must include basements, "safe rooms"), or other afforded protections to protect occupants against the deadly impacts of hurricanes and tornadoes.

The point is; Detached one- and two-family need to consider the maximum Magnitude of realistic scenario earthquakes that they could, in fact, experience. And they should not be constructed vulnerable to earthquakes, because a flawed numerical hazard model "guesses" incorrectly as to the likelihood or possibility of earthquakes. This should remain a rational and a scientific decision based upon protecting both public safety and property. A second point is that "cost" due to structural elements is almost always less than 80% of the cost of a building!

"In general, better seismic performance is achieved through increased lateral design forces (i.e., base shear), and detailing requirements that improve structural connection strength or structural member behavior in the inelastic range of response. Requirements for seismic bracing and anchorage of nonstructural components reduce potential for nonstructural damage and loss of building (or system) functionality."*

* viii, Executive Summary, NIST GCR 14-917-26

NEHRP Consultants Joint Venture A partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering.

In general, where costs might be increased, cost premiums above requirements for wind tend to fall within a range of +1-3%. For cases where seismic requirements would be now additional to what previous codes either applied/neglected/failed to enforce, estimates probably would fall within the range of 0.25 - 1%.

Committee Reason: This proposed change would cause a disconnect between various sections of the IBC as well as create conflicts with the referenced standard, ASCE7.

Commenter's Reason: "It is better to be right than to be consistent."
- Winston Churchill

"If your definition is wrong, you'll look for the wrong thing."
- Carol Cleland

"Anyone who conducts an argument by appealing to authority is not using his intelligence; he is just using his memory."
- Leonardo da Vinci

"Just because it comes from a consensus standard doesn't mean it isn't without problems."
- Jay Crandell

"It is difficult to get a man to understand something, when his salary depends upon his not understanding it!"
- Upton Sinclair
The Committee Reason for Disapproval is not an argument, but simply a statement of the Logical Fallacy of SLIPPERY SLOPE. “You said that if we allow A to happen, then Z will eventually happen too, therefore A should not happen. The problem with this reason is that it avoids engaging with the issue at hand, and instead shifts attention to extreme hypotheticals. Because no proof is presented to show that such extreme hypotheticals will in fact occur, this fallacy has the form of an APPEALto EMOTIONFallacy by leveraging fear. In effect the argument at hand is unfairly tainted by unsubstantiated conjecture.”

And neither is the Committee Reason correct, since, in fact, no “disconnects” or “conflicts with the referenced standard, ASCE 7” actually do occur. This is general language necessary to be in harmony with the new Figures 1613.3.1(1) (New) and 1613.3.1(2) (New) – as described in S118-16; as well as to guide the design professional with more specific understanding as to what he is really applying in the seismic design process. Furthermore, any potential conflicts between “this code” and a “referenced standard” are long-understood to be then resolved in favor of the code – which governs!

"When the code has specific requirements that vary from those found in a referenced standard, the requirements of the code take precedence over the standard."

ICC REFERENCED STANDARDS GUIDE (2006), p. 3

Besides, how can you not question ASCE 7, given that, really, nothing is more slippery than its broad-speaking and really irresponsible Disclaimer (“disclaiming any and all liability”) below:

“This standard was developed by a consensus standards development process . . .

“While ASCE's process is designed to promote standards that reflect a fair and reasoned consensus among all interested participants, while preserving the public health, safety, and welfare that is paramount to its mission, it has not made an independent assessment of and does not warrant the accuracy, completeness, suitability, or utility of any information, apparatus, product, or process discussed herein. ASCE [BUMMER!]
does not intend, nor should anyone interpret, ASCE's standards to replace the sound judgment of a competent professional, having knowledge and experience in the appropriate field(s) of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the contents of this standard.

ASCE has no authority to enforce compliance with its standards and does not undertake to certify products for compliance or to render any professional services to any person or entity.

ASCE disclaims any and all liability for any personal injury, property damage, financial loss or other damages of any nature whatsoever, including without limitation any direct, indirect, special, exemplary, or consequential damages resulting from any person's use of, or reliance on, this standard. Any individual who relies on this standard assumes full responsibility for such use.

ASCE and American Society of Civil Engineers---Registered in U.S. Patent and Trademark Office
In fact, ASCE 7 does not really meet the intended definition of an actual Standard – and it is really more of a “Guideline” or an “Alternate Method,” than it is a true Standard. It’s drafting committees, “often in error, but never in doubt,” seem more committed to business-as-usual and “the need not to know” than they are to sound engineering principles (like correct mathematics) and also to logical and defensible thinking. “Do No Harm” and “public safety” are, evidently, non-persuasive!

Confucius say: “To see what is right and not to do it, is lack of courage.” And this reality should not be confused with “disconnect”-edness!

This really minor change in language, to more clear and more specific meaning, is more than adequately explained in the Proposal Reason Statement:

“To better clarify that the code is anchoring the lateral strength resistance (or base shear) requirement for design to a more scientific and systematically consistent scenario earthquake magnitude criteria; and, is therefore, also not downgrading the lateral design strength or connection detailing requirements based upon a hazard model, which is largely a numerical creation heavily dependent on the perceived likelihoods of earthquake occurrences, as a controlling factor.”

The Problem with ASCE 7’s Earthquake Hazard Model

Remember the quote by Upton Sinclair...

“It is difficult to get a man to understand something, when his salary depends upon his not understanding it!”

“All sciences are vain and full of errors that are not born of Experience, the mother of all Knowledge.”  
- Leonardo da Vinci

This recently published article Reality Check: Seismic Hazard Models You Can Trust provides the comprehensive understanding (based on empirical evidence) of why these and other associated proposed changes are needed to provide a more reasonable and more practical approach to seismic safety for the general public that is exposed to major earthquake risk.
namely because the "current probabilistic methods to quantify earthquake hazards have serious problems."
https://eos.org/opinions/reality-check-seismic-hazard-models-you-can-trust

**Improving Earthquake Hazard Assessments** . . . is the realizable goal of this Code Change Proposal

" . . . BUILDINGS AND OTHER CRITICAL STRUCTURES SHOULD BE DESIGNED CAPABLE TO RESIST FUTURE EARTHQUAKES. When an earthquake with a given magnitude M occurs, it causes a specific ground shaking that certainly does not take into account whether the event is rare or not; thus ground motion parameters for seismic design should not be scaled depending on earthquake recurrence. Accordingly, in a cost-effective prevention perspective, when considering two sites prone to earthquakes with the same magnitude M, given that all the remaining conditions are the same, the site where large earthquakes are more sporadic appears naturally preferable for new settlements. Nevertheless parameters in seismic design must be equal at the two sites, since the expected magnitude is the same. In fact, although the shaking is more likely at one site, an element that should be factored in insurance, we favor building to the same standard to promote public safety [and community resilience], since there are no evidences that what happened in the past cannot repeat in the near future, or that it will occur only after a very long time."

- Bibliography:

FORUM: Improving Earthquake Hazard Assessments in Italy: An Alternative to “Texas Sharpshooting”

ASCE 7 and SEI Standards
http://www.asce.org/structural-engineering/asce-7-and-sei-standards/

Definition of “Texas Sharpshooter Fallacy”
"The Texas sharpshooter fallacy takes its name from a gunman who shoots at a side of a barn, only later to draw targets around a cluster of points that were hit. The gunman didn’t aim for the target specifically (instead aiming for the barn), but outsiders might believe that he meant to hit the target.”
http://www.investopedia.com/terms/t/texas-sharpshooter-fallacy.asp

Earthquake Magnitude Scale and Class
http://www.geo.mtu.edu/UPSeis/magnitude.html

M 5.8 Aug 23, 2011 Mineral VA Earthquake
https://www2.usgs.gov/blogs/features/usgs_top_story/one-year-anniversary-magnitude-5-8-virginia-earthquake/

The Mw 5.8 Virginia Earthquake of August 23, 2011

Residential and Building Damage near epicenter: M 5.8 Mineral, Virginia EQ MMI VII – VIII
Louisa County High School Building Damage
(http://www.eqclearinghouse.org/2011-08-23-virginia/2011/09/02/louisa-county-high-school-damage/)

Damage from M 6.0 Wells, Nevada EQ 2008
https://www.google.com/search?q=wells+nevada+earthquake+2008&biw=1280&bih=899&tbs=isch&tbo=u&source=univ&sa=X&ved=0ahUKEwjO7KmNo__JAhULzGMK

IBHS Earthquake

Response Assessment [Cost Breakdown] of Nonstructural Building Elements
(http://peer.berkeley.edu/publications/peer_reports/reports_2003/0305.pdf)

Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake
http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032
(http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032%20L)

Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Read More: http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630
(http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630)

Homeowner’s Guide to Earthquake Safety

Retrofitting Questions and Answers
Earthquake Safety, Inc., 2015 (web based)

Cost and Seismic Design
https://www.google.com/#q=cost+and+seismic+design+christopher+arnold+pdf
(https://www.google.com/#q=cost+and+seismic+design+christopher+arnold+pdf)

Earthquake Architecture website
http://www.iit.ac.in/nicee/wcee/article/14_05-06-0185.PDF (http://www.iit.ac.in/nicee/w%20cee/article/14_05-06-0185.PDF)

Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee

1988 Uniform Building Code
IBC: 1613.3.1, 1613.3.1(1) (New), 1613.3.1(2) (New).

Proposed Change as Submitted

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com)

2015 International Building Code

FIGURE 1613.3.1 (1)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE\textsubscript{r}) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5\% OF CRITICAL DAMPING), SITE CLASS B

FIGURE 1613.3.1 (2)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE\textsubscript{r}) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 1-SECOND SPECTRAL RESPONSE ACCELERATION (5\% OF CRITICAL DAMPING), SITE CLASS B

FIGURE 1613.3.1 (3)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE\textsubscript{r}) GROUND MOTION RESPONSE ACCELERATIONS FOR HAWAII OF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5\% OF CRITICAL DAMPING), SITE CLASS B

Delete and substitute as follows:

1613.3.1 (4)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE\textsubscript{r}) GROUND MOTION RESPONSE ACCELERATIONS FOR ALASKA OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5\% OF CRITICAL DAMPING), SITE CLASS B
FIGURE 1613.3.1 (5)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR ALASKA OF 1.0-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

(Existing code figure not shown for clarity)

FIGURE 1613.3.1 (6)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR PUERTO RICO AND THE UNITED STATES VIRGIN ISLANDS OF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

FIGURE 1613.3.1 (7)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR GUAM AND THE NORTHERN MARIANA ISLANDS OF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

FIGURE 1613.3.1 (8)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR AMERICAN SAMOA OF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

2016 ICC PUBLIC COMMENT AGENDA
DAMPING), SITE CLASS B

FIGURE 1613.3.1(1)
LATERAL DESIGN STRENGTH (BASE SHEAR) COEFFICIENT EXPRESSED AS SEISMIC ZONES 0-4 1994/1997 UBC

Reason: "Any problem, no matter how complicated, can be made simple; if looked at in the right way."

- Prof. Theodore Von Karman
  Caltech, Engineering Mechanics – 1930s

"Any problem, no matter how complicated, can be made still more complicated; if looked at in the right way!"

- Prof. George W. Housner
  Caltech, Earthquake Engineering – 1992

(https://www.eeri.org/site/images/projects/oralhistory/housner.pdf)


"I NEVER USE WORDS IN A STORY, THAT I DON’T KNOW WHAT THEY MEAN!"

- Lou Costello

2016 ICC PUBLIC COMMENT AGENDA  Page 2315
"In general, better seismic performance is achieved through increased lateral design forces (i.e., base shear), and detailing requirements that improve structural connection strength or structural member behavior in the inelastic range of response. Requirements for seismic bracing and anchorage of nonstructural components reduce potential for nonstructural damage and loss of building (or system) functionality."


Let's look at the basic problems:

**Why Earthquake Hazard Maps often fail and what to do about it . . .**

Why Earthquake Hazard Maps often fail an what to do about it -
http://web.missouri.edu/~lium/pdfs/Papers/seth2012-tecto-hazardmap.pdf

"The 2011 Tohoku earthquake is another striking example – after the 2008 Wenchuan and 2010 Haiti earthquakes – of highly destructive earthquakes that occurred in areas predicted by earthquake hazard maps to be relatively safe. Here, we examine what went wrong for Tohoku, and how this failure illustrates limitations of earthquake hazard mapping. We use examples from several seismic regions to show that earthquake occurrence is typically more complicated than the models on which hazard maps are based, and that the available history of seismicity is almost always too short to reliably establish the spatiotemporal pattern of large earthquake occurrence. As a result, key aspects of hazard maps often depend on poorly constrained parameters, whose values are chosen based on the mapmakers' preconceptions. When these are incorrect, maps do poorly. This situation will improve at best slowly, owing to our limited understanding of earthquake processes. However, because hazard mapping has become widely accepted and used to make major decisions, we suggest two changes to improve current practices. First, the uncertainties in hazard map predictions should be assessed and clearly communicated to potential users. Recognizing the uncertainties would enable users to decide how much credence to place in the maps and make them more useful in formulating cost-effective hazard mitigation policies. Second, hazard maps should undergo rigorous and objective testing to compare their predictions to those of null hypotheses, including ones based on uniform regional seismicity or hazard. Such testing, which is common and useful in similar fields, will show how well maps actually work and hopefully help produce measurable improvements. There are likely, however, limits on how well hazard maps can ever be made because of the intrinsic variability of earthquake processes.” (Stein et. al. 2012)

**PSHA: Is it Science?**
PSHA: is it science? (https://www.researchgate.net/publication/238378491_PSHA_is_it_science)
Probabilistic seismic hazard analysis (PSHA) is beginning to be seen as unreliable. The problem with PSHA is that its data are inadequate and its logic is defective. Much more reliable, and more scientific, are deterministic procedures, especially when coupled with engineering judgment. (Castaños and Lomnitz 2002) DOI: 10.1016/S0013-7952(02)00039-X

**Why are the Standard Probabilistic Methods of Estimating Seismic Hazard and Risks Too Often Wrong?**

According to the probabilistic seismic hazard analysis (PSHA) approach, the deterministically evaluated or historically defined largest credible earthquakes (often referred to as Maximum Credible Earthquakes, MCEs) are “an unconvincing possibility” and are treated as “likely impossibilities” [since PSHA assumes “the risk quickly decreases as the ground motion intensity increases.”] within individual seismic zones. However, globally over the last decade such events keep occurring where PSHA predicted seismic hazard to be low. (Panza et. al. 2014)
According to the probabilistic seismic hazard analysis (PSHA) approach, the deterministically evaluated or historically defined largest credible earthquakes (often referred to as Maximum Credible Earthquakes, MCEs) are “an unconvincing possibility” and are treated as “likely impossibilities” within individual seismic zones. However, globally over the last decade such events keep occurring where PSHA predicted seismic hazard to be low. Systematic comparison of the observed ground shaking with the expected one reported by the Global Seismic Hazard Assessment Program (GSHAP) maps discloses gross underestimation worldwide. Several inconsistencies with available observation are found also for national scale PSHA maps (including Italy), developed using updated data sets. As a result, the expected numbers of fatalities in recent disastrous earthquakes have been underestimated by these maps by approximately two to three orders of magnitude. The total death toll in 2000–2011 (which exceeds 700,000 people, including tsunami victims) calls for a critical reappraisal of GSHAP results, as well as of the underlying methods.

In this chapter, we discuss the limits in the formulation and use of PSHA, addressing some theoretical and practical issues of seismic hazard assessment, which range from the overly simplified assumption that one could reduce the tensor problem of seismic-wave generation and propagation into a scalar problem (as implied by ground motion prediction equations), to the insufficient size and quality of earthquake catalogs for a reliable probability modeling at the local scale. Specific case studies are discussed, which may help to better understand the practical relevance of the mentioned issues. The aim is to present a critical overview of different approaches, analyses, and observations in order to provide the readers with some general considerations and constructive ideas toward improved seismic hazard and effective risk assessment. Specifically, we show that seismic hazard analysis based on credible scenarios for real earthquakes, defined as neo-deterministic seismic hazard analysis, provides a robust alternative approach for seismic hazard and risk assessment. Therefore, it should be extensively tested as a suitable method for formulating scientifically sound and realistic public policy and building code practices.

"Yes, I'm bein' followed by a moon shadow."

~ Cat Stevens

"The map is not the territory"

~ Alfred Korzybski

"The 'map' is what we think resembles reality, and we should use it as a guide in our thinking and actions. One is well advised, when traveling to a new territory, to take a good map and then to heck the map with the actual territory during the journey. This map must be subject to new objective scientific insights with due consideration of the potential imminence of the global changes. Our actions should reflect this viewpoint."

~ G.J. Wasserburg

LETTERS – Comment of “AGU Statement: Investigation of Scientists and Officials L’Aquila, Italy, Is Unfounded.”

"There is only one nature – the division into science and engineering is a human imposition, not a natural one. Indeed, the division is a human failure; it reflects our limited capacity to comprehend the whole,"

~ Bill Wulf


Too generous to a fault? Is reliable earthquake safety a lost art? Errors in expected human losses due to incorrect seismic hazard estimates

Too generous to a fault? Is reliable earthquake safety a lost art? Errors in expected human losses due to incorrect seismic hazard estimates
Errors in Seismic Hazard Assessment are Creating Huge Human Losses

"One is well advised, when traveling to a new territory, to take a good map and then to check the map with the actual territory during the journey." In just such a reality check, Global Seismic Hazard Assessment Program (GSHAP) maps (prepared using PSHA) portrayed a "low seismic hazard," which was then also assumed to be the "risk to which the populations were exposed." But time-after-time-after-time the actual earthquakes that occurred were not only "surprises" (many times larger than those implied on the maps), but they were often near the maximum potential size (Maximum Credible Earthquake or MCE) that geologically could occur. Given these "errors in expected human losses due to incorrect seismic hazard estimates" revealed globally in these past performances of the GSHAP maps (> 700,000 deaths 2001–2011), we need to ask not only: "Is reliable earthquake safety a lost art?" but also: "Who and what were the 'Raiders of the Lost Art?'"

The Problem with Probabilistic Methods


"Probabilistic methods of estimating earthquake hazard (the one by Cornell [1968] and its reappraisals) are not based on physically sound models and have some fundamental flaws [Castaños and Lomnitz, 2002; Cyranoski, 2011]. In particular, the dimensionless probability of exceedance (the probability that a given level of ground shaking will be exceeded in a given period of time) is erroneously equated to the dimensional rate of occurrence (the number of events per given period of time [Wang, 2011]), making problematic even the math of probabilistic seismic hazard analysis (PSHA)."

Seismic Hazard Assessment: Issues and Alternatives
https://www.researchgate.net/publication/225727148_Seismic_Hazard_Assessment_Issues_and_Alternatives

Two approaches, probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA), are commonly used for seismic hazard assessment. Although PSHA has been proclaimed as the best approach for seismic hazard assessment, it is scientifically flawed (i.e., the physics and mathematics that PSHA is based on are not valid). Use of PSHA could lead to either unsafe or overly conservative engineering design or public policy, each of which has dire consequences to society. On the other hand, DSHA is a viable approach for seismic hazard assessment even though it has been labeled as unreliable.

TAKE ME HOME . . . SEISMIC LOADS!
I haven't seen anything regarding Site Class, for Mineral or Louisa, VA, as well as the estimated epicentral region of Central Virginia's Piedmont? Cuckoo seems to be the closest built environment to the epicenter (with still an uncertainty: horizontal +/- 2.3 km (1.4 miles); depth +/- 3.1 km (1.9 miles)). No one has officially designated this as the CUCKOO Earthquake. But read below and see if, perhaps, that term might be better reserved for USGS seismic hazard mapping and U.S. Building Code requirements in both the Central Virginia Seismic Zone and in other known and active seismic zones throughout the Central and Eastern U.S. (CEUS)?

Also, the MMI intensity of earthquake ground shaking (VII - VIII at the estimated epicentral location) was more correctly indicative of SDC D.

Since 2000, the USGS Seismic Hazard Maps have continued to lower the hazard [SDS = SDS design earthquake spectral response accelerations:

SS = 0.31g (1997) (2000); 0.26g (2003); 0.22g (2009)
SCB: SDS = 0.21g (1997) (2000); 0.17g (2003); 0.15g (2009).
SCC: SDS = 0.25g (1997) (2000); 0.20g (2003); 0.17g (2009).
SCD: SDS = 0.32g (1997) (2000); 0.27g (2003); 0.23g (2009).],

making building code earthquake provisions less safe regarding both public safety and economic well-being.

These numbers translate to about a 30% decline in design strength (from a low number to an even lower number) in just the last decade! ( for the SDS "Design Earthquake Spectral Response Acceleration Parameter"). A 33% increase in design strength used to be the difference between Seismic Zone 3 and Seismic Zone 4 requirements!

For Site Class B, this now makes the epicentral region of this M 5.8 Virginia (Cuckoo) earthquake Seismic Design Category A (SDC A) - the same as Florida and Michigan (which have no active seismic zones or geologic evidence of mountain building). [ http://www.eqclearinghouse.org/2011-08-23-virginia/category/general-information/ ] , formerly [ http://www.dmme.virginia.gov/DMR3/Va_5_8_earthquake.shtml ]

This "minor" earthquake now seems to be amongst the most widely felt earthquakes in U.S. history. (i.e., "ever!") -- "Felt strongly in much of central Virginia and southern Maryland. Felt throughout the eastern US from central Georgia to central Maine and west to Detroit, Michigan and Chicago, Illinois. Felt in many parts of southeastern Canada from Montreal to Windsor." Source USGS

Clearly we are no longer in Florida, Michigan . . . or even in Kansas any more!

Too many (a) unsafe conditions and (b) brittle-failure-mode susceptible building products are allowed in the low SDC's A, B, and C - and it defies both logic, engineering judgment, common sense, as well as the professional responsibility of our combined professions. I doubt if any of the brick veneer that separated during this M 5.8 Virginia earthquake would have even been required to be adequately attached for earthquake (lateral force) resistance in these SDC's of A,B and C?

Remember: "The buck stops shear!"

West Virginia, Mountain Mama . . . Take Me Home . . . Seismic Loads!" . . . because http://www.youtube.com/watch?v=oN86d0CdHgQ

"We have nothing to fear but veneer itself!"
WHAT IS THE CHANCE OF AN EARTHQUAKE?

Making sense of earthquake forecasts is difficult, in part because standard interpretations of probability are inadequate. A model-based interpretation is better, but lacks empirical justification. Furthermore, probability models are only part of the forecasting machinery. For example, the USGS San Francisco Bay Area forecast for 2000–2030 involves geological mapping, geodetic mapping, viscoelastic loading calculations, paleoseismic observations, extrapolating rules of thumb across geography and magnitude, simulation, and many appeals to expert opinion. Philosophical difficulties aside, the numerical probability values seem rather arbitrary. Another large earthquake in the San Francisco Bay Area is inevitable, and imminent in geologic time. **Probabilities are a distraction.** Instead of making forecasts, the **USGS could help to improve building codes and to plan the government’s response to the next large earthquake.** Bay Area residents should take reasonable precautions, including bracing and bolting their homes as well as securing water heaters, bookcases, and other heavy objects. They should keep first aid supplies, water, and food on hand. They should largely ignore the USGS probability forecast.
Maps to the Future


Of particular note was the creation of a number of new deterministic zones associated with faults having low activity rate. After initial rejection of the maps, the [9] PUC suggested revision of the deterministic zone definitions. [10] The USGS revised the maps, and the [11] PUC adopted the revised maps. However, this adoption was not by unanimous vote and [12] several PUC members expressed dissatisfaction with the process for developing the maps and the lack of opportunity for the structural engineering community to provide input to map development. This dissatisfaction carried over into the [13] ASCE-7 committee, which as of the time of preparation of this report, had rejected the new maps yo-yo-results for inclusion in [14] ASCE 7-16. [15] FEMA conceived of the concept for [16] Project 17 to address these concerns and authorized the planning effort which resulted in this [17] report. This pin-ball machine analogy to earthquake hazard mapping is obviously as unsatisfactory as it is defective - in addressing public safety concerns from real earthquakes!

Called Project 17 - Developing Next Generation Seismic Design Values Maps (http://www.nehrp.gov/pdf/Project17PlanningReport.pdf), a small group effort has been assembled to characterize (a) what the problems are; and (b), importantly, what to do about them! However, since the 17 Project 17 Planning Committee Participants (+ 1 FEMA Project Officer), p. 7, are comprised of the very same “experts” who have created the problem, it is very unlikely to me that they will overcome Einstein’s formidable observation that: “The significant problems we face cannot be solved at the same level of thinking with which we created them.” Dissappointingly, these Project and Committee Chairs and Members (7, or almost half of which are USGS), in their “Preliminary Planning Report,” fail to mention that probabilistic seismic hazard analysis, or psha, might be a significant part of the problem!

PSHA, a bastardization of Cornell's 1968 "Engineering Seismic Risk Analysis;" while "a mixture of mythology and clever thinking," is more ideology than it is either reliable and therefore useful - and, like cosmologists, Committee Participants are “often in error but never in doubt!” - despite an overwhelming preponderance of evidence (from real world earthquake occurrences) against them.

http://www.ce.memphis.edu/7137/PDFs/Cornell/1583.pdf

So this code change proposal is duly needed to restore the code format to a Step 1 that identifies the starting point for earthquake resistant design and construction: lateral design strength, or base shear, along with ductile detailing requirements to enhance toughness “in the inelastic range of response.” Simply stated: “If you don’t know where you're going, when you get there you’ll be lost!” However, remember that the important major changes that have occurred in seismic design procedures in building codes have all occurred following observations of unsatisfactory structural damage (poor performance) in recent earthquakes. Wallace, J. (2004). CE 243 Seismic Code Requirements, UCLA, Fall 2004, 34 p. (http://www.seas.ucla.edu/~wallace/Files%20-%20Teaching%20Page/CE%20243A/Seismic%20Code%20Handout%20F04.pdf)

With a now stable platform for thinking about earthquake resistant design; over one’s long practice career, engineering judgment will again mean something. With a yo-yo-ing lateral design strength or base shear, as is the present situation, engineers may acquire experience, but not engineering judgment. Re, This comment: “I'm designing buildings now for one-half the strength they had one year ago!” Worse yet, they probably will turn their brains off and go-to-sleep ... a common yo-yo phenomenon! After the 2010 M 8.8 Maule, Chile earthquake, a Chilean engineer, in accounting for the unexpected damage, commented: “We believed our computer programs!” String Theory is great, String Theory sounds “modern” ... but not when it is attached to yo-yos!

AS Mark Twain observed: “Good judgment comes from experience.” “And where does experience come from?,” he was asked! “FROM BAD JUDGMENT!” said Twain

Now that the American Society of Civil Engineers, or ASCE, has usurped all of the public’s financial investments (since 1977) in developing the NEHRP Provisions under the National Earthquake Hazards Reduction Program under their umbrella of ASCE-7, which rewards them now monetarily, since they can now “sell” this document (which, as a designated code standard, becomes de facto ... a required best seller); the code is, alas, no longer under more democratic protocols and therefore, at least potentially, openly inviting and more available to greater public involvement and participation. There was no public
involvement in the [16 steps] listed above on the way to the Project 17 report, which did not even tabulate the "map issues" . . . as has been made known previously through the "USGS Map Users Workshops," "ICC Code Change Process," and "ASCE 41-13 Public Comment," and most recently, the "State of Kentucky Building Code Adoption."

**History of Modern Earthquake Hazard Mapping and Assessment in California Using a Deterministic or Scenario Approach**

History of Modern Earthquake Hazard Mapping and Assessment in California Using a Deterministic or Scenario Approach - [http://www.academia.edu/7849458/History_of_Modern_Earthquake_Hazard_Mapping_and_Assessment_in_California_Using_a_Deterministic_or_Scenario_Approach](http://www.academia.edu/7849458/History_of_Modern_Earthquake_Hazard_Mapping_and_Assessment_in_California_Using_a_Deterministic_or_Scenario_Approach)

http://www.academia.edu/7849458/History_of_Modern_Earthquake_Hazard_Mapping_and_Assessment_in_California_Using_a_Deterministic_or_Scenario_Approach - presentation

History of Modern Earthquake Hazard Mapping and Assessment in California Using Deterministic or Scenario Approach - [http://indico.ictp.it/event/a09145/session/51/contribution/37/material/0/0.pdf](http://indico.ictp.it/event/a09145/session/51/contribution/37/material/0/0.pdf)
Kentucky Geological Survey Challenges the USGS Seismic Hazard Map
https://www.youtube.com/watch?v=zlHM9tIUF8g

Seismic Hazard Assessment for Kentucky
Seismic Hazard Assessment for Kentucky - http://www.uky.edu/KGS/geologichazards/research_assessment.htm
(http://www.uky.edu/KGS/geologichazards/research_assessment.htm)
http://www.uky.edu/KGS/geologichazards/research_assessment.htm

“The future’s whatever you make it to be . . . make it a good one!”

- Doc Emmet Brown

Back to the Future III

Or . . . more of this?

"Well, it's hard for me to see
How you got such a hold on me
First I'm up and then I'm down
Then my heart goes around and around."
Lateral Design Strength (Base Shear) Coefficient expressed as Seismic Zones 0 - 4

Relationship between lateral design strength coefficient or base shear - and effective peak ground acceleration, 10% exceedance probability for 50 years exposure time
Lateral Design Strength (Base Shear) Coefficient expressed as Seismic Zones 0-4_Color

Figure A7  1994 Uniform Building Code zone map. Zones are identified by the numbers from 0 to 4. Seismic zone factors are assigned to each zone: Zone 0 = 0, Zone 1 = 0.075, Zone 2A = 0.15, Zone 2B = 0.25, Zone 3 = 0.35, Zone 4 = 0.50. Each zone also has specific structural detailing requirements.

After IBC, 1994 (This map was redrawn from the original source, if differences occur, the original source should be used).

Lateral Design Strength (Base Shear) Coefficient expressed as Seismic Zones 0-4_1994/1997 UBC_Color
Lateral Design Strength (Base Shear) Coefficient expressed as Seismic Zones 0-4

Bibliography: S110-12 (2012 IBC) - deleting MCE Ground Motion Maps - restoring previous MCE Ground Motion Maps
S110-12 (2012 IBC) - deleting MCE Ground Motion Maps - restoring previous MCE Ground Motion Maps -

Earthquake-Resistant Design According to 1997 UBC
**PSHA: is it science?**
Castanos, H., and C. Lomnitz (2002). PSHA: is it science? Engineering Geology 66, no. 3, pp. 315-317. [Abstract: Probabilistic seismic hazard analysis (PSHA) is beginning to be seen as unreliable. The problem with PSHA is that its data are inadequate and its logic is defective. Much more reliable, and more scientific, are deterministic procedures, especially when coupled with engineering judgment.] DOI: 10.1016/S0013-7952(02)00039-X
https://www.researchgate.net/publication/238378491_PSHA_is_it_science

Why earthquake hazard maps often fail and what to do about it
doi:10.1016/j.tecto.2012.06.047

George W. Housner, EERI Oral History 1997 (https://www.eeri.org/site/images/projects/oralhistory/housner.pdf) particularly . . . ch. 8 Development of Seismic Codes; ch. 9 Earthquake Engineering and Seismic Design; ch. 10 Seismologists and Earthquake Engineers

According to the probabilistic seismic hazard analysis (PSHA) approach, the deterministically evaluated or historically defined largest credible earthquakes (often referred to as Maximum Credible Earthquakes, MCEs) are “an unconvincing possibility” and are treated as “likely impossibilities” [since PSHA assumes “the risk quickly decreases as the ground motion intensity increases.”] within individual seismic zones. However, globally over the last decade such events keep occurring where PSHA predicted seismic hazard to be low.

**Why are the Standard Probabilistic Methods of Estimating Seismic Hazard and Risks Too Often Wrong?**
DOI: 10.1016/B978-0-12-394848-9.00012-2

https://www.researchgate.net/publication/257419530_Errors_in_expected_human_losses_due_to_incorrect_seismic_hazard_estimates


Angelou, Maya: (1928 - 2014). “All great achievements require time.”


Bela, J.L. (2012). International Code Council ICC S110-12 Public Comment AS – Figs. 1613.3.1 Deleting MCER Maps, 10 p. [S110-12 Public Comment AS Figs. 1613.3.1 Deleting MCE subR Maps.doc]


Brilliant, Ashleigh: (1933 - ). "My sources are unreliable, but their information is fascinating." "To be sure of hitting the target, shoot first, and call whatever you hit the target."

Castanos, H., and C. Lomnitz (2002). PSHA: is it science? Engineering Geology 66, no. 3, pp. 315-317. [Abstract: Probabilistic seismic hazard analysis (PSHA) is beginning to be seen as unreliable. The problem with PSHA is that its data are inadequate and its logic is defective. Much more reliable, and more scientific, are deterministic procedures, especially when coupled with engineering judgment.] DOI: 10.1016/S0013-7952(02)00039-X

De Vivo, B. (2011). Vesuvius: volcanic hazard and civil defense, Resignation Letter to AGU; April 21. 2011– sent to Eos FORUM eds. B. Richman and J. Geissman, re: lack of fairness and due process during very long 2 yr. review period with Eos, and then FORUM capitulation to Italian political dysfunction in public safety policy.


http://www.stat.berkeley.edu/~stark/Preprints/611.pdf


Heraclitus: (544 - 483 BC). "If you do not expect the unexpected you will not find it, for it is not to be reached by search or trail [or psha]."


"In view of the devastation produced by large earthquakes and associated phenomena exemplified by the 2004 Sumatra earthquake and tsunamis, the 2008 Wenchuan earthquake in China, the 2010 Haiti earthquake, and the 2011 Tohoku earthquake and tsunamis in Japan [see Table 1], it is imperative that structures should be designed and constructed to withstand the largest or Maximum Credible Earthquake (MCE) events that include or exceed such historic events; and the public should be advised to be prepared and ready for such possible events beforehand. These are the most dangerous and destructive events that can happen at any time regardless of their low frequencies or long recurrence intervals. Therefore,
earthquake hazard assessment to determine seismic design loads should consider the MCE events. Emergency management policy should consider scenarios for possible MCE events."


https://eos.org/opinions/reality-check-seismic-hazard-models-you-can-trust


Mualchin, L. (2011). History of Modern Earthquake Hazard Mapping and Assess-ment in California Using a Deterministic or


Wilder, Billy: (1906 - 2002). "If you're going to tell people the truth, be funny or they will kill you."


"We have nothing to fear but shear itself." "We're all subducting in this together." "Do not look back in anger, or forward in fear, but around in awareness"

Cost Impact: Will increase the cost of construction
These are changes in terminology, for the purpose of clarifying both the intent of the code and the practice of earthquake engineering. Cost increase or decrease will be realized when the cited “lateral design strength parameters, or base shear coefficients,” are actually used, as determined from Figures 16.13.1(1) through 1613.1(8). This more scientific approach reflects a much more straightforward and transparent of “seismic zonation,” which is based upon the magnitude size of potential deterministic or scenario earthquakes.
This proposal may or may not affect the cost of construction, but only as a small portion of the less than 20% of total building cost that comprises the structural portion of a building. This is (1) because commercial buildings, as well as detached one-
and two-family dwellings, must be already built to withstand the lateral forces due to wind; and (2) must include basements, "safe rooms", or other afforded protections to protect occupants against the deadly impacts of hurricanes and tornadoes.

The point is: Both Commercial buildings as well as detached one- and two-family need to consider the maximum Magnitude of realistic scenario earthquakes that they could, in fact, experience. And they should not be constructed vulnerable to earthquakes, because a flawed numerical hazard model "guesses" incorrectly as to the likelihood or possibility of earthquakes. This should remain a rational and a scientific decision based upon protecting both public safety and property. A second point is that "cost" due to structural elements is almost always less than 80% of the cost of a building!

"In general, better seismic performance is achieved through increased lateral design forces (i.e., base shear), and detailing requirements that improve structural connection strength or structural member behavior in the inelastic range of response. Requirements for seismic bracing and anchorage of nonstructural components reduce potential for nonstructural damage and loss of building (or system) functionality."*

* viii, Executive Summary, NIST GCR 14-917-26
NEHRP Consultants Joint Venture A partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering.

In general, where costs might be increased, cost premiums above requirements for wind tend to fall within a range of +1-3%. For cases where seismic requirements would be now additional to what previous codes either applied/neglected/failed to enforce, estimates probably would fall within the range of 0.25 - 1%.

Analysis: See RB18-16 for IRC coordination proposal

Public Hearing Results

Committee Action: Disapproved
Committee Reason: The earthquake maps that are proposed would put the IBC earthquake provisions in opposition to the ASCE 7 referenced standard.

Assembly Action: None

Individual Consideration Agenda

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com) requests Approve as Submitted.

Commenter's Reason: "It is better to be right than to be consistent."

- Winston Churchill

"If your definition is wrong, you'll look for the wrong thing."

- Carol Cleland

"Anyone who conducts an argument by appealing to authority is not using his intelligence; he is just using his memory."

- Leonardo da Vinci

"Just because it comes from a consensus standard doesn't mean it isn't without problems."

- Jay Crandell

The Committee Reason for Disapproval is not an argument, but simply a statement of the Logical Fallacies (1) Authority: "You said that because an authority [ASCE 7] thinks something, it must therefore be true." The proposed deletion of the current RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS OF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B design values maps, under this Proposal, would be replaced by a new design values map: displaying LATERAL DESIGN STRENGTH (BASE SHEAR) COEFFICIENTS, 0 – 0.4 (as may be converted from "Figure A7. 1994 Uniform Building Code zone map" of...
S118-16). This more scientifically correct deterministic or scenario approach comes from the same level of understanding as that underlying ASCE 7, and it simply is applying the empirical evidence of our recent experiences with global earthquakes for a better, more logical and more consistent approach to earthquake engineering and seismic design; and

(2) Begging the Question [Circular Argument]: “You presented a circular argument in which the conclusion (Disapproval) was included in the premise [Nothing Must Be “in opposition to the ASCE 7 referenced standard.”] This logically incoherent argument often arises in situations where people have an assumption that is very ingrained, and therefore taken in their minds as a given. Circular reasoning is bad mostly because it is not good. Example: the word of Zorba [ASCE 7] the Great is flawless and perfect. We know this because it says so in [ASCE 7] The Great and Infallible Book of [ASCE 7’s] Best and Most Truest Things that are Definitely True and Should Not Ever Be Questioned.”

But how can you not question ASCE 7? - given its broad-speaking and really irresponsible Disclaimer (“disclaiming any and all liability”) below:

“This standard was developed by a consensus standards development process . . .

"While ASCE’s process is designed to promote standards that reflect a fair and reasoned consensus among all interested participants, while preserving the public health, safety, and welfare that is paramount to its mission, it has not made an independent assessment of and does not warrant the accuracy, completeness, suitability, or utility of any information, apparatus, product, or process discussed herein. ASCE [BUMMER!]
does not intend, nor should anyone interpret, ASCE’s standards to replace the sound judgment of a competent professional, having knowledge and experience in the appropriate field(s) of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the contents of this standard.

ASCE has no authority to enforce compliance with its standards and does not undertake to certify products for compliance or to render any professional services to any person or entity.

ASCE disclaims any and all liability for any personal injury, property damage, financial loss or other damages of any nature whatsoever, including without limitation any direct, indirect, special, exemplary, or consequential damages resulting from any person's use of, or reliance on, this standard. Any individual who relies on this standard assumes full responsibility for such use.

ASCE and American Society of Civil Engineers — Registered in U.S. Patent and Trademark Office
Furthermore, any potential conflicts between “this code” and a “referenced standard” are long-understood to be then resolved in favor of the code – which governs!

“When the code has specific requirements that vary from those found in a referenced standard, the requirements of the code take precedence over the standard.”

ICC REFERENCED STANDARDS GUIDE (2006), p. 3

(In fact, ASCE 7 does not really meet the intended definition of an actual Standard – and it is really more of a “Guideline” or an “Alternate Method,” than it is a true Standard. It’s drafting committees, “often in error, but never in doubt,” seem more committed to business-as-usual and “the need not to know” than they are to sound engineering principles (like correct mathematics) and also to logical and defensible thinking. “Do No Harm” and “public safety” are, evidently, non-persuasive!)

These proposed new design values maps will . . . .
“better clarify that the code: (a) is anchoring the lateral strength resistance (or base shear) requirement for design to a more scientific and systematically consistent scenario earthquake magnitude criteria; and (2) is, therefore, also not downgrading the lateral design strength or connection detailing requirements based upon a hazard model, which is largely a numerical creation heavily dependent on the perceived likelihoods of earthquake occurrences, as a controlling factor.”

The Problem with ASCE 7’s Earthquake Hazard Model

What is the Chance of an Earthquake?

Remember the quote by Upton Sinclair...

“It is difficult to get a man to understand something, when his salary depends upon his not understanding it!”
“All sciences are vain and full of errors that are not born of Experience, the mother of all Knowledge.”
- Leonardo da Vinci

This recently published article **Reality Check: Seismic Hazard Models You Can Trust** provides the comprehensive understanding (based on empirical evidence) of why these and other associated proposed changes are needed to provide a more reasonable and more practical approach to seismic safety for the general public that is exposed to major earthquake risk – namely because the “current probabilistic methods to quantify earthquake hazards have serious problems.”

**Improving Earthquake Hazard Assessments . . . is the realizable goal of this Code Change Proposal**

”. . . BUILDINGS AND OTHER CRITICAL STRUCTURES SHOULD BE DESIGNED CAPABLE TO RESIST FUTURE EARTHQUAKES. When an earthquake with a given magnitude M occurs, it causes a specific ground shaking that certainly does not take into account whether the event is rare or not; thus ground motion parameters for seismic design should not be scaled depending on earthquake recurrence. Accordingly, in a cost-effective prevention perspective, when considering two sites prone to earthquakes with the same magnitude M, given that all the remaining conditions are the same, the site where large earthquakes are more sporadic appears naturally preferable for new settlements. Nevertheless parameters in seismic design must be equal at the two sites, since the expected magnitude is the same. In fact, although the shaking is more likely at one site, an element that should be factored in insurance, we favor building to the same standard to promote public safety [and community resilience], since there are no evidences that what happened in the past cannot repeat in the near future, or that it will occur only after a very long time.”

**Bibliography: Let's Look at the Basic Problems:**

**Why Earthquake Hazard Maps often fail and what to do about it . . .**
http://www.earth.northwestern.edu/people/sets/Texts/mapfailure.pdf
(http://www.earth.northwestern.edu/people/sets/Texts/mapfailure.pdf)
"The 2011 Tohoku earthquake is another striking example – after the 2008 Wenchuan and 2010 Haiti earthquakes – of highly destructive earthquakes that occurred in areas predicted by earthquake hazard maps to be relatively safe. Here, we examine what went wrong for Tohoku, and how this failure illustrates limitations of earthquake hazard mapping. We use examples from several seismic regions to show that earthquake occurrence is typically more complicated than the models on which hazard maps are based, and that the available history of seismicity is almost always too short to reliably establish the spatiotemporal pattern of large earthquake occurrence. As a result, key aspects of hazard maps often depend on poorly constrained parameters, whose values are chosen based on the mapmakers' preconceptions. When these are incorrect, maps do poorly. This situation will improve at best slowly, owing to our limited understanding of earthquake processes. However, because hazard mapping has become widely accepted and used to make major decisions, we suggest two changes to improve current practices. First, the uncertainties in hazard map predictions should be assessed and clearly communicated to potential users. Recognizing the uncertainties would enable users to decide how much credence to place in the maps and make them more useful in formulating cost-effective hazard mitigation policies. Second, hazard maps should undergo rigorous and objective testing to compare their predictions to those of null hypotheses, including ones based on uniform regional seismicity or hazard. Such testing, which is common and useful in similar fields, will show how well maps actually work and hopefully help produce measurable improvements. There are, however, likely limits on how well hazard maps can ever be made because of the intrinsic variability of earthquake processes." (Stein et. al. 2012)

PSHA: Is it Science?

https://www.researchgate.net/publication/238378491_PSHA_is_it_science

"Probabilistic seismic hazard analysis (PSHA) is beginning to be seen as unreliable. The problem with PSHA is that its data are inadequate and its logic is defective. Much more reliable, and more scientific, are deterministic procedures, especially when coupled with engineering judgment.”

(Castaños and Lomnitz 2002) DOI: 10.1016/S0013-7952(02)00039-X

Why are the Standard Probabilistic Methods of Estimating Seismic Hazard and Risks Too Often Wrong?


According to the probabilistic seismic hazard analysis (PSHA) approach, the deterministically evaluated or historically defined largest credible earthquakes (often referred to as Maximum Credible Earthquakes, MCEs) are “an unconvincing possibility” and are treated as “likely impossibilities” [since PSHA assumes “the risk quickly decreases as the ground motion intensity increases.”] within individual seismic zones. However, globally over the last decade such events keep occurring where PSHA predicted seismic hazard to be low (Panza et. al. 2014)

According to the probabilistic seismic hazard analysis (PSHA) approach, the deterministically evaluated or historically defined largest credible earthquakes (often referred to as Maximum Credible Earthquakes, MCEs) are “an unconvincing possibility” and are treated as “likely impossibilities” within individual seismic zones. However, globally over the last decade such events keep occurring where PSHA predicted seismic hazard to be low. Systematic comparison of the observed ground shaking with the expected one reported by the Global Seismic Hazard Assessment Program (GSHAP) maps discloses gross underestimation worldwide. Several inconsistencies with available observation are found also for national scale PSHA maps (including Italy), developed using updated data sets. As a result, the expected numbers of fatalities in recent disastrous earthquakes have been underestimated by these maps by approximately two to three orders of magnitude. The total death toll in 2000–2011 (which exceeds 700,000 people, including tsunami victims) calls for a critical reappraisal of GSHAP results, as well as of the underlying methods.

In this chapter, we discuss the limits in the formulation and use of PSHA, addressing some theoretical and practical issues of seismic hazard assessment, which range from the overly simplified assumption that one could reduce the tensor problem of
seismic-wave generation and propagation into a scalar problem (as implied by ground motion prediction equations), to the insufficient size and quality of earthquake catalogs for a reliable probability modeling at the local scale. Specific case studies are discussed,

"The Map is not the Territory"

- Alfred Korzybski

“The ‘map’ is what we think resembles reality, and we should use it as a guide in our thinking and actions. One is well advised, when traveling to a new territory, to take a good map and then to check the map with the actual territory during the journey. This map must be subject to new objective scientific insights with due consideration of the potential imminence of the global changes. Our actions should reflect this viewpoint.”

- G.J. Wasserburg

"There is only one nature –

the division into science and engineering is a human imposition, not a natural one.

Indeed, the division is a human failure; it reflects our limited capacity to comprehend the whole,"

- Bill Wulf

Too generous to a fault? Is reliable earthquake safety a lost art? Errors in expected human losses due to incorrect seismic hazard estimates


Errors in Seismic Hazard Assessment are Creating Huge Human Losses


"One is well advised, when traveling to a new territory, to take a good map and then to check the map with the actual territory during the journey." In just such a reality check, Global Seismic Hazard Assessment Program (GSHAP) maps (prepared using PSHA) portrayed a “low seismic hazard,” which was then also assumed to be the "risk to which the populations were exposed." But time-after-time-after-time the actual earthquakes that occurred were not only “surprises” (many times larger than those implied on the maps), but they were often near the maximum potential size (Maximum Credible Earthquake or MCE) that geologically could occur. Given these “errors in expected human losses due to incorrect seismic hazard estimates” revealed globally in these past performances of the GSHAP maps (> 700,000 deaths 2001–2011), we need to ask not only: "Is reliable earthquake safety a lost art?" but also: "Who and what were the 'Raiders of the Lost Art?' "

The Problem with Probabilistic Methods

Reality Check: Seismic Hazard Models You Can Trust

https://eos.org/opinions/reality-check-seismic-hazard-models-you-can-trust

(https://eos.org/opinions/reality-check-seismic-hazard-models-you-can-trust)"Probabilistic methods of estimating earthquake hazard (the one by Cornell [1968] and its reappraisals) are not based on physically sound models and have some fundamental flaws [Castaños and Lomnitz, 2002; Cyranoski, 2011]. In particular, the dimensionless probability of exceedance (the probability that a given level of ground shaking will be exceeded in a given period of time) is erroneously equated to the dimensional rate of occurrence (the number of events per given period of time [Wang, 2011]), making problematic even the math of probabilistic seismic hazard analysis (PSHA)."

Seismic Hazard Assessment: Issues and Alternatives

https://www.researchgate.net/publication/225727148_Seismic_Hazard_Assessment_Issues_and_Alternatives

(https://www.researchgate.net/publication/225727148_Seismic_Hazard_Assessment_Issues_and_Alternatives)

Two approaches, probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA), are commonly used for seismic hazard assessment. Although PSHA has been proclaimed as the best approach for seismic hazard assessment, it is scientifically flawed (i.e., the physics and mathematics that PSHA is based on are not valid). Use of PSHA could lead to either unsafe or overly conservative engineering design or public policy, each of which has dire consequences to society. On the other hand, DSHA is a viable approach for seismic hazard assessment even though it has been labeled as unreliable.

WHAT IS THE CHANCE OF AN EARTHQUAKE?
Making sense of earthquake forecasts is difficult, in part because standard interpretations of probability are inadequate. A model-based interpretation is better, but lacks empirical justification. Furthermore, probability models are only part of the forecasting machinery. For example, the USGS San Francisco Bay Area forecast for 2000–2030 involves geological mapping, geodetic mapping, viscoelastic loading calculations, paleoseismic observations, extrapolating rules of thumb across geography and magnitude, simulation, and many appeals to expert opinion. Philosophical difficulties aside, the numerical probability values seem rather arbitrary. Another large earthquake in the San Francisco Bay Area is inevitable, and imminent in geologic time. 

**Probabilities are a distraction. Instead of making forecasts, the USGS could help to improve building codes and to plan the government's response to the next large earthquake.** Bay Area residents should take reasonable precautions, including bracing and bolting their homes as well as securing water heaters, bookcases, and other heavy objects. They should keep first aid supplies, water, and food on hand. They should largely ignore the USGS probability forecast.
Proposed Change as Submitted

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com)

2015 International Building Code

Delete and substitute as follows:

1613.3.3 Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. The maximum considered earthquake spectral response acceleration for short periods, \( S_{MS} \), and at 1-second period, \( S_{M1} \), adjusted for site class effects shall be determined by Equations 16-37 and 16-38, respectively:

\[
S_{MS} = F_a S_a \quad \text{(Equation 16-37)}
\]
\[
S_{M1} = F_v S_v \quad \text{(Equation 16-38)}
\]

where:

\( F_a \) = Site coefficient defined in Table 1613.3.3(1).
\( F_v \) = Site coefficient defined in Table 1613.3.3(2).
\( S_a \) = The mapped spectral accelerations for short periods as determined in Section 1613.3.1.
\( S_v \) = The mapped spectral accelerations for a 1-second period as determined in Section 1613.3.1.

- Each site shall be assigned a soil profile in accordance with UBC Table 16-J. Six soil profile types, which are dependent on the mapped lateral design strength coefficients [formerly seismic zone factors] (SA to SF) are based on previous earthquake records. Seismic Coefficient, \( C_a \), shall be in accordance with UBC Table 16-Q. Seismic Coefficient, \( C_v \), shall be in accordance with UBC Table 16-R.

- Reference standards type: This reference standard is new to the ICC Code Books

Add new standard(s) as follows:

UBC-97  1997 Uniform Building Code International Conference of Building Officials

Reason: 1613.3.1 Mapped acceleration parameters are deleted in Proposal 1613.3.1 and replaced by: 1613.3.1 Mapped lateral design strength parameters.

The language deleted here:

“The maximum considered earthquake spectral response acceleration for short periods, \( S_{MS} \), and at 1-second period, \( S_{M1} \), adjusted for site class effects shall be determined by Equations 16-37 and 16-38, respectively.”

is a fictitious numerical creation, not a real earthquake, and the implication that public safety and community resilience can be addressed by accounting, rather than engineering experience and judgement is not only unwarranted, but also dangerous! Moreover, the precision implied by applying the SITE COEFFICIENTS \( F_a \) and \( F_v \) (since, in most cases, the factors are close to unity anyway) is rapidly lost in the shuffle in comparison to other considerations, such as the Response Modification, or R Factors.

1988 Uniform Building Code

1990 SEAOC BLUE BOOK

1997 Uniform Building Code

Robert E. Bachman and David R. Bonneville (2000)
Bibliography:
See also BIBLIOGRAPHY in Proposal: Figure 1613.3.1 RISK-TARGETED MCER
Cost Breakdown of Nonstructural Building Elements
Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake.
http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032
Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Permalink: http://dx.doi.org/10.1061/(ASCE)CF.1943-5509.0000630
Read More: http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630
Homeowner's Guide to Earthquake Safety
Retrofitting Questions and Answers
Earthquake Safety, Inc., 2015 (web based)
http://www.earthquakesafety.com/earthquake-retrofitting-faq.html
Cost and Seismic Design
https://www.google.com/#q=cost+and+seismic+design+christopher+arnold+pdf
Earthquake Architecture website
http://www.iitk.ac.in/nicee/wcee/article/14_05-06-0185.PDF
Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee

COST IMPACT

Cost Impact: Will increase the cost of construction
Will, in some cases, not increase the cost of construction
Will, in some cases, increase the cost of construction
Since the mapped maximum considered earthquake spectral response accelerations can fluctuate up-or-down between map editions, this could effect cost accordingly, but I estimate that this would be minor.
This proposal may or may not affect the cost of construction. This is (1) because detached one- and two-family dwellings must be already built to withstand the lateral forces due to wind; and (2) must include basements, ("safe rooms"), or other afforded protections to protect occupants against the deadly impacts of hurricanes and tornadoes.
The point is: Detached one- and two-family need to consider the maximum Magnitude of realistic scenario earthquakes that they could, in fact, experience.
And not be constructed vulnerable to earthquakes, because a flawed numerical hazard model "guesses" incorrectly as to the likelihood or possibility of earthquakes. This should remain a rational and a scientific decision based upon protecting both public safety and property. A second point is that "cost" due to structural elements is almost always less than 80% of the cost of a building!
"In general, better seismic performance is achieved through increased lateral design forces (i.e., base shear), and detailing requirements that improve structural connection strength or structural member behavior in the inelastic range of response. Requirements for seismic bracing and anchorage of nonstructural components reduce potential for nonstructural damage and loss of building (or system) functionality."
* viii, Executive Summary, NIST GCR 14-917-26
NEHRP Consultants Joint Venture A partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering.

In general, where costs might be increased, cost premiums above requirements for wind tend to fall within a range of +1-3%. For cases where seismic requirements would be now additional to what previous codes either applied/neglected/failed to enforce, estimates probably would fall within the range of 0.25 - 1%.

{{{1143}}}

**Analysis:** A review of the standard(s) proposed for inclusion in the code, 1997 UBC, with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 1, 2016.

<table>
<thead>
<tr>
<th>Committee Action:</th>
<th>Disapproved</th>
</tr>
</thead>
<tbody>
<tr>
<td>Committee Reason:</td>
<td>This proposal would introduce undefined terms into the IBC. Furthermore, utilizing the UBC would put the IBC in conflict with the referenced standard, ASCE 7.</td>
</tr>
<tr>
<td>Assembly Action:</td>
<td>None</td>
</tr>
</tbody>
</table>

**Public Hearing Results**

Individual Consideration Agenda

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com) requests Approve as Submitted.

Commenter's Reason: "*It is better to be right than to be consistent.*"

- Winston Churchill

"*If your definition is wrong, you'll look for the wrong thing.*"

- Carol Cleland

"*Anyone who conducts an argument by appealing to authority is not using his intelligence; he is just using his memory.*"

- Leonardo da Vinci

"*Just because it comes from a consensus standard doesn't mean it isn't without problems.*"

- Jay Crandell

The Committee Reason for Disapproval – that "utilizing the Uniform Building Code (UBC) would put the IBC in conflict with the referenced standard, ASCE 7," is *not* correct; since Standards are, by definition, "not intended to be used as primary law but as a referenced authoritative resource." That is, having one code, in effect, adopt *another* code; and thereby then *prohibit* consideration of *any* change to *what should have been secondary resource material* -- is really neither wise nor acceptable!

Since maximum considered earthquake spectral response accelerations are being *deleted* and *replaced* with *new design values* maps, "Site coefficients" must be alternatively described and introduced here. Standards are, by common definition, published technical documents relating to design, materials, and methods of installation. Therefore, since the UBC is: (a) technically accurate; (b) published; (c) developed according to consensus process; and (d) sets forth "detailed procedures for design" – it is referenced here as the recommended resource for these new Site Coefficients.
Furthermore, any potential conflicts between “this code” and a “referenced standard” are long-understood to be then resolved in favor of the code – which governs!

"When the code has specific requirements that vary from those found in a referenced standard, the requirements of the code take precedence over the standard."

**ICC REFERENCED STANDARDS GUIDE** (2006), p. 3

Besides, how can you not question ASCE 7, noting its broad-speaking and really irresponsible Disclaimer ("disclaiming any and all liability") below:

"This standard was developed by a consensus standards development process . . .

"While ASCE's process is designed to promote standards that reflect a fair and reasoned consensus among all interested participants, while preserving the public health, safety, and welfare that is paramount to its mission, it has not made an independent assessment of and does not warrant the accuracy, completeness, suitability, or utility of any information, apparatus, product, or process discussed herein. ASCE [BUMMER!]
does not intend, nor should anyone interpret, ASCE's standards to replace the sound judgment of a competent professional, having knowledge and experience in the appropriate field(s) of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the contents of this standard.

ASCE has no authority to enforce compliance with its standards and does not undertake to certify products for compliance or to render any professional services to any person or entity.

ASCE disclaims any and all liability for any personal injury, property damage, financial loss or other damages of any nature whatsoever, including without limitation any direct, indirect, special, exemplary, or consequential damages resulting from any person's use of, or reliance on, this standard. Any individual who relies on this standard assumes full responsibility for such use.

ASCE and American Society of Civil Engineers --- Registered in U.S. Patent and Trademark Office

( In fact, ASCE 7 does not really meet the intended definition of an actual Standard – and it is really more of a “Guideline” or an "Alternate Method," than it is a true Standard. It's drafting committees, "often in error, but never in doubt," seem more committed to business-as-usual and "the need not to know" than they are to sound engineering principles (like correct mathematics) and also to logical and defensible thinking. "Do No Harm" and "public safety" are, evidently, non-persuasive!

These proposed new Site coefficients will work in concert with the new design values maps . . . "to better clarify that (a) the code is anchoring the lateral strength resistance (or base shear) requirement for design to a more scientific and systematically consistent scenario earthquake magnitude criteria; and (2) is therefore, also not downgrading the lateral design strength or connection detailing requirements based upon a hazard model, which is largely a numerical creation heavily dependent on the perceived likelihoods of earthquake occurrences, as a controlling factor."

The Problem with ASCE 7's Earthquake Hazard Model
What is the Chance of an Earthquake?

Remember the quote by Upton Sinclair "It is difficult to get a man to understand something, when his salary depends upon his not understanding it!"

"All sciences are vain and full of errors that are not born of Experience, the mother of all Knowledge."
- Leonardo da Vinci

This recently published article Reality Check: Seismic Hazard Models You Can Trust provides the comprehensive understanding (based on empirical evidence) of why these and other associated proposed changes are needed to provide a more reasonable and more practical approach to seismic safety for the general public that is exposed to major earthquake risk – namely because the "current probabilistic methods to quantify earthquake hazards have serious problems." https://eos.org/opinions/reality-check-seismic-hazard-models-you-can-trust

Improving Earthquake Hazard Assessments . . . is the realizable goal of this Code Change Proposal

" . . . BUILDINGS AND OTHER CRITICAL STRUCTURES SHOULD BE DESIGNED CAPABLE TO RESIST FUTURE EARTHQUAKES. When an earthquake with a given magnitude M occurs, it causes a specific ground shaking that certainly does not take into account whether the event is rare or not; thus ground motion parameters for seismic design should not be scaled depending on earthquake recurrence. Accordingly, in a cost-effective prevention perspective, when considering two sites prone to earthquakes with the same magnitude M, given that all the remaining conditions are the same, the site where large earthquakes are more sporadic appears naturally preferable for new settlements. Nevertheless parameters in seismic design must be equal at the two sites, since the expected magnitude is the same. In fact, although the shaking is more likely at
one site, an element that should be factored in insurance, we favor building to the same standard to promote public safety [and community resilience], since there are no evidences that what happened in the past cannot repeat in the near future, or that it will occur only after a very long time."


**Bibliography:** See also BIBLIOGRAPHY in Public Comment S118-16.
Proposed Change as Submitted

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com)

2015 International Building Code

Delete without substitution:

1613.3.4 Design spectral response acceleration parameters. Five percent damped design spectral response acceleration at short periods, $S_{DS}$, and at 1-second period, $S_{D1}$, shall be determined from Equations 16-39 and 16-40, respectively:

$S_{MS} = \ldots$

$S_{M1} = \ldots$

where:

$S_{MS}$ = The maximum considered earthquake spectral response accelerations for short period as determined in Section 1613.3.3.

$S_{M1}$ = The maximum considered earthquake spectral response accelerations for 1-second period as determined in Section 1613.3.3.

Reason: 1613.3.1 Mapped acceleration parameters are deleted in Proposal 1613.3.1 and replaced by: 1613.3.1 Mapped lateral design strength parameters.

The language deleted here:

"Five percent damped design spectral response acceleration at short periods, $S_{DS}$, and at 1-second period, $S_{D1}$, shall be determined from equations 16-39 abd 16-40, respectively:

is a fictitious numerical creation, not a real earthquake, and the implication that public safety and community resilience can be addressed by, rather than engineering experience and judgement, is not only unwarranted, but also dangerous! Moreover, the precision implied by applying the SITE COEFFICIENTS $F_a$ and $F_v$ (since, in most cases, the factors are close to unity anyway) is rapidly lost in the shuffle in comparison to other considerations, such as the Response Modification, or R Factors.

Furthermore, taking 2/3rds of a number that is not stable between successive code editions is not only problematical, but also very unwise with regard to both public safety and community resilience!

1988 Uniform Building Code

1990 SEAOC BLUE BOOK

1997 Uniform Building Code

Robert E. Bachman and David R. Bonneville (2000)

Bibliography: See also BIBLIOGRAPHY in Proposal: Figure 1613.3.1 RISK-TARGETED MCER
Cost Breakdown of Nonstructural Building Elements
Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake.
doi:http://dx.doi.org/10.1193/1.4000032
http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032
Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Permalink: http://dx.doi.org/10.1061/(ASCE)CF.1943-5509.0000630
Read More: http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630

Homeowner's Guide to Earthquake Safety

Retrofitting Questions and Answers
Earthquake Safety, Inc., 2015 (web based)
http://www.earthquakesafety.com/earthquake-retrofitting-faq.html

Cost and Seismic Design
https://www.google.com/#q=cost+and+seismic+design+christopher+arnold+pdf

Earthquake Architecture website
http://www.iitk.ac.in/nicee/wcee/article/14_05-06-0185.PDF

Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee

**Cost Impact:** Will increase the cost of construction

Will increase the cost of construction?
Will, in some cases, not increase the cost of construction
Will, in some cases, increase the cost of construction

Since the mapped maximum considered earthquake spectral response accelerations can fluctuate up-or-down between map editions, this could effect cost accordingly, but I estimate that this would be minor.

This proposal may or may not affect the cost of construction. This is (1) because detached one- and two-family dwellings must be already built to withstand the lateral forces due to wind; and (2) must include basements, "safe rooms"), or other afforded protections to protect occupants against the deadly impacts of hurricanes and tornadoes.

The point is; Detached one- and two-family need to consider the maximum Magnitude of realistic scenario earthquakes that they could, in fact, experience.

And not be constructed vulnerable to earthquakes, because a flawed numerical hazard model "guesses" incorrectly as to the likelihood or possibility of earthquakes. This should remain a rational and a scientific decision based upon protecting both public safety and property. A second point is that "cost" due to structural elements is almost always less than 80% of the cost of a building!

"In general, better seismic performance is achieved through increased lateral design forces (i.e., base shear), and detailing requirements that improve structural connection strength or structural member behavior in the inelastic range of response.
Requirements for seismic bracing and anchorage of nonstructural components reduce potential for nonstructural damage and loss of building (or system) functionality."*

* viii, Executive Summary, NIST GCR 14-917-26


In general, where costs might be increased, cost premiums above requirements for wind tend to fall within a range of +1-3%.
For cases where seismic requirements would be now additional to what previous codes either applied/neglected/failed to enforce, estimates probably would fall within the range of 0.25 - 1%.

{{1143}}}
Committee Action: Disapproved

Committee Reason: This code change would break the IBC earthquake provisions by removing a key provision and in addition would create a conflict with the referenced standard, ASCE 7.

Assembly Action: None

Individual Consideration Agenda

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com) requests Approve as Submitted.

Commenter's Reason: “It is better to be right than to be consistent.”

- Winston Churchill

“If your definition is wrong, you'll look for the wrong thing.”

- Carol Cleland

“Anyone who conducts an argument by appealing to authority is not using his intelligence; he is just using his memory.”

- Leonardo da Vinci

“Just because it comes from a consensus standard doesn't mean it isn't without problems.”

- Jay Crandell

“I break for earthquakes, quake it to me gently!”

The Committee Reason for Disapproval is based on these Logical Fallacies:

- Appeal to Authority;
- Begging the Question (Circular Reasoning);
- and (my favorite) STRAW MAN.
What is a Straw Man fallacy?

The Straw Man fallacy is committed when a person simply ignores a person's actual position and substitutes a distorted, exaggerated or misrepresented version of that position. This sort of "reasoning" has the following pattern:

- Person A has position X.
- Person B presents position Y (which is a distorted version of X).
- Person B attacks position Y.
- Therefore X is false/incorrect/flawed.

The deletion here is necessary to achieve consistency with the package of associated code changes. Since the design values map is to be changed, the language/terminology of this section no longer applies:

"Five-percent damped design spectral response acceleration at short periods, $S_D$, and at 1-second period, $S_D^1$, determined from equations 16-39 and 16-40," respectively, are fictitious numerical creations, not a real earthquake tensor phenomena. Further, the implication that both public safety and community resilience can be addressed by present design methods, rather than more systematically applying both engineering experience and good judgement, is not only unwarranted, but also dangerous!

Moreover, the precision implied by applying the SITE COEFFICIENTS Fa and Fv (since, in most cases, the factors are close to unity anyway) is rapidly lost in the shuffle (and doesn't improve the overall accuracy) - in comparison to other considerations, such as the Response Modification, or R Factors.

Finally, taking 2/3rds of a number that is not stable between successive code editions is not only problematical, but also very unwise with regard to both public safety and community resilience!

See further background and discussion in Public Comment S118-16

Bibliography: See BIBLIOGRAPHY under Public Comment S118-16
Proposed Change as Submitted

Proponent : James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com)

2015 International Building Code

Delete without substitution:

1613.3.5 Determination of seismic design category. Structures classified as Risk Category I, II or III that are located where the mapped spectral response acceleration parameter at 1-second period, $S_1$, is greater than or equal to 0.75 shall be assigned to Seismic Design Category E. Structures classified as Risk Category IV that are located where the mapped spectral response acceleration parameter at 1-second period, $S_1$, is greater than or equal to 0.75 shall be assigned to Seismic Design Category F. All other structures shall be assigned to a seismic design category based on their risk category and the design spectral response acceleration parameters, $S_0$ and $S_1$, determined in accordance with Section 1613.3.4 or the site-specific procedures of ASCE 7. Each building and structure shall be assigned to the more severe seismic design category in accordance with Table 1613.3.5(1) or 1613.3.5(2), irrespective of the fundamental period of vibration of the structure, $T$.

1613.3.5.1 Alternative seismic design category determination. Where $S_1$ is less than 0.75, the seismic design category is permitted to be determined from Table 1613.3.5(1) alone when all of the following apply:
1. In each of the two orthogonal directions, the approximate fundamental period of the structure, $T_d$, in each of the two orthogonal directions determined in accordance with Section 12.8.2.1 of ASCE 7, is less than 0.8 $T_S$ determined in accordance with Section 11.4.5 of ASCE 7.
2. In each of the two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than $T_S$.  
3. Equation 12.8-2 of ASCE 7 is used to determine the seismic response coefficient, $C_S$.
4. The diaphragms are rigid or are permitted to be idealized as rigid in accordance with Section 12.3.1 of ASCE 7 or, for diaphragms permitted to be idealized as flexible in accordance with Section 12.3.1 of ASCE 7, the distances between vertical elements of the seismic force-resisting system do not exceed 40 feet (12 192 mm).

1613.3.5.2 Simplified design procedure. Where the alternate simplified design procedure of ASCE 7 is used, the seismic design category shall be determined in accordance with ASCE 7.

Reason: "Science is simply common sense at its best—that is, rigidly accurate in observation, and merciless to fallacy in logic."

— Thomas Henry Huxley

Proposal 1613.3.1 Mapped acceleration lateral design strength parameters. deletes "the parameters $S_0$ and $S_1$ . . . determined from the 0.2 and 1-second spectral response accelerations shown on Figures 1613.3.1(1) through 1613.3.1(8). This means so-called "seismic design categories" are, by definition, no longer determinable!

- This is a good thing!
  The rules are arbitrary and are not based on any kind of "observation," rigidly accurate or otherwise.
- The case that earthquake-resistant design should be relaxed - based upon Risk Occupancy and Use Categories has never been logically presented.
- The present heightened awareness and interest in "community resilience" against earthquakes and other natural disasters argues against preserving this, apparently penny, nickel, dime, and quarter (mostly throw-away) buildings. Community resilience is predicated on thinking and planning for "the earthquake after the next earthquake" - not just recovering from the next earthquake!
- With the yo-yo-ing swings in the USGS so-called ground motion response accelerations; seismic design categories can change in unpredictable and unmonitored fashion.
- When the real earthquake happens, all structures will suffer its full impact; and those with the relaxed design requirements and less stringent inspection and quality assurance will bear the greater brunt!
- It is both a fabrication and a great dishonesty to have all this convoluted and high sounding terminology that for too long has been misleading not only engineers but also the greater public as to what we realistically ought to be preparing for.
- A great deficiency is that seismic design category is not amenable to presentation in a map format. Whereas "lateral design strength coefficients" are.

1988 Uniform Building Code

2016 ICC PUBLIC COMMENT AGENDA Page 2349
COST IMPACT

**Cost Impact:** Will increase the cost of construction

Will increase the cost of construction?

Will, in some cases, not increase the cost of construction

Will, in some cases, increase the cost of construction.

But, when one looks at the true "life-cycle" costs of a large population of buildings, when subjected to the maximum potential earthquakes that are inevitably in their future . . . better earthquake-resistance for all "occupancy and use" categories is the most economical choice.

Since the mapped maximum considered earthquake spectral response accelerations can fluctuate up-or-down between map editions, this could effect cost accordingly, but I estimate that this would be minor.
This proposal may or may not affect the cost of construction. This is (1) because detached one- and two-family dwellings must be already built to withstand the lateral forces due to wind; and (2) must include basements, "safe rooms"), or other afforded protections to protect occupants against the deadly impacts of hurricanes and tornadoes.

The point is: Detached one- and two-family need to consider the maximum Magnitude of realistic scenario earthquakes that they could, in fact, experience.

And not be constructed vulnerable to earthquakes, because a flawed numerical hazard model "guesses" incorrectly as to the likelihood or possibility of earthquakes. This should remain a rational and a scientific decision based upon protecting both public safety and property. A second point is that "cost" due to structural elements is almost always less than 80% of the cost of a building!

"In general, better seismic performance is achieved through increased lateral design forces (i.e., base shear), and detailing requirements that improve structural connection strength or structural member behavior in the inelastic range of response. Requirements for seismic bracing and anchorage of nonstructural components reduce potential for nonstructural damage and loss of building (or system) functionality."*

* viii, Executive Summary, NIST GCR 14-917-26
NEHRP Consultants Joint Venture A partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering.

In general, where costs might be increased, cost premiums above requirements for wind tend to fall within a range of +1-3%. For cases where seismic requirements would be now additional to what previous codes either applied/neglected/failed to enforce, estimates probably would fall within the range of 0.25 - 1%.

`{{1143}}`

**Public Hearing Results**

**Committee Action:** Disapproved

**Committee Reason:** The proposed deletion of the seismic design category requirements would be inconsistent with utilization of the ASEC 7 earthquake load provisions.

**Assembly Action:** None

**Individual Consideration Agenda**

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com) requests Approve as Submitted.

Commenter's Reason: "It is better to be right than to be consistent."
- Winston Churchill

"If your definition is wrong, you'll look for the wrong thing."
- Carol Cleland

“Anyone who conducts an argument by appealing to authority is not using his intelligence; he is just using his memory."
- Leonardo da Vinci

“Just because it comes from a consensus standard doesn't mean it isn't without problems."
- Jay Crandell


“Science is simply common sense at its best—that is, rigidly accurate in observation, and merciless to fallacy in logic.”

— Thomas Henry Huxley

The Committee Reason for Disapproval is based on Logical Fallacies: Appeal to Authority; Begging the Question (Circular Reasoning); and (my favorite) STRAWMAN. It does not even address the matters that are well developed and described in the Proponent Reason Statement.

What is a Straw Man fallacy?

In philosophy, a formal fallacy (also called logical fallacy) is a pattern of reasoning rendered invalid by a flaw in its logical structure that can neatly be expressed in a standard logic system, for example propositional logic. An argument that is formally fallacious is always considered wrong.

The deletion here is necessary to achieve consistency with the package of associated code changes. Since the design values map is to be changed, per S118-16, there will no longer be S5 and S2 -- with which to determine seismic design categories -- and the language/terminology of this section, therefore, no longer applies.

See further background and discussion under Public Comment S 118-16.

Bibliography:

See BIBLIOGRAPHY under Public Comment S118-16

S122-16
Proposed Change as Submitted

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com)

2015 International Building Code

Delete and substitute as follows:

1615.1 General. High-rise buildings that are assigned to Risk Category III or IV shall comply with the requirements of this section. Frame structures shall comply with the requirements of Section 1615.3. Bearing wall structures shall comply with the requirements of Section 1615.4.

High-rise buildings are required to be assigned to Occupancy and Use Category III or IV, and shall comply with the requirements of this section. Frame structures shall comply with the requirements of Section 1615.3. Bearing wall structures shall comply with the requirements of Section 1615.4.

Reason: High rise buildings (> 75 ft) are an important factor in determining community resilience, and therefore they are worthy of additional design effort and care.

Cities, Earthquakes, and Time


https://www.youtube.com/watch?v=EdKJna-MYY4

Bibliography:

See also BIBLIOGRAPHY in Proposal: Figure 1613.3.1 RISK-TARGETED MCER

Cost Breakdown of Nonstructural Building Elements

Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake.
doi:http://dx.doi.org/10.1193/1.4000032
http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032

Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Permalink: http://dx.doi.org/10.1061/(ASCE)CF.1943-5509.0000630
Read More: http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630

Homeowner's Guide to Earthquake Safety

Retrofitting Questions and Answers
Earthquake Safety, Inc., 2015 (web based)
http://www.earthquakesafety.com/earthquake-retrofitting-faq.html

Cost and Seismic Design
https://www.google.com/#q=cost+and+seismic+design+christopher+arnold+pdf

Earthquake Architecture website
http://www.iitk.ac.in/nicee/wcee/article/14_05-06-0185.PDF

Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee

Cost Impact: Will increase the cost of construction
Occupancy and Use Category II is too low, and it will also adversely impact life-cycle costs.
Committee Action: Disapproved

Committee Reason: This proposal was not justified. Replacing risk categories in this section would create conflicts with other code sections.

Assembly Action: None

Public Hearing Results

Individual Consideration Agenda

Public Comment 1:

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

1615.1 General. **High-rise buildings** shall be assigned to **Occupancy and Use Category Risk Categories III or IV**, and shall comply with the requirements of this section. Frame structures shall comply with the requirements of Section 1615.3. Bearing wall structures shall comply with the requirements of Section 1615.4.

Commenter's Reason: "It is better to be right than to be consistent."

- Winston Churchill

"If your definition is wrong, you'll look for the wrong thing."

- Carol Cleland

“Anyone who conducts an argument by appealing to authority is not using his intelligence; he is just using his memory.”

- Leonardo da Vinci

“Just because it comes from a consensus standard doesn’t mean it isn’t without problems.”

- Jay Crandell

The Committee Reason for Disapproval is based on the **Logical Fallacy: STRAW MAN**. It does not even address the matters that are well developed and described in the Reason Statement, namely that High rise buildings (> 75 ft) are an important factor in determining community resilience, and therefore they are worthy of additional design effort and care, **requiring** now that they shall be assigned to **Risk Categories III or IV**. The issue was always about requiring them to be Risk Category III or Risk Category IV . . . never Risk Category II. The charging language was formerly changed to be consistent with the Proponents other Code Change Proposals, predicated on S118-16.
What is a Straw Man fallacy?

- The Straw Man fallacy is committed when a person simply ignores a person's actual position and substitutes a distorted, exaggerated or misrepresented version of that position. This sort of "reasoning" has the following pattern:
  - Person A has position X.
  - Person B presents position Y (which is a distorted version of X).
  - Person B attacks position Y.
  - Therefore X is false/incorrect/flawed.

See further background and discussion under Public Comment S118-16

Bibliography:
See BIBLIOGRAPHY under Public Comment S118-16
Proposed Change as Submitted

Proponent: Bonnie Manley, AISI, representing American Iron and Steel Institute (bmanley@steel.org)

2015 International Building Code

Revise as follows:

1704.2 Special inspections and tests. Where application is made to the building official for construction as specified in Section 105, the owner or the owner's authorized agent, other than the contractor, shall employ one or more approved agencies to provide special inspections and tests during construction on the types of work specified in Section 1705 and identify the approved agencies to the building official. These special inspections and tests are in addition to the inspections by the building official that are identified in Section 110.

Exceptions:

1. Special inspections and tests are not required for construction of a minor nature or as warranted by conditions in the jurisdiction as approved by the building official.
2. Unless otherwise required by the building official, special inspections and tests are not required for Group U occupancies that are accessory to a residential occupancy including, but not limited to, those listed in Section 312.1.
3. Special inspections and tests are not required for portions of structures designed and constructed in accordance with the cold-formed steel light-frame construction provisions of Section 2211.2 or the conventional light-frame construction provisions of Section 2308.
4. The contractor is permitted to employ the approved agencies where the contractor is also the owner.

Add new text as follows:

1705.2.2 Cold-formed steel. Special inspections for cold-formed steel shall be in accordance with this section.

Revise as follows:

1705.2.2.1 Cold-formed steel deck. No change to text.

Add new text as follows:

1705.2.2.2 Cold-formed steel light-frame construction. Special inspections and qualifications of welding and mechanical fastening special inspectors for cold-formed steel light frame construction, which is designed and installed in accordance with Section 2211.1, shall be in accordance with the quality assurance inspection requirements of AISI S240 Chapter D, excluding AISI S240 Section D6.9.

1705.2.2.3 Cold-formed steel trusses, Special inspection of trusses of cold-formed steel light frame construction shall be in accordance with this section.

1705.2.2.3.1 General. For cold-formed steel trusses, quality assurance inspection in accordance with AISI S240 Chapter D, excluding AISI S240 Section D6.9, shall verify compliance with the approved construction documents and the approved truss submittal package as defined in AISI S202.

Revise as follows:

1705.2.4 1705.2.2.3.2 Cold-formed steel trusses spanning 60 feet or greater. Where a cold-formed steel truss clear span is 60 feet (18 288 mm) or greater, the special inspector shall additionally verify that the temporary installation restraint/bracing and the permanent individual truss member restraint/bracing are installed in accordance with the approved truss submittal design as specified in Section 2211.1.3.

Delete and substitute as follows:

1705.11.2 Cold-formed steel light-frame construction. Periodic special inspection is required for welding operations of elements of the main windforce-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring, and other fastening of elements of the main windforce resisting system, including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

Exceptions: Special inspections are not required for cold-formed steel light frame shear walls and diaphragms, including screwing, bolting, anchoring and other fastening to components of the windforce resisting system, where either of the following applies:

1. The sheathing is gypsum board or fiberboard.
2. The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c.).
Special inspections for the wind resistance of cold-formed steel lateral force resisting systems, which are designed and installed in accordance with Section 2211.1, shall be in accordance with the quality assurance inspection requirements of AISI S240 Section D6.9.

1705.12.3 Cold-formed steel light-frame construction. For the seismic force-resisting systems of structures assigned to Seismic Design Category C, D, E or F, periodic special inspection shall be required:
1. For welding operations of elements of the seismic force-resisting system; and
2. For screw attachment, bolting, anchoring and other fastening of elements of the seismic force-resisting system, including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

Exception: Special inspections are not required for cold-formed steel light-frame shear walls and diaphragms, including screw installation, bolting, anchoring and other fastening to components of the seismic force-resisting system, where either of the following applies:
1. The sheathing is gypsum board or fiberboard.
2. The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) on center.

Special inspections for the seismic resistance of cold-formed steel lateral force resisting systems, which are designed and installed in accordance with Section 2211.1, shall be in accordance with the quality assurance inspection requirements of AISI S240 Section D6.9.

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
AISI S202, Code of Standard Practice for Cold-Formed Steel Structural Framing, 2015
AISI S240, North American Standard for Cold-Formed Steel Structural Framing, 2015

Reason: This proposal is one in a series adopting the latest generation of AISI standards for cold-formed steel. It has been offered by the industry with the intent of coordinating with a comparable NCSEA proposal adopting special inspection requirements for wood light frame construction. In the IBC, light frame construction includes a solution in both wood and cold-formed steel framing. Often on projects, these materials are direct competitors. Rather than upset the balance in the light frame industry, the special inspection requirements should remain consistent and coordinated between the two structural materials, which means that both proposals must succeed or fail together.

This particular proposal focuses on Chapter 17 by incorporating references to two new cold-formed steel standards -- AISI S240 and AISI S202. Both standards are published and available for a free download at: www.aisistandards.org.

AISI S240, North American Standard for Cold-Formed Steel Structural Framing, addresses requirements for construction with cold-formed steel structural framing that are common to prescriptive and engineered light frame construction. This comprehensive standard was formed by merging the following AISI standards:
- AISI S200, North American Standard for Cold-Formed Steel Framing-General Provisions
- AISI S210, North American Standard for Cold-Formed Steel Framing--Floor and Roof System Design
- AISI S211, North American Standard for Cold-Formed Steel Framing--Wall Stud Design
- AISI S212, North American Standard for Cold-Formed Steel Framing--Header Design
- AISI S213, North American Standard for Cold-Formed Steel Framing-- Lateral Design
- AISI S214, North American Standard for Cold-Formed Steel Framing--Truss Design

Consequently, AISI S240 supersedes all previous editions of the above mentioned individual AISI standards. Additionally, the standard builds upon this foundation by adding the first comprehensive chapter on quality control and quality assurance for cold-formed steel light frame construction.

AISI S202, Code of Standard Practice for Cold-formed Steel Structural Framing, is intended to service as a state-of-the-art mandatory document for establishing contractual relationships between various parties in a construction project where cold-formed steel structural materials, components and assemblies are used. While it is not specifically intended to be a direct reference in the building code, portions of AISI S202 are recommended for adoption in this proposal to establish the minimum requirements for cold-formed steel truss design drawings.

Modifications specific to Chapter 17 include the following:
- Section 1705.2.2: Requirements for special inspection of cold-formed steel are editorially consolidated into this section.
- Section 1705.2.2.1: No substantive changes are proposed for cold-formed steel deck.
- Section 1705.2.2.2: Reference to AISI S240, Chapter D is made for special inspection of cold-formed steel structural framing systems that are designed and installed in accordance with Section 2211.1. AISI S240 Chapter D provides requirements for both quality control and quality assurance; however, for the purposes of IBC Chapter 17, the quality assurance inspection requirements are invoked for special inspections along with the qualifications of special inspectors.
This approach is similar to that taken by both AISC 360, Chapter N, and SDI QA/QC. In fact, the provisions in AISI S240 have been closely modeled on those two standards. AISI S240, Section D6.9 is specific to lateral force resisting systems intended for high wind and high seismic areas. Therefore, it has been excluded from adoption here and, instead, is adopted in Section 1705.11 for high wind and Section 1705.12 for high seismic.

- Section 1705.2.2.3: Special inspection requirements for cold-formed steel trusses are consolidated into a dedicated section.
- Section 1705.2.2.3.1: Upon review, it was determined that in addition to verifying compliance with the approved construction documents (specifically per AISI S240 Section D6.8), the special inspector (i.e. quality assurance inspector) needed to also check compliance with the approved truss submittal package. This ensures that the special inspector reviews not only the placement of the trusses, but also the permanent individual truss member restraint/bracing details. It is anticipated that this particular requirement will be clarified in the next edition of AISI S240. Truss submittal package is defined in AISI S202 as follows:

  Truss Submittal Package. Package consisting of each individual truss design drawing, and, as applicable, the truss placement diagram, the cover/truss index sheet, permanent individual truss member restraint/bracing details designed in accordance with generally accepted engineering practice, applicable permanent individual truss member restraint/bracing details, and any other structural details germane to the trusses. [AISI S202]

- Section 1705.2.2.3.2: Now that inspection of the permanent individual truss member restraint/bracing is covered in Section 1705.2.2.3.1 for all cold-formed steel light frame trusses, reference to permanent individual truss member restraint/bracing in this section is redundant and potentially confusing, so it is recommended for deletion.
- Section 1705.11.2: AISI S240, Section D6.9 now includes these requirements for cold-formed steel lateral force resisting systems, so the requirement has been replaced with a direct reference to the provisions of AISI S240, Section D6.9.
- Section 1705.12.3: AISI S240, Section D6.9 now includes these requirements for cold-formed steel lateral force resisting systems, so the requirement has been replaced with a direct reference to the provisions of AISI S240, Section D6.9.

**Cost Impact:** Will increase the cost of construction

This code change proposal adopts the latest industry standard for cold-formed steel. At this time, it is difficult to anticipate how cost of construction will be fully impacted, other than to note that some of the additional costs will be offset by new efficiencies in the design, installation, and inspection of cold-formed steel.

**Analysis:** A review of the standard(s) proposed for inclusion in the code, AISI S202 & AISI S240, with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 1, 2016.

S129-16 : 1704.2-
MANLEY11503

<table>
<thead>
<tr>
<th>Public Hearing Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>Committee Action:</td>
</tr>
<tr>
<td>Committee Reason:</td>
</tr>
<tr>
<td>Assembly Action:</td>
</tr>
</tbody>
</table>

**Individual Consideration Agenda**

Public Comment 1:

Proponent : Bonnie Manley, AISI, representing American Iron and Steel Institute (bmanley@steel.org) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

1705.2.2.2 Cold-formed steel light-frame construction. Special inspections and qualifications of welding and mechanical fastening special inspectors for cold formed steel light frame construction, which is designed and installed in accordance with Section 2211.1, shall be in accordance with the quality assurance inspection requirements of AISI S240 Chapter D, excluding AISI S240 Section D6.9.

1705.2.2.3 1705.2.2.2 Cold-formed steel trusses. No change to text.
1705.2.2.2.1 General. For cold-formed steel trusses, with overall height of 60 inches (1,524 mm) or greater, quality assurance inspection in accordance with AISI S240 Chapter D, excluding AISI S240 Section D6.9, D6.8 shall verify compliance that installation of the permanent individual truss web member restraint/bracing is in accordance with the approved construction documents and the approved truss submittal package as defined in AISI S202.

1705.2.2.2.2 Cold-formed steel trusses spanning 60 feet or greater. No change to text.

Commenter's Reason: The purpose of this public comment is twofold. First, it coordinates with the Proposal S137-16 on special inspection of wood construction, which was disapproved. Section 1705.2.2.2 of the original proposal, which introduces requirements for cold-formed steel light frame construction quality assurance, is recommended for deletion. Second, the public comment coordinates with the modifications that were approved as submitted in Proposal S138-16. In that proposal, the proponent, NCSEA, instituted special inspection for permanent individual truss member restraint/bracing in wood trusses 60 inches and higher, presenting evidence of failures resulting from missing or inadequate truss web member bracing. To bring that concept into Proposal S129, this public comment introduces the same 60 inch trigger for quality assurance inspection for the permanent individual truss member restraint/bracing in CFS trusses and clarifies the applicable section in AISI S240.

Additionally, for CFS trusses, the inspection of permanent restraint/bracing has been further focused to truss web member bracing only. This is because, with wood truss buildings, the bottom chord is typically braced with rigid ceiling panels; whereas, for CFS truss buildings, bottom chord bracing is typically accomplished with discrete braces. To require inspection of all permanent individual truss member restraint/bracing in CFS trusses would disproportionately increase the number of CFS truss jobs that would require inspection, since most installations have discrete bottom chord braces while they may or may not have truss web member bracing. Focusing this requirement on inspection of just the truss web member bracing for CFS trusses addresses the primary concern raised in Proposal S138-16, while triggering inspection on a similar number of light frame jobs.
**Proposed Change as Submitted**

**Proponent**: Satyendra Ghosh, representing Precast/Prestressed Concrete Institute (skghoshinc@gmail.com)

**2015 International Building Code**

Revise as follows:

### TABLE 1705.3

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

<table>
<thead>
<tr>
<th>TYPE</th>
<th>CONTINUOUS SPECIAL INSPECTION</th>
<th>PERIODIC SPECIAL INSPECTION</th>
<th>REFERENCED STANDARD</th>
<th>IBC REFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Inspect reinforcement, including prestressing tendons, and verify placement.</td>
<td>—</td>
<td>X</td>
<td>ACI 318 Ch. 20, 25.2, 25.3, 26.6.1-26.6.3</td>
</tr>
<tr>
<td>2.</td>
<td>Reinforcing bar welding:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a.</td>
<td>Verify weldability of reinforcing bars other than ASTM A 706;</td>
<td>—</td>
<td>X</td>
<td>AWS D1.4</td>
</tr>
<tr>
<td>b.</td>
<td>Inspect single-pass fillet welds, maximum 5/16&quot;; and inspect welding of reinforcing steel resisting flexural and axial forces in intermediate and special moment frames, and boundary elements of special structural walls of concrete.</td>
<td>X</td>
<td>X</td>
<td>ACI 318: 26.6.4</td>
</tr>
<tr>
<td>c.</td>
<td>Inspect welding of shear reinforcement.</td>
<td>X</td>
<td></td>
<td></td>
</tr>
<tr>
<td>d.</td>
<td>Inspect all other welds.</td>
<td>X</td>
<td>X</td>
<td>ACI 318: 26.6.4</td>
</tr>
<tr>
<td>3.</td>
<td>Inspect anchors cast in concrete.</td>
<td>—</td>
<td>X</td>
<td>ACI 318: 17.8.2</td>
</tr>
<tr>
<td>4.</td>
<td>Inspect anchors post-installed in hardened concrete members.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a.</td>
<td>Adhesive anchors installed in horizontally or upwardly inclined orientations to resist sustained tension loads.</td>
<td>X</td>
<td></td>
<td>ACI 318: 17.8.2.4</td>
</tr>
<tr>
<td>b.</td>
<td>Mechanical anchors and adhesive anchors not defined in 4.a</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.</td>
<td>Verify use of required design mix.</td>
<td>—</td>
<td>X</td>
<td>ACI 318: Ch. 19, 26.4.3, 26.4.4</td>
</tr>
<tr>
<td>6.</td>
<td>Prior to concrete placement, fabricate specimens for strength tests, perform slump and air content tests, and determine the temperature of the concrete.</td>
<td>X</td>
<td>—</td>
<td>ASTM C 172 ASTM C 31 ACI 318: 26.4, 26.12</td>
</tr>
<tr>
<td>7.</td>
<td>Inspect concrete and shotcrete placement for proper application techniques.</td>
<td>X</td>
<td>—</td>
<td>ACI 318: 26.5</td>
</tr>
<tr>
<td>8.</td>
<td>Verify maintenance of specified curing temperature and techniques.</td>
<td>—</td>
<td>X</td>
<td>ACI 318: 26.5.3-26.5.5</td>
</tr>
<tr>
<td>9.</td>
<td>Inspect prestressed concrete for:</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a.</td>
<td>Application of prestressing forces; and</td>
<td>X</td>
<td>—</td>
<td>ACI 318: 26.10</td>
</tr>
<tr>
<td>b.</td>
<td>Grouting of bonded prestressing tendons.</td>
<td>X</td>
<td>—</td>
<td></td>
</tr>
<tr>
<td>10.</td>
<td>Inspect erection of precast concrete members.</td>
<td>—</td>
<td>X</td>
<td>ACI 318: Ch. 26.9</td>
</tr>
<tr>
<td>11.</td>
<td>Verify in-situ concrete strength, prior to stressing of tendons in post-tensioned concrete and prior to removal of shores and forms from beams and structural slabs.</td>
<td>—</td>
<td>X</td>
<td>ACI 318: 26.11.2</td>
</tr>
<tr>
<td>12.</td>
<td>Inspect formwork for shape, location and dimensions of the concrete member being formed.</td>
<td>—</td>
<td>X</td>
<td>ACI 318: 26.11.1.2(b)</td>
</tr>
</tbody>
</table>
Committee Action: Disapproved

For SI: 1 inch = 25.4 mm.

a. Where applicable, see also Section 1705.12, Special inspections for seismic resistance.

b. Specific requirements for special inspection shall be included in the research report for the anchor issued by an approved source in accordance with 17.8.2 in ACI 318, or other qualification procedures. Where specific requirements are not provided, special inspection requirements shall be specified by the registered design professional and shall be approved by the building official prior to the commencement of the work

**Reason:** This proposal seeks to reverse a substantive change made as part of an organizational change in the 2015 IBC by Code Change S148-12. The change is shown below.

The Committee's reason for approving this code change as submitted was: "This code change simplifies the special inspections for steel by removing requirements for reinforcing bars that don't belong under steel." This reason obviously is strictly organizational.

We believe that tying the extent of special inspection of reinforcing bars (continuous or periodic) to the function of those bars (reinforcement resisting flexural and axial forces in intermediate and special moment frames, and boundary elements of special structural walls of concrete; shear reinforcement) is logical. Continuous special inspection can then be mandated for welds, the failure of which is liable to have serious, even catastrophic, consequences. The logic behind mandating special inspection for all welds other than those of a particular type (and even there only up to a maximum size) is, on the other hand, difficult to see. The exception provided almost never applies. Fillet welds are used only at the ends of reinforcing bars, to connect them to plates; those welds are done at the shop using an automated welding process. Otherwise, the welds used on reinforcing bars are flare bevel groove welds or full penetration butt welds. Thus the 2015 IBC change represents an unnecessary expansion of special inspection requirements that does not result in any apparent benefit.

**Cost Impact:** Will not increase the cost of construction

The cost of precast concrete construction, where welding of reinforcing bars is not uncommon, should decrease modestly through elimination of unnecessary continuous special inspection in many cases.

<table>
<thead>
<tr>
<th>TABLE 1705.2.2 REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL</th>
</tr>
</thead>
<tbody>
<tr>
<td>VERIFICATION AND INSPECTION</td>
</tr>
<tr>
<td>1. Material verification of cold-formed steel deck:</td>
</tr>
<tr>
<td>a. Identification markings to conform to ASTM standards specified in the approved construction documents</td>
</tr>
<tr>
<td>b. Manufacturers' certified test reports.</td>
</tr>
<tr>
<td>2. Inspection of welding;</td>
</tr>
<tr>
<td>a. Cold-formed steel deck</td>
</tr>
<tr>
<td>1. Floor and roof deck welds</td>
</tr>
<tr>
<td>b. Reinforcing steel bars:</td>
</tr>
<tr>
<td>1. Verification of weldability of reinforcing steel bars other than ASTM A 706.</td>
</tr>
<tr>
<td>2. Reinforcing steel resisting flexural and axial forces in intermediate and special moment frames, and boundary elements of special structural walls of concrete and shear reinforcement.</td>
</tr>
<tr>
<td>3. Shear reinforcement.</td>
</tr>
<tr>
<td>4. Other reinforcing steel.</td>
</tr>
<tr>
<td>5. Single-pass fillet welds, maximum 5/16”.</td>
</tr>
<tr>
<td>3. All other welds</td>
</tr>
</tbody>
</table>

The Committee's reason for approving this code change as submitted was: "This code change simplifies the special inspections for steel by removing requirements for reinforcing bars that don't belong under steel." This reason obviously is strictly organizational.

The exception provided almost never applies. Fillet welds are used only at the ends of reinforcing bars, to connect them to plates; those welds are done at the shop using an automated welding process. Otherwise, the welds used on reinforcing bars are flare bevel groove welds or full penetration butt welds. Thus the 2015 IBC change represents an unnecessary expansion of special inspection requirements that does not result in any apparent benefit.

Public Hearing Results

Committee Action: Disapproved

S136-16 : TABLE 1705.3-1705.12 GHOSH12750

2016 ICC PUBLIC COMMENT AGENDA Page 2361
Committee Reason: The committee had a concern that, with the proposed changes to the concrete special inspections, some critical welds could be missed. It is suggested that a public comment be submitted to address this concern.

Assembly Action: None

Individual Consideration Agenda

Proponent: Scott Campbell, representing Portland Cement Association (scampbell@cement.org) requests Approve as Submitted.

Commenter's Reason: The proposed change ensures continuous special inspection of reinforcing bar welding in critical locations based on the loading conditions, and adds continuous special inspection for shear reinforcing. It allows periodic inspection in less critical regions. All these changes are in line with earlier versions of the code and enhance safety through inspection of critical items. The current language only requires periodic special inspection of all reinforcing bar welding which does not provide for adequate safety in all cases.

Proponent: Satyendra Ghosh, representing Precast/Prestressed Concrete Institute (skghoshinc@gmail.com) requests Approve as Submitted.

Commenter's Reason: We believe that tying the extent of special inspection of reinforcing bar welding (continuous or periodic) to the function of those bars (reinforcement resisting flexural and axial forces in intermediate and special moment frames, and boundary elements of special structural walls of concrete; shear reinforcement) is logical. Continuous special inspection can then be mandated for welds, the failure of which is liable to have serious, even catastrophic, consequences. That is the way IBC Chapter 17 requirements were from the 2000 through the 2012 IBC. The requirements were changed in the 2015 IBC through a code change that was supposed to be organizational. The logic behind mandating special inspection for all welds other than those of a particular type (and even there only up to a maximum size) is, on the other hand, difficult to see. The exception provided almost never applies. Fillet welds are used only at the ends of reinforcing bars, to connect them to plates; those welds are typically done at the shop using an automated welding process. Otherwise, the welds used on reinforcing bars are flare bevel groove welds or full penetration butt welds. The presumably organizational change made in the 2015 IBC represents an unnecessary expansion of special inspection requirements that does not result in any apparent benefit. It causes a particular and totally unnecessary hardship for one important segment of the concrete industry – the precast concrete industry.

2015 IBC Section 1705.3.1 requires special inspection of welding of reinforcing bars to be in accordance with AWS D1.4-2011. In the AWS D1.4 chapter on inspection, there is only one reference to inspection intervals. In Section 7.5.4, AWS states: “The inspector shall, at suitable intervals, observe the technique and performance of each welder to verify that the applicable requirements of this code are met.” If the inspection of basically all welding of reinforcing bars were so critical, wouldn't it seem logical that AWS, the authority on welding, would have specified continuous inspection, rather than inspection at undefined “suitable intervals”?

The Committee's reason for recommending disapproval was: "The committee had a concern that, with the proposed changes to the concrete special inspections, some critical welds could be missed." This concern is unfounded. The proposed changes bring IBC requirements back to where they always were. The only critical welds specifically mentioned at the IBC structural hearing were welds on reinforcement in columns supporting discontinued shear walls or frames. By ACI 318 requirements those columns would be treated as special moment frame columns (ACI 318-14 Section 18.7.5.6) or intermediate moment frame columns (ACI 318-14 Section 18.4.3.6) and the continuous special inspection requirements for welds would apply.
Proposed Change as Submitted

Proponent: Gregory Robinson, representing National Council of Structural Engineers Associations (grobinson@lbyd.com)

2015 International Building Code

Revise as follows:

1705.5 Wood construction. Special inspections of prefabricated wood structural elements and assemblies shall be in accordance with Section 1704.2.5. Special inspections of site-built assemblies including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs, shall be in accordance with this section and Table 1705.5.

Exceptions:
1. Buildings and structures assigned to Risk Category I.
2. Buildings and structures assigned to Risk Category II that are not more than 3 stories in height above grade plane and are not included in Sections 1705.11 or 1705.12.

Add new text as follows:

<table>
<thead>
<tr>
<th>TABLE 1705.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>REQUIRED SPECIAL INSPECTION OF WOOD CONSTRUCTION</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TYPE</th>
<th>CONTINUOUS SPECIAL INSPECTION</th>
<th>PERIODIC SPECIAL INSPECTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Verify that grade stamp on framing lumber, plywood and OSB panels conform to the approved construction documents.</td>
<td>-</td>
<td>X</td>
</tr>
<tr>
<td>2. Verify that wood connections including quantity, diameter, length and spacing of nails or staples; bolt diameter, length and location; anchor bolt diameter, length, spacing and location; tie-down and hold down diameter, length, location and configuration; and types of beam hangers and framing anchors, conform to the approved construction documents.</td>
<td>-</td>
<td>X</td>
</tr>
<tr>
<td>3. Verify that details of wood framing including framing layout, member sizes, blocking, bridging and bearing lengths conform to the approved construction documents.</td>
<td>-</td>
<td>X</td>
</tr>
<tr>
<td>4. Inspect diaphragms and shear walls to verify that wood structural panel sheathing is of the grade and thickness, the nominal size of framing members at adjoining panel edges and the nail or staple diameter and length, conform to the approved construction documents.</td>
<td>-</td>
<td>X</td>
</tr>
</tbody>
</table>

a. Where applicable, also see Section 1705.11 Special inspections for wind resistance and Section 1705.12 Special inspections for seismic resistance.

Revise as follows:

1705.11.1 Structural wood. Continuous special inspection is in addition to the requirements of Section 1705.5. Continuous special inspection shall be required during field gluing operations of elements of the main windforce-resisting system.

Periodic special inspection is required for nailing, bolting, anchoring and other fastening of elements of the main windforce-resisting system, including wood shear walls, wood diaphragms, drag struts, braces and hold-downs.

Exception: Special inspections are not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other elements of the main windforce-resisting system, where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center.

1705.12.2 Structural wood. For additional special inspections shall be required for the seismic force-resisting systems of structures assigned to Seismic Design Category C, D, E or F:

1. Continuous special inspection shall be required during field gluing operations of elements of the seismic force-resisting system.
2. Periodic special inspection shall be required for nailing, bolting, anchoring and other fastening of elements of the seismic force-resisting system, including wood shear walls, wood diaphragms, drag struts, braces, shear panels and hold-downs.

Exception: Special inspections are not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other elements of the seismic force-resisting system, where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center.

Reason: NCSEA believes that wood construction has become more commonly used for load bearing applications of significant height and in regions with moderate and high seismic and wind concerns. These types of construction should be subject to Special Inspections in a similar manner and to a comparable extent as other systems such as concrete, structural steel and masonry. There is a large group of buildings constructed with wood construction that is not subject to the same requirements for Special Inspection as the same buildings constructed with structural steel, concrete or masonry. This proposal seeks to
correct this deficiency in the Code.

Exceptions are provided to limit the applicability of these provisions to exclude single and two family dwellings, small commercial, agricultural structures and buildings of lesser occupancies unless these minor structures are subject to the existing requirements of 1705.11 Special inspections for wind resistance, and 1705.12 Special inspection for seismic resistance.

The proposed revisions to 1705.11 and 1705.12 are to coordinate between the additional requirements for Special Inspections in high seismic and high wind conditions and the proposed provisions. The proposed changes to 1705.11 and 1705.12 do not reduce the requirements of these sections they only prevent the exceptions for these sections from conflicting with the new requirements. In addition, notes are added to the Table 1705.5 to refer to 1705.11 and 1705.112 for additional requirements.

For example, one can compare the current Special Inspection requirements of the IBC for Wood Light Frame Construction to other structural systems using a sample building. For comparison, use a five story building where 1705.11 (wind) and 1705.12 (seismic) do not apply; there are no trusses spanning 60 feet or more and there are no high-load diaphragms.

If this building is constructed using a cast in place reinforced concrete system, the IBC requires Special Inspection of twelve (12) different items. These range from concrete mix designs, field sampling and testing for strength to reinforcing steel placement. If this building is constructed using structural steel, the IBC requires Special Inspections of six (6) items for anchor rods, twenty two (22) items to for welding, twelve (12) items for bolting and four (4) general items for a total of 44 areas of Special Inspection. Currently there are no requirements for Special Inspections for this same building constructed with wood.

For this example building, the same level of inspection by the Building Official is required for each system but there is a vast disparity between the systems with regard to Special Inspections; twelve vs. forty four vs. zero.

Cost Impact: Will not increase the cost of construction
There will be no increase in construction cost due to the increased Special Inspection that will take place. Currently structural engineers provide for these inspections in project specifications. However, individual requirements vary greatly and there is not a consistent level of requirements. Standardization of these requirements in the Code will reduce delays and added costs due to confusion created by varying specifications. The improved field quality assurance will improve safety and reduce field errors resulting in a savings in construction cost and schedule. The improved public safety and potential reduction in construction cost support adoption of this proposal.

Committee Reason: There may be a place for special inspection in wood frame buildings. The code already contains required special inspections where high lateral loads are a concern. Other inspections could be done by the building official. The committee encourages those on both side of this issue to work on proposals that will have wider support.

Assembly Action: None

Individual Consideration Agenda

Proponent: Scott Campbell, representing Portland Cement Association (scampbell@cement.org) requests Approve as Submitted.

Commenter's Reason: Special inspection of critical items in wood structures is necessary to ensure safety at a level comparable to other building systems. The lack of special inspection can compromise safety and does not ensure compliance with the design documents. Adequate exemptions are included in the proposal for small structures that do not typically require special inspection.
Proponent: Gregory Robinson, National Council of Structural Engineers Association, representing National Council of Structural Engineers Associations (grobinson@lbyd.com) requests Approve as Submitted.

Commenter's Reason: Many Professional Engineers across the United States believe that inspection of commercial wood structures is not currently addressed adequately within the building code or by the wood industry. These Professional Engineers who design and observe wood projects during construction will occasionally find that the intent of the drawings is not being incorporated into the structure. While most wood structures are designed, constructed, and inspected by the Building Official without issue, there are situations where exceptions do occur that could impact the safety of the public or the integrity of the structure after a disaster.

The special inspections for wood construction specified in the code are so specific that rarely will any additional inspections occur on the project other than those provided by the Building Official. The lack of third party inspections often results in issues discovered far too late for cost effective correction resulting in delays in construction and added costs.

The inspections proposed within this proposal are reasonable to incorporate during construction, effective in reducing the time and cost to correct non-compliant work and will help to insure that buildings are safe for the public. Costs for these inspections will be cited as a reason to not support this proposal, but how much is too much to pay to have the wood framed fire station that we rely on after a disaster be inspected?

Professional Engineers designing fire stations on which we trust will be there after a disaster, the apartment complexes we live in and the schools that our children attend believe that if these structures are wood, they deserve a level of inspection that is much greater than is currently found with the building code. Please support this code change proposal as submitted.

S137-16
Proposed Change as Submitted

Proponent: Edwin Huston, representing National Council of Structural Engineers’ Associations (NCSEA)
(huston@smithhustoninc.com)

2015 International Building Code

Revise as follows:

1705.5.2 Metal-plate-connected wood trusses spanning 60 feet or greater. Where a truss clear span is Special Inspections of wood trusses, with overall height of 60 feet inches (18,288 1,524 mm) or greater, the special inspector shall be performed to verify that the temporary installation restraint/bracing and of the permanent individual truss member restraint/bracing are has been installed in accordance with the approved truss submittal package. For wood trusses with a clear span of 60 feet (18288 mm) or greater, the special inspector shall also verify during construction, that the temporary installation restraint/bracing is installed in accordance with the approved truss submittal package.

Reason: This code change proposal will harmonize the special inspection requirements for wood trusses so that they have the same requirements as open web steel joists and joist girders (section 1705.2.3), and cold formed steel trusses (AISI S240, Chapter D).

The Truss designer, as part of their design and in accordance with their responsibilities, determines if and where an individual member of a truss needs to be braced to prevent out of plane buckling when subjected to design loads. This is what is called permanent individual truss member restraint/bracing. The locations where this restraint/bracing is needed has historically been shown on the truss design drawings as a small rectangle with an “x” thru it, along with a key note that says something to the effect “bracing required at location indicated”. Then there may be a general note at the end of the submittal that typically says “Lateral bracing location indicated by symbol shown and/or by text in the bracing section of the output”, and “BSCI is the industry standard to be used”. This information is often misinterpreted or ignored by the truss installer.

The installation of the restraint/bracing is critical for the safe performance of wood trusses, and if the bracing is not installed at all, or is not installed correctly, can become a life safety issue. This issue is important enough to rise to the level requiring special inspections. This code change will ensure the bracing gets installed where it is required by the Truss designer.

One such collapse was of a fire station in Pinetop Arizona, where the permanent bracing was not installed.

![Image of a truss collapse](image-url)
Missing bracing on tall, slender compression members could be seen in several buildings in Joplin, MO after the May 2011 tornado removed portions of roof sheathing or gable end walls.
A 2008 inspection of a building in Gig Harbor, WA showed "piggy-back" trusses with missing bracing and collectors.

Cost Impact: Will increase the cost of construction. The cost of construction will increase slightly by the amount of the cost of the special inspector. This increased cost however is minimal and justified.

Committee Action: Approved as Submitted
Committee Reason: The committee agrees that because the temporary truss bracing is part of the structural design, it needs to be verified by special inspection.

Public Hearing Results

Proponent: Gary Ehrlich, National Association of Home Builders, representing National Association of Home Builders (gehrlich@nahb.org) requests Approve as Modified by this Public Comment.

Modify as Follows:
2015 International Building Code
1705.5.2 Metal-plate-connected wood trusses. Special Inspections of wood trusses, with a total length between end bearing points of 36 feet (10,973 mm) or greater and overall height of 60 inches (1,524 mm) or greater, or of wood piggyback trusses, shall be performed to verify that the installation of the permanent individual truss member restraint/bracing has been installed in accordance with the approved truss submittal package. For wood trusses with a clear span of 60 feet (18288 mm) or greater, the special inspector shall also verify during construction, that the temporary installation restraint/bracing is installed in accordance with the approved truss submittal package.

Commenter's Reason: The purpose of this public comment is to add a minimum clear span trigger for the special inspection of permanent truss bracing introduced by this code change. Roof slopes of 6:12 and greater are common in multifamily buildings, four-story townhouses and other residential buildings constructed under the IBC, and in some areas of the country the average roof slope is on the order of 8:12. For such buildings, this provision as written would trigger a special inspection for a building just 20 feet in width, or even narrower. Absent significant evidence of widespread truss failures in small residential and multifamily buildings built to the IBC, there is no justification for such a broad expansion of special inspection requirements. IBC Section 2308.5 limits the clear span of trusses on conventionally-framed buildings to 40 feet, and IRC Section R802.10.2.1 limits the width of buildings using trusses to 36 feet. The primary intent of these two provisions is to limit the roof load delivered to prescriptively-selected wall studs, but they also suggest a reasonable minimum trigger for special inspection requirements of a 36 foot length between end bearing points.

The proponent's reason statement identified piggyback trusses as a concern. This occurs where a truss is shipped to a jobsite in two or more pieces with one supporting truss and one or more cap trusses which are attached at the site during installation. Special attention to the bracing for the additional chord plane this creates in the truss assembly may be warranted, and thus piggyback trusses are called out for special inspection regardless of building width or truss span.

Proponent: Joseph Holland, representing Hoover Treated Wood Products (j holland@frtw.com) requests Disapprove.

Commenter's Reason: This change is inappropriate. It will require a special inspection for almost all building constructed with trusses. A pitch of 5 in 12 is typical today. A truss with a span of 24 feet would require a special inspector. The proponent states it is required for steel trusses 5 feet high. That is not representative of what the two material will span. How far can a steel truss five feet tall span. it is significantly more.

The examples given in the proposal show the effect of a tornado on a building and piggyback trusses but yet the picture does not show piggyback trusses. This imposes an additional cost without an incremental increase in benefit.

Proponent: Marvin Strzyzewski, representing MiTek USA, Inc. (marvins@mii.com) requests Disapprove.

Commenter's Reason: Proponent is asking for a special inspection on framing members that a Building Official is adequately capable of inspecting. The lateral restraints and diagonal bracing members along with any other material and connection specifications should be clearly specified on the bracing plan for the structure included in the submittal documents. These members should be installed and thus available for inspection during the rough framing inspection, as are all other framing members under the inspection review.

A 60 inch height limitation would require that a lot structures would incur higher costs in both inspection fees and increased time added to the construction schedule. This 60 inch height requirement seems arbitrary, what is the rationale for this?

Proponent: Maureen Traxler, representing WA Assn of Bldg Officials Code Committee (maureen.traxler@seattle.gov) requests Disapprove.

Commenter's Reason: This proposal creates a massive change in the threshold for required special inspections for wood trusses. The current threshold for permanent bracing of metal-plate-connected wood trusses is at 60-foot clear span. This proposal changes from requiring special inspection for a span of 60 feet to requiring it for height of 60 inches. With this change, almost every structure with wood trusses would require special inspection. The additional special inspections will add significantly to the cost of construction with limited added value.
While there may be justification for requiring special inspection of more projects, there is no justification for such a drastic change. Special inspection is meant for special types of construction that aren’t within the typical expertise of building inspectors. The justification offered by the proponents is consistency with the requirements for open web steel joists, joist girders, and cold formed steel trusses. Steel construction has different varying requirements and requires more specialized knowledge than wood-frame trusses. Wood construction is different than steel and doesn’t need to be treated the same.

Proponent: Larry Wainright, representing Structural Building Components Association (lwainright@qualtim.com) requests Disapprove.

Commenter’s Reason: This code proposal establishes special inspection requirements for all metal plate connected wood trusses that are 60-inches in height or deeper. The proponent argues that the special inspections are necessary because trusses are getting installed without the proper permanent web member plane lateral restraint and diagonal bracing being applied. This means that trusses are being installed where the manufacturer’s installation instructions, IBC referenced standard industry details (see Section 2303.4.1.2) and SBCA’s publicly available Building Component Safety Information (BCSI; http://support.sbcindustry.com/pubs/BCSIED2-D (http://support.sbcindustry.com/pubs/BCSIED2-D)) are all being ignored. Does this mean there is a need to have special inspections for OSB sheathing, LVL beams, 2x10 joists, 2x6 rafters, etc. since all may get installed improperly? Everyone knows how hard it is to install the OSB 3/8” edge distance requirement accurately and consistently, yet for shear walls to perform properly this is what SDPWS says shall be done.

Compared to a 3/8” edge distance inspection of all OSB installations, which does not require special inspections, getting the permanent lateral restraint and diagonal bracing installed and inspected properly should be comparatively easy. Hence, these items can easily be inspected by the building official during their normal inspections.

The proponent further states that this code proposal will “harmonize the special inspection requirements for wood trusses so that they have the same requirements as open web steel joists and joist girders (section 1705.2.3), and cold formed steel trusses (AISI S240, Chapter D.)” For reference please see https://cfsei.memberclicks.net/assets/docs/publications/freestandards/s200-15/aisis240-151stprinting.pdf (https://cfsei.memberclicks.net/assets/docs/publications/freestandards/s200-15/aisis240-151stprinting.pdf).

Chapter D does not reference a 60-inch height limit for truss inspections. The following section contains the framing inspection requirements:

D6.8 Inspection of Cold-Formed Steel Light-Frame Construction

D6.8.1 The component manufacturer’s quality control inspector shall perform inspections of the component assemblies to verify compliance with the details shown on the shop drawings.

D6.8.2 The installer’s quality control inspector shall perform inspections of the field-installed cold-formed steel structural members, component assemblies and connections to verify compliance with the details shown on the installation drawings.

D6.8.3 The quality assurance inspector shall perform verifications and inspections, as applicable, to verify compliance with the construction documents.

D6.8.4 Inspection tasks shall be in accordance with Table D6.8-1.

<table>
<thead>
<tr>
<th>Task</th>
<th>QC</th>
<th>QA</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Perform</td>
<td>Perform</td>
</tr>
<tr>
<td>B</td>
<td>Required*</td>
<td>Perform</td>
</tr>
</tbody>
</table>

* Documentation tasks for quality control should be as defined by the applicable quality control program of the component manufacturer or installer.

Chapter D defines the typical truss manufacturing in-plant and 3rd party quality procedures and define the installers QC and 3rd party quality assurance process. The final inspection is performed as defined in IBC section 110.

Finally, the 60-inch height trigger is completely arbitrary. To put this in perspective, a 20 foot, 5/12 pitch truss with 2’ overhangs would exceed the 60-inch limit. This proposal would require special inspections of nearly all wood truss construction, even accessory maintenance or storage buildings. No specific evidence has been provided to warrant such a change.

IBC section 1705.5.2 as provided in the 2015 building code is sufficient. It properly requires special inspections where trusses are 60 feet or greater in length. These are the trusses where history has shown that there is a need for special inspections. This code section is not broken.
If enforcement is the problem, then that needs to be addressed in a very specific manner that adding this language to the IBC will not do.
Committee Action: Approved as Modified

S145-16
IBC: 1705.11.1.

Proposed Change as Submitted

Proponent: Randy Shackelford (rshackelford@strongtie.com)

2015 International Building Code

Revise as follows:

1705.11.1 Structural wood. Continuous special inspection is required during field gluing operations of elements of the main windforce-resisting system. Periodic special inspection is required for nailing, bolting, anchoring and other fastening of elements of the main windforce-resisting system, including wood shear walls, wood diaphragms, drag struts, braces and hold-downs.

Exception: Special inspections are not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other elements of the main windforce-resisting system, where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center and fasteners are installed in a single row.

Reason: This proposal is meant to clarify when special inspection is required for shearwalls. It is possible that the designer may specify two rows of nails at 6” o.c. in an effort to circumvent the special inspection process.

In addition, there are types of wood-framed shearwalls that are designed to resist both shear and wind uplift, that use nails in multiple rows. Special inspection of shearwalls that resist both shear and uplift from wind are particularly important because this single system is being used as the main wind-force resisting system in both primary directions.

I believe the intent of this section is to require special inspection of any nail spacing at 4” o.c. or less without regard to whether the nails are staggered or not, but this change will make sure it is interpreted that way.

Bibliography: These standards have requirements for nails in multiple rows for shearwalls resisting both shear and wind uplift:

www.awc.org

Wood Frame Construction Manual (ANSI/AWC WFCM-2015), American Wood Council, 2015, Page 130, Figure 3.2f
www.awc.org

Cost Impact: Will increase the cost of construction

It is possible that this proposal could increase the cost of construction if a designer were specifying fasteners in two rows at 6” o.c. in an effort to avoid special inspection. The additional cost would be the cost of special inspection, but this cost would be incurred only if no other triggers for special inspection were met.

This additional cost is justified by the fact that this system is acting as both the lateral force resisting system and the uplift force resisting system.

But I believe the intent of this section is to require the special inspection using the nail spacing regardless of staggering in rows. In this case, there would be no increase in construction.

S145-16:
1705.11.1-
SHACKELFORD13299

Public Hearing Results

Modification:

2015 International Building Code

1705.11.1 Structural wood. Continuous special inspection is required during field gluing operations of elements of the main windforce-resisting system. Periodic special inspection is required for nailing, bolting, anchoring and other fastening of elements of the main windforce-resisting system, including wood shear walls, wood diaphragms, drag struts, braces and hold-downs.

• Exception: Special inspections are not required for wood shear walls, shear panels and diaphragms, including nailing,
bolting, anchoring and other fastening to other elements of the main windforce-resisting system, where the specified fastener spacing of the sheathing at panel edges is more than 4 inches (102 mm) on center and fasteners are installed in a single row.

Committee Reason: By clarifying the exception, this code change helps determine where special inspection of the main windforce-resisting system is required. The modification substitutes more suitable wording to accomplish the intent of the code change.

Assembly Action: None

Public Comment 1:
Proponent: Scott Campbell, representing Portland Cement Association (scampbell@cement.org) requests Approve as Modified by this Public Comment.

Further Modify as Follows:

2015 International Building Code

1705.11.1 Structural wood. Continuous special inspection is required during field gluing operations of elements of the main windforce-resisting system. Periodic special inspection is required for nailing, bolting, anchoring and other fastening of elements of the main windforce-resisting system, including wood shear walls, wood diaphragms, drag struts, braces and hold-downs.

Exception: Special inspections are not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other elements of the main windforce-resisting system, where the specified fastener spacing at panel edges is more than 4 inches (102 mm) on center and fasteners are installed in a single row.

Commenter's Reason: Special inspection of wood sheathing fasteners is important to maintain safety at a level comparable to other building systems. The modification approved by the committee removes a crucial portion of the proposal and would allow for avoidance of special inspection by staggering fasteners without reducing the effective fastener spacing.
Proposed Change as Submitted

Proponent: Bonnie Manley, AISI, representing American Institute of Steel Construction (bmanley@steel.org)

2015 International Building Code
Revise as follows:

1705.12.1.1 Seismic force-resisting systems. Special inspections of structural steel in the seismic force-resisting systems of buildings and structures assigned to Seismic Design Category B, C, D, E or F shall be performed in accordance with the quality assurance requirements of AISC 341.

Exception: Special inspections are not required in the seismic force-resisting systems of buildings and structures assigned to Seismic Design Category B or C that are not specifically detailed for seismic resistance, with a response modification coefficient, $R$, of 3 or less, excluding cantilever column systems.

Exceptions:
1. In buildings and structures assigned to Seismic Design Category B or C, special inspections are not required for structural steel seismic force-resisting systems where the response modification coefficient, $R$, designated for “Steel systems not specifically detailed for seismic resistance, excluding cantilever column systems” in ASCE 7, Table 12.2-1 has been used for design and detailing.
2. In buildings and structures assigned to Seismic Design Category D, E, or F, special inspections are not required for structural steel force-resisting systems where design and detailing in accordance with AISC 360 is permitted by ASCE 7, Table 15.4-1.

1705.12.1.2 Structural steel elements. Special inspections of structural steel elements in the seismic force-resisting systems of buildings and structures assigned to Seismic Design Category B, C, D, E or F other than those covered in Section 1705.12.1.1, including struts, collectors, chords and foundation elements, shall be performed in accordance with the quality assurance requirements of AISC 341.

Exception: Special inspections of structural steel elements are not required in the seismic force-resisting systems of buildings and structures assigned to Seismic Design Category B or C with a response modification coefficient, $R$, of 3 or less.

Exceptions:
1. In buildings and structures assigned to Seismic Design Category B or C, special inspections of structural steel elements are not required for seismic force-resisting systems with a response modification coefficient, $R$, of 3 or less.
2. In buildings and structures assigned to Seismic Design Category D, E, or F, special inspections are not required for seismic force-resisting systems where design and detailing other than AISC 341 is permitted by ASCE 7, Table 15.4-1. Special inspection shall be in accordance with the applicable reference standard listed in ASCE 7, Table 15.4-1.

1705.13.1.1 Seismic force-resisting systems. Nondestructive testing of structural steel in the seismic force-resisting systems of buildings and structures assigned to Seismic Design Category B, C, D, E or F shall be performed in accordance with the quality assurance requirements of AISC 341.

Exception: Nondestructive testing is not required in the seismic force-resisting systems of buildings and structures assigned to Seismic Design Category B or C that are not specifically detailed for seismic resistance, with a response modification coefficient, $R$, of 3 or less, excluding cantilever column systems.

Exceptions:
1. In buildings and structures assigned to Seismic Design Category B or C, nondestructive testing is not required for structural steel seismic force-resisting systems where the response modification coefficient, $R$, designated for “Steel systems not specifically detailed for seismic resistance, excluding cantilever column systems” in ASCE 7, Table 12.2-1 has been used for design and detailing.
2. In buildings and structures assigned to Seismic Design Category D, E, or F, nondestructive testing is not required for structural steel force-resisting systems where design and detailing in accordance with AISC 360 is permitted by ASCE 7, Table 15.4-1.

1705.13.1.2 Structural steel elements. Nondestructive testing of structural steel elements in the seismic force-resisting systems of buildings and structures assigned to Seismic Design Category B, C, D, E or F other than those covered in Section 1705.13.1.1, including struts, collectors, chords and foundation elements, shall be performed in accordance with the quality assurance requirements of AISC 341.

Exception: Nondestructive testing of structural steel elements is not required in the seismic force-resisting systems of buildings and structures assigned to Seismic Design Category B or C with a response modification coefficient, $R$, of 3 or
Exceptions:

1. In buildings and structures assigned to Seismic Design Category B or C, nondestructive testing of structural steel elements is not required for seismic force-resisting systems with a response modification coefficient, R, of 3 or less.

2. In buildings and structures assigned to Seismic Design Category D, E, or F, nondestructive testing of structural steel elements is not required for seismic force-resisting systems where design and detailing other than AISC 341 is permitted by ASCE 7, Table 15.4-1. Nondestructive testing of structural steel elements shall be in accordance with the applicable reference standard listed in ASCE 7, Table 15.4-1.

Reason: This proposal provides a needed clarification in the exceptions in these sections on special inspection and nondestructive testing for structural steel seismic force-resisting systems and for structural steel elements in other types of seismic force-resisting systems.

In buildings and structures assigned to SDC D, E or F, IBC Section 2205.2.1.2 recognizes a few structural steel seismic force-resisting systems in ASCE 7, Table 15.4-1 where detailing in accordance with AISC 360 is permitted in lieu of AISC 341. For these particular systems, it would be almost impossible to conduct special inspections and nondestructive testing in accordance with AISC 341 when they have not been detailed in accordance with AISC 341. The new second exception in Sections 1705.12.1.1 and 1705.13.1.1 recognizes this by permitting special inspection and nondestructive testing in accordance with AISC 360. Modifications to the first exception in both sections are simply editorially fixes of the existing exception so that it matches Section 2205.2.1.

In buildings and structures assigned to SDC D, E or F, IBC Section 2205.2.2 recognizes structural steel elements in seismic force-resisting systems in ASCE 7, Table 15.4-1 where detailing in accordance with AISC 341 is not required. For these particular systems, it would be almost impossible to conduct special inspections and nondestructive testing in accordance with AISC 341 when they have not been detailed in accordance with AISC 341. The new second exception in Sections 1705.12.1.2 and 1705.13.1.2 recognizes this by permitting special inspection and nondestructive testing in accordance with the applicable standard. Modifications to the first exception in both sections are simply editorially fixes of the existing exception so that it matches Section 2205.2.2.

Cost Impact: Will not increase the cost of construction
This proposal is intended to be a clarification of the provisions. No increase in the cost of construction is anticipated.

S146-16 :
1705.12.1.1-1705.13.1.1
MANLEY12857

Public Hearing Results

Committee Action: Approved as Modified

Modification:

2015 International Building Code

1705.12.1.2 Structural steel elements. Special inspections of structural steel elements in the seismic force-resisting systems of buildings and structures assigned to Seismic Design Category B, C, D, E or F other than those covered in Section 1705.12.1.1, including struts, collectors, chords and foundation elements, shall be performed in accordance with the quality assurance requirements of AISC 341.

Exceptions:

1. In buildings and structures assigned to Seismic Design Category B or C, special inspections of structural steel elements are not required for seismic force-resisting systems with a response modification coefficient, R, of 3 or less.

2. In buildings and structures assigned to Seismic Design Category D, E, or F, special inspections of structural steel elements are not required for seismic force-resisting systems where design and detailing other than AISC 341 is permitted by ASCE 7, Table 15.4-1. Special inspection shall be in accordance with the applicable reference standard listed in ASCE 7, Table 15.4-1.

1705.13.1.2 Structural steel elements. Nondestructive testing of structural steel elements in the seismic force-resisting systems of buildings and structures assigned to Seismic Design Category B, C, D, E or F other than those covered in Section 1705.13.1.1, including struts, collectors, chords and foundation elements, shall be performed in accordance with the quality assurance requirements of AISC 341.

Exceptions:
1. In buildings and structures assigned to Seismic Design Category B or C, nondestructive testing of structural steel elements is not required for seismic force-resisting systems with a response modification coefficient, R, of 3 or less.
2. In buildings and structures assigned to Seismic Design Category D, E, or F, nondestructive testing of structural steel elements is not required for seismic force-resisting systems where design and detailing other than AISC 341 is permitted by ASCE 7, Table 15.4-1. Nondestructive testing of structural steel elements shall be in accordance with the applicable reference standard listed in ASCE 7, Table 15.4-1.

Committee Reason: This proposal clarifies the special inspection of steel elements that resist seismic forces. The modification makes editorial corrections to the proposed wording.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Bonnie Manley, AISI, representing American Institute of Steel Construction (bmanley@steel.org) requests Approve as Modified by this Public Comment.

Further Modify as Follows:

2015 International Building Code

1705.12.1.1 Seismic force-resisting systems. Special inspections of structural steel seismic force-resisting systems in buildings and structures assigned to Seismic Design Category B, C, D, E or F shall be performed in accordance with the quality assurance requirements of AISC 341.

Exceptions:

1. In buildings and structures assigned to Seismic Design Category B or C, special inspections are not required for structural steel seismic force-resisting systems where the response modification coefficient, R, designated for “Steel systems not specifically detailed for seismic resistance, excluding cantilever column systems” in ASCE 7, Table 12.2-1 has been used for design and detailing.
2. In buildings and structures assigned to Seismic Design Category D, E, or F, special inspections are not required for structural steel seismic force-resisting systems where design and detailing in accordance with AISC 360 is permitted by ASCE 7, Table 15.4-1.

1705.12.1.2 Structural steel elements. Special inspections of structural steel elements in the seismic force-resisting systems of buildings and structures assigned to Seismic Design Category B, C, D, E or F other than those covered in Section 1705.12.1.1, including struts, collectors, chords and foundation elements, shall be performed in accordance with the quality assurance requirements of AISC 341.

Exceptions:

1. In buildings and structures assigned to Seismic Design Category B or C, special inspections of structural steel elements are not required for seismic force-resisting systems with a response modification coefficient, R, of 3 or less.
2. In buildings and structures assigned to Seismic Design Category D, E, or F, special inspections of structural steel elements are not required for seismic force-resisting systems where design and detailing other than AISC 341 is permitted by ASCE 7, Table 15.4-1. Special inspection shall be in accordance with the applicable reference standard listed in ASCE 7, Table 15.4-1.

1705.13.1.1 Seismic force-resisting systems. Nondestructive testing of structural steel seismic force-resisting systems in buildings and structures assigned to Seismic Design Category B, C, D, E or F shall be performed in accordance with the quality assurance requirements of AISC 341.

Exceptions:

1. In buildings and structures assigned to Seismic Design Category B or C, nondestructive testing is not required for structural steel seismic force-resisting systems where the response modification coefficient, R, designated for “Steel systems not specifically detailed for seismic resistance, excluding cantilever column systems” in ASCE 7, Table 12.2-1 has been used for design and detailing.
2. In buildings and structures assigned to Seismic Design Category D, E, or F, nondestructive testing is not required for structural steel seismic force-resisting systems where design and detailing in accordance with AISC 360 is permitted by ASCE 7, Table 15.4-1.

1705.13.1.2 Structural steel elements. Nondestructive testing of structural steel elements in the seismic force-resisting systems of buildings and structures assigned to Seismic Design Category B, C, D, E or F other than those covered in Section 1705.13.1.1, including struts, collectors, chords and foundation elements, shall be performed in accordance with the quality assurance requirements of AISC 341.
assurance requirements of AISC 341.

Exceptions:

1. In buildings and structures assigned to Seismic Design Category B or C, nondestructive testing of *structural steel elements* is not required for seismic force-resisting systems with a response modification coefficient, R, of 3 or less.

2. In buildings and structures assigned to Seismic Design Category D, E, or F, nondestructive testing of *structural steel elements* is not required for seismic force-resisting systems where design and detailing other than AISC 341 is permitted by ASCE 7, Table 15.4-1. Nondestructive testing of *structural steel elements* shall be in accordance with the applicable reference standard listed in ASCE 7, Table 15.4-1.

**Commenter’s Reason:** These editorial modifications clean up the charging language to ASCE 7, Section 15.4, which governs nonbuilding structures. Use of the term "building" in these exceptions could prove to be confusing and is therefore recommended for deletion.
Proposed Change as Submitted

Proponent: John Gillengerten (johng5155@live.com); Henry Green, John D. Gillengerten, representing National Institute of Building Sciences Building Seismic Safety Council Code Resource Support Committee

2015 International Building Code

Revise as follows:

1705.12.6 Plumbing, mechanical and electrical components: Periodic special inspection of plumbing, mechanical and electrical components shall be required for the following:

1. Anchorage of electrical equipment for emergency and standby power systems in structures assigned to Seismic Design Category C, D, E or F.
2. Anchorage of other electrical equipment in structures assigned to Seismic Design Category E or F.
3. Installation and anchorage of piping systems designed to carry hazardous materials and their associated mechanical units in structures assigned to Seismic Design Category C, D, E or F.
4. Installation and anchorage of ductwork designed to carry hazardous materials in structures assigned to Seismic Design Category C, D, E or F.
5. Installation and anchorage of vibration isolation systems in structures assigned to Seismic Design Category C, D, E or F where the approved construction documents require a nominal clearance of 1/4 inch (6.4 mm) or less between the equipment support frame and restraint.
6. Installation of mechanical and electrical equipment including duct work, piping systems and their structural supports where automatic fire sprinkler systems are installed in structures assigned to Seismic Design Category C, D, E or F to verify either of the following:
   6.1 Minimum clearances have been provided as required by Section 13.2.3 ASCE/SEI 7; or
   6.2 That a nominal clearance of at least 3 inches (76 mm) has been be provided between fire protection sprinkler system drops and sprigs and structural members not used collectively or independently to support the sprinklers, or from equipment attached to the building structure, or from other systems' piping.

Where flexible sprinkler hose fittings are used, special inspection of minimum clearances is not required.

Reason: Experience in recent earthquakes has shown that pounding between sprinkler piping drops and sprigs and adjacent nonstructural components such as pipes and ducts has resulted in pipe connection failures and accidental activation, which resulted in flooding and potentially compromising the operability of the system should fire following earthquake occur.

ASCE/SEI 7-16 identifies fire protection sprinkler systems as components that are required to function for life-safety purposes after an earthquake, classifying them as a Designated Seismic System. Section 13.2.3 ASCE/SEI 7-16 requires that interaction between Designated Seismic Systems and adjacent components be avoided. The intent is described in Section C13.2.3 of the ASCE/SEI 7-16 commentary, which states in part:

... It is the intent of the standard that the seismic displacements considered include both relative displacement between multiple points of support (addressed in Section 13.3.2) and, for mechanical and electrical components, displacement within the component assemblies. Impact of components must be avoided, unless the components are fabricated of ductile materials that have been shown to be capable of accommodating the expected impact loads. ...

It further cites specific examples using fire protection sprinkler systems to illustrate the types of interactions to be avoided.

... Consequential damage may occur because of displacement of components and systems between support points. For example, in older suspended ceiling installations, excessive lateral displacement of a ceiling system may fracture sprinkler heads that project through the ceiling. A similar situation may arise if sprinkler heads projecting from a small-diameter branch line pass through a rigid ceiling system. Although the branch line may be properly restrained, it may still displace sufficiently between lateral support points to affect other components or systems. ...

Maintaining adequate clearances is critical to good seismic performance of fire protection sprinkler systems, and Section 13.2.3 ASCE/SEI 7-16 requires that interaction between Designated Seismic Systems and adjacent components be avoided.

This proposal provides periodic special inspection to verify that adequate clearance is provided between sprinkler drops and sprigs and adjacent structural and nonstructural components. In some cases, an evaluation of the required clearance to avoid interaction is not provided by the registered design professionals. In such cases, a nominal 3 inch clearance from adjacent items is permitted, which is the same as the NFPA13 clearance requirement from structural members to pounding. Due to their inherent flexibility, clearance between listed flexible sprinkler hose fittings and other components, equipment, or structural members is not required.

Cost Impact: Will increase the cost of construction
This change might have a very minor impact on the cost of installation of electrical, mechanical and plumbing installations and their inspection.

**Public Hearing Results**

**Committee Action:** Approved as Submitted

**Committee Reason:** This code change will require special inspection for elements that are problems and are capable of taking out a building for many months if they fail.

**Assembly Action:** None

**Public Comment 1:**

**Proponent:** John Gillengerten, representing National Institute of Building Sciences Building Seismic Safety Council Code Resource Support Committee (johng5155@live.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

1705.12.6 Plumbing, mechanical and electrical components Periodic special inspection of plumbing, mechanical and electrical components shall be required for the following:

1. Anchorage of electrical equipment for emergency and standby power systems in structures assigned to Seismic Design Category C, D, E or F.
2. Anchorage of other electrical equipment in structures assigned to Seismic Design Category E or F.
3. Installation and anchorage of piping systems designed to carry hazardous materials and their associated mechanical units in structures assigned to Seismic Design Category C, D, E or F.
4. Installation and anchorage of ductwork designed to carry hazardous materials in structures assigned to Seismic Design Category C, D, E or F.
5. Installation and anchorage of vibration isolation systems in structures assigned to Seismic Design Category C, D, E or F where the approved construction documents require a nominal clearance of $\frac{1}{4}$ inch (6.4 mm) or less between the equipment support frame and restraint.
6. Installation of mechanical and electrical equipment including duct work, piping systems and their structural supports where automatic fire sprinkler systems are installed in structures assigned to Seismic Design Category C, D E or F to verify either of the following:
   6.1. Minimum clearances have been provided as required by Section 13.2.3 ASCE/SEI 7-16; or
   6.2. That a nominal clearance of at least 3 inches (76 mm) has been provided between fire protection sprinkler system drops and sprigs and structural members not used collectively or independently to support the sprinklers, or from equipment attached to the building structure, or from other systems' piping. Where flexible sprinkler hose fittings are used for sprinkler drops, special inspection of minimum clearances is not required.

Where flexible sprinkler hose fittings are used, special inspection of minimum clearances is not required.

**Commenter's Reason:** This editorial change was requested by a member of the Group B Hearings Structural Committee in Louisville. Flexible hoses are only used for sprinkler drops.

**Proponent:** Ed Berkel, representing ICC Code Correlation Committee (ccc@iccsafe.org) requests Disapprove.

**Commenter's Reason:** The Code Correlation Committee requests Disapproval of this code change proposal in order to bring a correlation issue to the attention of the full membership at the Public Comment Hearings and to allow the membership to coordinate action on this code change proposal with action taken on Code Change Proposal ADM94-16.

ADM94-16 is the administrative update to referenced standards in the I-Codes. One of these standards, ASCE7, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, was proposed for update to ASCE7-16. However, a successful assembly motion requests that the referenced ASCE7 remain at ASCE7-10 as it presently is referenced in the 2015 I-Codes. This code change proposal coordinates with and relies upon reference to ASCE7-16.
The Code Correlation Committee is a standing committee of the International Code Council whose objectives, procedures and organization are set forth in Council Policy CP#44-13. The objective of the Code Correlation Committee is to maintain technical and editorial consistency among the International Codes and to assist staff in the evaluation and processing of code change proposals and comments that are exclusively editorial.
1708.2.1 Within 24 hours after removal of the test load, the structure shall have recovered not less than 75 percent of the
During and immediately after the test, the structure shall not show evidence of failure
Under the design load, the deflection shall not exceed the limitations specified in Section 1604.3.
remain stable
1708.2.2 Whenever there is a reasonable doubt as to the stability or load-bearing capacity of a completed building,
structure or portion thereof for the expected loads, an engineering assessment shall be required. The engineering assessment
shall involve either a structural analysis or an in-situ load test, or both. The structural analysis shall be based on actual material
properties and other as-built conditions that affect stability or load-bearing capacity, and shall be conducted in accordance with
the applicable design standard. If the structural assessment determines that the load bearing capacity is less than that required
by the code, the in-situ load tests shall be conducted in accordance with Section 1708.2. If the building, structure or portion
thereof is found to have inadequate stability or load-bearing capacity for the expected loads, modifications to ensure structural
adequacy or the removal of the inadequate construction shall be required.
1708.3 In-situ load tests. In-situ load tests shall be conducted in accordance with Section 1708.2.1 or 1708.2.2 and shall be supervised by a registered design professional. The test shall simulate the applicable loading conditions specified in Chapter 16 as necessary to address the concerns regarding structural stability of the building, structure or portion thereof.
Delete without substitution:
1708.2 Test standards. Structural components and assemblies shall be tested in accordance with the appropriate referenced
standards. In the absence of a standard that contains an applicable load test procedure, the test procedure shall be developed
by a registered design professional and approved. The test procedure shall simulate loads and conditions of application that
the completed structure or portion thereof will be subjected to in normal use.
Revise as follows:
1708.3.1 Load test procedure specified. Where a referenced material standard contains an applicable load test
procedure and acceptance criteria, the test procedure and acceptance criteria in the standard shall apply. In the absence of
specific load factors or acceptance criteria, the load factors and acceptance criteria in Section 1708.2.2 shall apply.
1708.3.2 Load test procedure not specified. In the absence of applicable load test procedures contained within a
material standard referenced by this code or acceptance criteria for a specific material or method of construction, such existing
structure shall be subjected to a test procedure developed by a registered design professional that simulates
applicable loading and deformation conditions. For components that are not a part of the seismic force-resisting system, at a
minimum the test load shall be equal to the specified factored design loads. For materials such as wood that have strengths
that are dependent on load duration, the test load shall be adjusted to account for the difference in load duration of the test
compared to the expected duration of the design loads being considered. For statically loaded components, the test load shall
be left in place for a period of 24 hours. For components that carry dynamic loads (e.g., machine supports or fall arrest
anchors), the load shall be left in place for a period consistent with the component's actual function. The structure shall be
considered to have successfully met the test requirements where the following criteria are satisfied:
1. Under the design load, the deflection shall not exceed the limitations specified in Section 1604.3.
2. Within 24 hours after removal of the test load, the structure shall have recovered not less than 75 percent of the
maximum deflection.
3. During and immediately after the test, the structure shall not show evidence of failure remain stable.
Reason: This is an editorial tune-up of the in-situ load tests.
The first change deletes a superfluous phrase.
The second change (deletion of 1708.2) is intended to eliminate a duplicative provision. The requirements in 1708.2 are
covered in much greater depth and with better specificity in 1708.3, 1708.3.1, and 1708.3.2. In addition, the reference in
Section 1708.2 to "in normal use" is unclear. Section 1708.3 is much more specific with respect to what loads must be
simulated (i.e., those in Chapter 16).
The third change (addition of the word "material" in two locations) is intended to clarify that while loads come from Chapter 16,
the load test procedure must come from the relevant material standard (e.g., AISC or ACI).
The fourth change (deletion of "not show evidence of failure") is needed because the requirement is not clear. The term
"failure" is not defined, and can be interpreted a number of ways. In some cases, for example, even minor cracking of concrete
has been considered "failure" by misinformed parties. The modified acceptance criteria would require: that the deflection under
the design load not exceed the limits of 1604.3, that the structure recovers at least 75 percent of the maximum deflection after
removal of load, and that the structure remains stable.
This last requirement is more clear than requiring a structure "not show evidence of failure".

**Cost Impact:** Will not increase the cost of construction
As an editorial change, this proposal is intended to clarify and make the load test requirements more concise. It should have no measurable impact on the cost of construction.

**Public Hearing Results**

**Committee Action:** Disapproved
**Committee Reason:** This code change was disapproved because the wording proposed in item 3 of Section 1708.2.2 is unclear.

**Individual Consideration Agenda**

**Public Comment 1:**

**Proponent:** Gwenyth Searer, representing self requests Approve as Modified by this Public Comment.

**Modify as Follows:**

**2015 International Building Code**

**1708.2.2 Load test procedure not specified.** In the absence of applicable load test procedures contained within a material standard referenced by this code or acceptance criteria for a specific material or method of construction, such existing structure shall be subjected to an approved test procedure developed by a registered design professional that simulates applicable loading and deformation conditions. For components that are not a part of the seismic force-resisting system, at a minimum the test load shall be equal to the specified factored design loads. For materials such as wood that have strengths that are dependent on load duration, the test load shall be adjusted to account for the difference in load duration of the test compared to the expected duration of the design loads being considered. For statically loaded components, the test load shall be left in place for a period of 24 hours. For components that carry dynamic loads (e.g., machine supports or fall arrest anchors), the load shall be left in place for a period consistent with the component's actual function. The structure shall be considered to have successfully met the test requirements where the following criteria are satisfied:

1. Under the design load, the deflection shall not exceed the limitations specified in Section 1604.3.
2. Within 24 hours after removal of the test load, the structure shall have recovered not less than 75 percent of the maximum deflection.
3. During and immediately after the test, the structure shall remain stable not show evidence of failure.

**Commenter's Reason:** The original code change proposal was an editorial clean-up of the in-situ load test requirements in Section 1708 of the IBC.

The ICC Structural Committee liked the proposal but took exception to the proposed replacement of the words "not show evidence of failure" with "remain stable" in Item 3 of Section 1708.2.2. This is documented in the Committee Report for S149-16 Reason for Disapproval: "This code change was disapproved because the wording proposed in Item 3 of Section 1708.2.2 is unclear".

This public comment strikes the replacement in response to the Committee's concerns. The rest of the proposal cleans up various conflicts and unclear requirements in Section 1708. If this public comment is accepted, the rest of the proposal will be adopted, but the wording in Item 3 of Section 1708.2.2 will remain unchanged.
Proposed Change as Submitted

Proponent: T. Eric Stafford, PE, representing Institute for Business and Home Safety

2015 International Building Code

Revise as follows:

1709.5.2 Exterior windows and door assemblies not provided for in Section 1709.5.1. Exterior window and door assemblies shall be tested in accordance with ASTM E 330. Structural performance of garage doors and rolling 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 10 seconds at a load equal to 1.5 times the design pressure.

Add new text as follows:

1709.5.3 Garage doors and rolling doors. Garage doors and rolling doors shall be tested in accordance with either ASTM E 330 or ANSI/DASMA 108, and shall meet the acceptance criteria of ANSI/DASMA 108. Garage doors and rolling doors shall be labeled with a permanent label identifying the door manufacturer, the door model/series number, the positive and negative design wind pressure rating, the installation instruction drawing reference number, and the applicable test standard.

Reason: This proposal is one of several that are addressing labeling of critical components of the building envelope. The primary purpose of this code change is to require that garage doors have a permanent label that provides a way for building owners, homeowners, and others to be able to determine their performance characteristics after the building has been occupied. The 2015 IBC does not require any type of label for garage doors. For products that don't have permanent labels, it becomes nearly impossible for the owner to determine the structural wind load resistance and/or energy efficiency of the garage doors after they've occupied the building. This proposal would simply require some type of permanent marking on the garage door indicating the manufacturer and model/series number, and basic performance characteristics so that the specific performance characteristics could be retrieved at a later date.

For the past 10-15 years, there has been a push towards considering sustainability in the way our buildings are constructed in this country. As a result, building owners and occupants increasingly want more information about the sustainability of the buildings they occupy. Consequently, they need to be provided with ways to determine how critical components are expected to perform in the buildings they use. Garage doors are important components of the building envelope and their performance is critical in preventing wind and water infiltration as well as to maintaining the overall structural integrity of the building.

Some manufacturers already include permanent labels on their products that provide traceability to the manufacture and the product characteristics. The Florida Building Code has required this type of label since the 2007 edition and has continued to require it in subsequent editions. The following is the relevant text from the 5th Edition (2014) Florida Building Code, Building:

1710.5.2.1.1 Garage door labeling. Garage doors shall be labeled with a permanent label provided by the garage door manufacturer. The label shall identify the garage door manufacturer, the garage door model/series number, the positive and negative design pressure rating; indicate impact rated if applicable; the installation instruction drawing reference number; the Florida product approval or Miami-Dade product approval number if applicable; and the applicable test standards. The required garage door components for an approved garage door assembly may be indicated using a checklist form on the label. If a checklist format is used on the label, the door installer or the garage door manufacturer shall mark the selected components on the checklist that are required to assemble an approved garage door system. The installation instructions shall be provided and available on the job site.

Approval of this proposal will assure going forward that new or replaced doors will be labeled such that building owners and those considering the purchase of buildings with these products will be able to obtain information necessary for determining the expected performance of these critical components of the building envelope.

Cost Impact: Will increase the cost of construction

Will impact cost on some manufacturers. The code does not currently require a permanent label. However, some garage door manufacturers voluntarily apply a permanent label that identifies the critical performance characteristics. There will be no cost impact to those manufacturers.
Public Hearing Results

Committee Action: Disapproved
Committee Reason: Incomplete explanation of why the proposed labeling should be required. The explanation does not address why the specific performance characteristics of the garage door need to be retrieved at a later date and how this will increase safety, considering there will be increased costs across the country but not everyone will benefit from it. Requiring installation instruction drawings is considered superfluous. Should address whether similar labeling may already be required elsewhere. The attempt to separate garage doors from other assemblies has merit.

Assembly Action: None

Individual Consideration Agenda

Proponent: T. Eric Stafford, PE, representing Institute for Business and Home Safety; Joseph Hetzel (Jhetzel@thomasamc.com) requests Approve as Submitted.

Commenter's Reason: The structural committee's reason for disapproval indicates that the explanation does not address why specific performance characteristics need to be retrieved at a later date. However, our reason statement clearly states that the primary purpose for requiring permanent labels on garage doors that indicate specific performance characteristics is so that building owners can know the expected performance of critical components of the building envelope in preventing wind and water infiltration in addition to maintaining the overall structural integrity of the building. The move towards sustainable buildings requires knowledge of certain critical components. While building owners of new construction can be more certain of the performance characteristics of their building, this is not the case for existing buildings.

It is also worth noting that an opponent to this proposal incorrectly stated at the committee hearings that DASMA 108, which is referenced for garage doors, already requires a permanent label. DASMA 108 primarily focuses on testing and acceptance criteria for wind load resistance and does not address labeling. This proposal would not conflict with DASMA 108.

IBHS worked directly with the garage door industry association (DASMA) to craft language for the IBC and IRC. Most of the language regarding identification on the label was provided by DASMA. While a representative from DASMA was not available at the structural committee meeting to speak on this proposal, a representative did provide testimony in support of the companion change (RB257) to the IRC.
IBC: 1709.5.3 (New), 1709.5.3.1 (New), 202 (New).

Proposed Change as Submitted

Proponent: T. Eric Stafford, PE, representing Institute for Business and Home Safety

2015 International Building Code
Add new definition as follows:

SECTION 202 DEFINITIONS

IMPACT PROTECTIVE SYSTEM. Construction that has been shown by testing to withstand the impact of test missiles and that is applied, attached, or locked over exterior glazing.

Add new text as follows:

1709.5.3 Wind-borne debris protection. Protection of exterior glazed openings in buildings located in windborne debris regions shall be in accordance with Section 1609.1.2.

1709.5.3.1 Impact protective systems testing and labeling. Impact protective systems shall be tested for impact resistance by an approved independent laboratory for compliance with ASTM E 1886 and ASTM E 1996. Impact protective systems shall also be tested for design wind pressure by an approved independent laboratory for compliance with ASTM E 330. Required design wind pressures shall be determined in accordance with Section 1609.6 or ASCE 7, and for the purposes of this section, are permitted to be multiplied by 0.6. Impact protective systems shall bear a label identifying the manufacturer, performance characteristics, and approved inspection agency. Impact protective systems shall have a permanent label applied in accordance with Section 1703.5.4 that provides traceability to the manufacturer, product designation, and performance characteristics.

Reason: This proposal is one of several that are addressing labeling of critical components of the building envelope. The primary purpose of this code change is to require that impact protective systems (hurricane shutters) have a permanent label that provides a way for building owners, homeowners, and others to be able to determine their performance characteristics after the building has been occupied. The 2015 IBC does not require any type of label for impact protective systems. For products that don't have permanent labels, it becomes nearly impossible for the owner to determine the structural wind load resistance and impact resistance of the products after they've occupied the building. This proposal would simply require some type of permanent marking on the impact protective system indicating the manufacturer and model/series number, that provides traceability so specific performance characteristics can be retrieved at a later date. While the permanent label would only need to provide traceability to the product, it could provide all the required information. If the relevant information is not provided on a permanent label, a temporary removable label is required to be applied so that local code officials can verify that the appropriate impact protective system was provided.

For the past 10-15 years, there has been a push towards considering sustainability in the way our buildings are constructed in this country. If this goal is to be successful and building owners and occupants increasingly want more information about the sustainability of the buildings they occupy, they need to be provided ways to be able to determine how critical components are expected to perform in the buildings they use. Impact protective systems are important components of the building envelope and their performance is critical to maintaining the overall structural integrity of the building.

Some manufacturers already include permanent labels on their products that provide traceability to the manufacture and the product characteristics. The Florida Building Code has required a permanent label since the 2007 edition and has continued to require it in subsequent editions. The following is the relevant text from the 5th Edition (2014) Florida Building Code, Building:

1710.8 Impact resistant coverings.

1710.8.1 Labels. A permanent label shall be provided by the product approval holder on all impact-resistant coverings.

1710.8.2 The following information shall be included on the labels on impact-resistant coverings:

1. Product approval holder name and address.
2. All applicable methods of approval. Methods of approval include, but are not limited to Miami-Dade NOA; Florida Building Commission, TDI Product Evaluation; ICC-ES.
3. The test standard or standards specified in Section 1609.1.2, including standards referenced within the test standards specified in Section 1609.1.2 used to demonstrate code compliance.
4. For products with a Florida product approval number or a Miami-Dade County Building and Neighborhood Compliance Department Notice of Acceptance Number (NOA), such numbers shall be included on the label.

This proposal also provides some additional clarification for impact protective systems that is lacking in the IBC. New Section 1709.5.3.1 clarifies that impact protective systems also have to be capable of resisting the required design wind pressure as well as the impact criteria. New language is added to clarify the relationship between design wind loads calculated in...
accordance with ASCE 7-10 and the wind load testing requirements of ASTM E 330. Impact protective systems are important components of the building envelope and their performance is critical to maintaining the overall structural integrity of the building. Approval of this proposal will assure, going forward, that new or replaced impact protective systems will be labeled such that building owners and those considering the purchase of buildings with these products will be able to obtain information necessary for determining the expected performance of these critical components used to protect the building envelope in hurricane prone areas.

Cost Impact: Will increase the cost of construction
Will result in an increase in cost. A consultant representing the industry estimates the cost of providing labels on impact resistant covering products to be as follows:

a. Water Resistant Self-adhering Permanent Labels approximately $0.15 per label. Such labels would most likely be used on Accordion, Roll, Bahama, and Colonial style shutters.
b. Embossed or ink jet labels used on metal and plastic panels would cost approximately $0.05 per label.

Public Comment 1:

Proponent: T. Eric Stafford, PE, representing Institute for Business and Home Safety requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

1709.5.3 Impact protective systems testing and labeling. Impact protective systems shall be tested for impact resistance by an approved independent laboratory for compliance with ASTM E 1886 and ASTM E 1996. Impact protective systems shall also be tested for design wind pressure by an approved independent laboratory for compliance with ASTM E 330. Required design wind pressures shall be determined in accordance with Section 1609.6 or ASCE 7, and for the purposes of this section, are shall be permitted to be multiplied, converted to allowable stress design by multiplying by 0.6. Impact protective systems shall bear a label identifying the manufacturer, performance characteristics, and approved inspection agency. Impact protective systems shall have a permanent label applied in accordance with Section 1703.5.4 that provides traceability to the manufacturer, product designation, and performance characteristics.

Commenter's Reason: We primarily stand on our original supporting statement with regard to reason for this particular code change. The modification simply adds a clarification to language that permits design pressures to be converted to allowable stress design level pressures when these products are tested for design wind loads. This was one of the reasons given by the structural committee for disapproval. This modification is consistent the modification the structural committee approved on S150 regarding the same conversion for windows and doors, which also applies to impact protective systems such as shutters. The proposal does not duplicate what is already required by Chapter 16 as indicated in the reason for disapproval. It provides direction and clarity for testing in accordance with ASTM E 1886 and ASTM E 1996 similar to the specifics that currently exist in the code for testing windows and doors.

These were the primary reasons given by the structural committee for disapproval. However, the primary goal of the proposal is to require that impact protective systems have a permanent label or marking. This proposal is supported by the International Hurricane Protection Association which is the primary industry association for impact protective system manufacturers.
It's also worth noting that the IRC-Building committee approved RB259 which is the companion to the IRC. As published in the ROCAH, the committee's reason for approving the proposal was given as follows:

**RB 258 Committee Reason:** This is a needed change because it is difficult to identify whether a hurricane shutter or impact protective system meets the code specified requirements. Requiring a permanent label will alleviate this problem.

We urge support of this proposal as modified by this public comment.

**Proponent:** Thomas Johnston, representing International Hurricane Protection Association (tom.johnston@abcsupply.com) requests Approve as Submitted.

**Commenter's Reason:** I move full support representing the International Hurricane Protection Association. Impact Protective Devices (IPD's) generally carry nearly identical appearances and clear labeling ensures proper identification by all interested parties from building officials, insurance mitigation specialists as well as dwelling owners. The vast majority of manufacturers of IPD's already have this process in place so there is virtually no additional cost to industry.
Proposed Change as Submitted

**Proponent:** Edwin Huston, representing National Council of Structural Engineers' Associations (NCSEA)  
(huston@smithhustoninc.com)

**2015 International Building Code**

**Revise as follows:**

**1803.5.7 Excavation near foundations.** Where excavation will reduce support from any foundation, a registered design professional shall prepare an *preconstruction* assessment of the structure as determined from examination of the structure, the review of available design documents and, if necessary, excavation of test pits. The *registered design professional* shall determine the requirements for underpinning and protection and prepare site-specific plans, details and sequence of work for submission. Such support shall be provided by underpinning, sheeting and bracing, or by other means acceptable to the building official.

**Reason:** Support of soil below foundations must be determined prior to performing construction work. If the construction excavation exposes an existing footing, the foundations could be undermined. This provision will likely increase the speed of construction because the assessment portion will be required prior to construction and will not be required during the construction work.

**Cost Impact:** Will not increase the cost of construction
Most current practice currently follows this intent, even though it is not clearly stated in the code. The cost of construction will not increase by specifying the timing of the assessment.

Public Hearing Results

**Committee Action:** Disapproved

**Committee Reason:** This proposal would require a "preconstruction" assessment, but that may be too limiting as sometimes it should be "pre-permit". Also sometimes you can't assess a foundation "preconstruction".

**Individual Consideration Agenda**

**Proponent:** Edwin Huston, representing National Council of Structural Engineers' Associations (NCSEA)  
(huston@smithhustoninc.com) requests Approve as Submitted.

**Commenter's Reason:**
Many underpinning failures involve lack of knowledge of the adjacent foundations. Due to numerous failures of adjacent buildings during excavations, preconstruction assessments are nesessary before filing underpinning construction documents.

The Preconstruction Assessment involves an investigation of the adjacent structure using existing drawings of foundations when available, geotechnical studies of the proposed site and the use of test pits to determine depths of existing adjacent footings.

The typical field investigation begins by walking through the adjacent structure's lowest floor and reviewing existing drawings of adjacent foundations when available. The design professional may then estimate the depth and extent of any cellars, columns with footings and foundation walls. Test pits may then be shown on a drawing and excavated by the Contractor to determine the exact bottom of footing elevation below grade. The bottom of footing for the adjacent structure is then included on the construction drawings for the underpinning.
S166-16
IBC: 1803.12, 1809.13, 1810.3.11.2, 1810.3.12, 1810.3.6.1, 1810.3.9.4.

Proposed Change as Submitted

Proponent: Jennifer Goupil, AMERICAN SOCIETY OF CIVIL ENGINEERS, representing SELF (jgoupil@asce.org)

2015 International Building Code
Revise as follows:

1803.5.12 Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E or F, the geotechnical investigation required by Section 1803.5.11 shall also include all of the following as applicable:

1. The determination of dynamic seismic lateral earth pressures on foundation walls and retaining walls supporting more than 6 feet (1.83 m) of backfill height due to design earthquake ground motions.
2. The potential for liquefaction and soil strength loss evaluated for site peak ground acceleration, earthquake magnitude and source characteristics consistent with the maximum considered earthquake ground motions. Peak ground acceleration shall be determined based on one of the following:
   2.1 A site-specific study in accordance with Section 21.5 Chapter 21 of ASCE 7.
   2.2 In accordance with Section 11.8.3 of ASCE 7.
3. An assessment of potential consequences of liquefaction and soil strength loss including, but not limited to, the following:
   3.1 Estimation of total and differential settlement.
   3.2 Lateral soil movement.
   3.3 Lateral soil loads on foundations.
   3.4 Reduction in foundation soil-bearing capacity and lateral soil reaction.
   3.5 Soil downdrag and reduction in axial and lateral soil reaction for pile foundations.
   3.6 Increases in soil lateral pressures on retaining walls.
   3.7 Flotation of buried structures.
4. Discussion of mitigation measures such as, but not limited to, the following:
   4.1 Selection of appropriate foundation type and depths.
   4.2 Selection of appropriate structural systems to accommodate anticipated displacements and forces.
   4.3 Ground stabilization.
   4.4 Any combination of these measures and how they shall be considered in the design of the structure.

1809.13 Footing seismic ties. Where a structure is assigned to Seismic Design Category D, E or F, individual spread footings founded on soil defined in Section 1613.3.2 Chapter 20 of ASCE 7 as Site Class E or F shall be interconnected by ties. Unless it is demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade, ties shall be capable of carrying, in tension or compression, a force equal to the lesser of the product of the larger footing design gravity load times the seismic coefficient, $S_D$, divided by 10 and 25 percent of the smaller footing design gravity load.

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 nominal strength of the deep foundation element.
2. The axial and shear forces and moments from the seismic load effects including overstrength factor in accordance with Section 12.4.9 2.3.6 or 12.14.3.2 2.4.5 of ASCE 7.

1810.3.9.4 Seismic reinforcement. 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.

Exceptions:

1. Isolated deep foundation elements supporting posts of Group R-3 and U occupancies not exceeding two stories of light-frame construction shall be permitted to be reinforced as required by rational analysis but with not less than one No. 4 bar, without ties or spirals, where detailed so the element is not subject to lateral loads and the soil provides adequate lateral support in accordance with Section 1810.2.1.
2. Isolated 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 shall be permitted to be reinforced as required by rational analysis but with not less than one No. 4 bar, without ties or spirals, where the lateral load, $E$, to the top of the element does not exceed 200 pounds (890 N) and the soil provides adequate lateral support in accordance with Section 1810.2.1.
3. Deep foundation elements supporting the concrete foundation wall of Group R-3 and U occupancies not exceeding two stories of light-frame construction shall be permitted to be reinforced as required by rational analysis but with not less than two No. 4 bars, without ties or spirals, 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 , 2.3.6 or 12.14.3.2 2.4.5 of ASCE 7 and the soil provides adequate lateral support in accordance with Section 1810.2.1.

4. Closed ties or spirals where required by Section 1810.3.9.4.2 shall be permitted to be limited to the top 3 feet (914 mm) of 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.

1810.3.11.2 Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E or F, deep foundation element resistance to uplift forces or rotational restraint shall be provided by anchorage into the pile cap, designed considering the combined effect of axial forces due to uplift and bending moments due to fixity to the pile cap. Anchorage shall develop a minimum of 25 percent of the strength of the element in tension. Anchorage into the pile cap shall comply with the following:

1. In the case of uplift, the anchorage shall be capable of developing the least of the following:
   1.1. The nominal tensile strength of the longitudinal reinforcement in a concrete element.
   1.2. The nominal tensile strength of a steel element.
   1.3. The frictional force developed between the element and the soil multiplied by 1.3.

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

2. In the case of rotational restraint, the anchorage shall be designed to resist the axial and shear forces, and moments resulting from the seismic load effects including overstrength factor in accordance with Section 12.4.3 , or 12.14.3.2 2.4.5 of ASCE 7 or the anchorage shall be capable of developing the full axial, bending and shear nominal strength of the element.

   Where the vertical lateral force-resisting elements are columns, the pile cap flexural strengths shall exceed the column flexural strength. The connection between batter piles and pile caps shall be designed to resist the nominal strength of the pile acting as a short column. Batter piles and their connection shall be designed to resist forces and moments that result from the application of seismic load effects including overstrength factor in accordance with Section 12.4.3 or 12.14.3.2 of ASCE 7.

1810.3.12 Grade beams. For structures assigned to Seismic Design Category D, E or F, grade beams shall comply with the provisions in Section 18.13.3 of ACI 318 for grade beams, except where they are designed to resist the seismic load effects including overstrength factor in accordance with Section 12.4.3 or 12.14.3.2 of ASCE 7.

Reason: This proposal is a coordination proposal to bring the 2018 IBC up to date with the provision of the 2016 edition of ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7-16).

Section 1803.5.12 - This proposal corrects a reference to the ASCE 7 Standard. ASCE 7 Chapter 21 includes several different procedures for performing site specific seismic hazard studies. In order to properly permit all of these procedures, reference to Chapter 21 in its entirety is necessary.

Section 1810.3.6.1 , 1810.3.9.3 , 1810.3.11.2 , 1810.3.12 - ASCE 7-16 moved all of the Load Combinations including seismic from Chapter 12 to Chapter 2. This proposal is necessary to correct the reference on Load Combinations including overstrength seismic loads to the appropriate location in ASCE 7. [NOTE: The Exception for Section 1810.3.11.2 is not new. When revising the ASCE 7 Section number to 2.3.6, the entire Exception was underlined by the cdpAccesssss system. The only change is to the ASCE 7 Section number.]

Cost Impact: Will not increase the cost of construction

The proposed changes will not impact the cost of construction. This proposal is a re-organization of the pointers in the IBC to refer to the referenced loading standard ASCE 7. ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures will be updated from the 2010 edition to the 2016 edition as an Administrative Update to the 2018 I-Codes. As of the submission date of this code change proposal, the ASCE 7 Standards Committee has completed the committee balloting on the technical changes. The document designated ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures is expected to be competed, published, and available for purchase prior to the ICC Public Comment Hearings for Group B in October of 2016. Any person interested in obtaining a public comment copy of ASCE 7-16 may do so by contacting James Neckel at ASCE (jneckel "at" asce.org).

S166-16 :
1803.5.12-

2016 ICC PUBLIC COMMENT AGENDA Page 2390
Public Hearing Results

Committee Action: Approved as Submitted

Committee Reason: This code change updates the Chapter 18 provisions for consistency with the latest edition of the standard, ASCE 7, which was updated in ADM94-16.

Assembly Action: None

Individual Consideration Agenda

Proponent: Ed Berkel, representing ICC Code Correlation Committee (ccc@icc.org) requests Disapprove.

Commenter's Reason: The Code Correlation Committee requests Disapproval of this code change proposal in order to bring a correlation issue to the attention of the full membership at the Public Comment Hearings and to allow the membership to coordinate action on this code change proposal with action taken on Code Change Proposal ADM94-16.

ADM94-16 is the administrative update to referenced standards in the I-Codes. One of these standards, ASCE7, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, was proposed for update to ASCE7-16. However, a successful assembly motion requests that the referenced ASCE7 remain at ASCE7-10 as it presently is referenced in the 2015 I-Codes. This code change proposal coordinates with and relies upon reference to ASCE7-16.

The Code Correlation Committee is a standing committee of the International Code Council whose objectives, procedures and organization are set forth in Council Policy CP#44-13. The objective of the Code Correlation Committee is to maintain technical and editorial consistency among the International Codes and to assist staff in the evaluation and processing of code change proposals and comments that are exclusively editorial.
Proposed Change as Submitted

Proponent: Edwin Huston, representing National Council of Structural Engineers' Associations (NCSEA) (huston@smithhustoninc.com)

2015 International Building Code

Revise as follows:

1804.2 Underpinning. Where underpinning is chosen to provide the protection or support of adjacent structures, the underpinning system shall be designed and installed as permanent structural elements in accordance with provisions of this chapter and Chapter 33 and inspected in accordance with the provisions of Chapter 17.

Reason: Underpinning, as different from temporary shoring, is utilized to permanently stabilize foundations and remain as the permanent support element. Temporary construction elements are not subject to the same design and inspection requirements as permanent elements. This clarifies that these permanent elements should be designed and inspected as the permanent elements that they are.

Cost Impact: Will increase the cost of construction

Most underpinning is currently designed as a permanent element. The amount inspected as a permanent element in practice is hard to determine. It would increase costs for inspection in those instances where installers are not providing the requisite inspections.

Public Hearing Results

Committee Action: Disapproved

Committee Reason: The committee feels that a public comment is in order on this proposal. There is some concern with referring to the underpinning as permanent and it was suggested that perhaps the wording should be along the lines of "permanent protection of adjacent structure". Another suggestion is to consider adding a definition of underpinning and require that it be designed in accordance with the provision of the code. The reference to Chapter 17 is either too broad or not necessary at all. Prefer that it refer to the specific portions of Chapter 17 that are applicable.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Edwin Huston, representing National Council of Structural Engineers' Associations (NCSEA) (huston@smithhustoninc.com) requests Approve as Modified by this Public Comment.

Modify as follows:

2015 International Building Code

1804.2 Underpinning. Where underpinning is chosen to provide the protection or support of adjacent structures, the permanent elements of underpinning system that remain after construction and are required to support load indefinitely shall be designed and installed as permanent structural elements in accordance with provisions of this chapter and Chapter 33 and inspected in accordance with the provisions of Chapter 17. Temporary stabilization elements, such as shoring, needed during underpinning are not required to be designed as permanent elements.

Commenter's Reason: Public Comment: Underpinning is an extension of an existing structure's foundation to a lower elevation in order to match the depth of the new adjacent excavation. Underpinning permanently supports adjacent structure, making it a permanent foundation element that should be designed as such.

Underpinning is not "permanent protection" and in fact is not protection at all. Temporary shoring to allow the installation of underpinning should not be confused with the underpinning itself.
S179-16
IBC: 1806.2.

Proposed Change as Submitted

Proponent: Lori Simpson, P.E., G.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

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

Presumptive load-bearing values shall apply to materials with similar physical and engineering characteristics. Very soft to soft clay or silt (CL, CH, MH) and dispositions. Mud very loose to loose silt (ML), organic silt and clay (OL), organic clays (OH), peat (Pt), or unprepared undocumented fill shall not be assumed to have a presumptive load-bearing capacity unless data to substantiate the use of such a value are submitted and an analysis is performed.

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

Reason: There should be a requirement that an approved method of analysis be used to substantiate the data submitted for approval.

Mud is not a recognized geotechnical "CLASS OF MATERIAL". A disposition is also not a recognized geotechnical term for use in determining soil classification.

Undocumented fill is a more appropriate definition because it implies the fill has not been evaluated for load bearing and settlement. Very soft to soft clays and silts, very loose to loose silts, organic silts and clays, and undocumented fill shall be evaluated by a design professional with subsurface explorations and in situ testing. Allowing structures to be supported on undocumented fill is not in accordance with generally accepted engineering practice. Fill shall be engineered in accordance with Section 1804.5

Click here to view members of the GeoCoalition who prepared this proposal. (http://www.piledrivers.org/files/8e1f877f-e55e-438e-9896-97d962b138b3--8e2ea8a9-bafa-4bd8-9d45-16f92e11d37c/members-of-the-geocoalition.pdf)

Cost Impact: Will not increase the cost of construction

This code change will not increase the cost of construction because it is the standard of practice.

Public Hearing Results

Committee Action: Disapproved

Committee Reason: The committee believes it is not appropriate to revise this to read "an approved method of analysis" when addressing presumptive load-bearing values. When looking at higher load-bearing values, the current text requires that documentation is submitted for approval and the building official would have the data needed to support those higher values.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Lori Simpson, P.E., G.E., representing GeoCoalition (lsimpson@langan.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

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

Presumptive load-bearing values shall apply to materials with similar physical and engineering characteristics. Very soft to soft clay or silt (CL, CH, MH) and very loose to loose silt (ML), organic silt and clay (OL, OH), peat (Pt), or undocumented fill shall not be assumed to have a presumptive load-bearing capacity unless data to substantiate the use of such a value are submitted and an analysis is performed.

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

**Commenter's Reason:** 1 The committee did not like “an approved method of analysis”, so this requirement is removed.
2. As the committee did not object to the other changes, i.e. the correct terminology for soil types, these changes remain. As previously reasoned, it is important to use correct soil classification terminology - “mud” is not a correct term, for example, and “disposition” is not a recognized geotechnical term for use in determining soil classification.
Committee Action: Disapproved

Assembly Action: None

IBC: 1806.3 (New).

Proposed Change as Submitted

Proponent: Lori Simpson, P.E., G.E., representing GeoCoalition

2015 International Building Code

Add new text as follows:

1806.3 Settlement Settlement shall be considered in accordance with Section 1808.2.

Reason: Reference to settlement in Section 1806 Presumptive Load-Bearing Values of Soils bring consistency with Section 1808 Foundations in that allowable bearing capacity should not exceed limiting differential settlements.

Click here to view the members of the GeoCoalition who prepared this proposal. (http://www.piledrivers.org/files/8e1f877f-e55e-438e-9896-97d962b138b3-8e2ea8a9-bafa-4bd8-9d45-16f92e11d37c/members-of-the-geocoalition.pdf)

Cost Impact: Will not increase the cost of construction
There is not cost increase because it just references another section of the code.

Public Hearing Results

Committee Reason: The proposed cross reference is not necessary.

Assembly Action: None

Individual Consideration Agenda

Proponent: Scott Campbell, representing Portland Cement Association (scampbell@cement.org) requests Approve as Submitted.

Commenter's Reason: The proposed change clarifies the code and serves to notify that the settlement limits in Section 1808.2 must be met. Without the proposed change all the requirements might not be clear to users.
Committee Action: Disapproved

Assembly Action: None

Proposed Change as Submitted

Proponent: Gerald Gunny, representing Southern Nevada Chapter of the International Code Council

2015 International Building Code

Revise as follows:

1807.2.3 Safety factor. Retaining walls shall be designed to resist the lateral action of soil to produce sliding and overturning with a minimum safety factor of 1.5 in each case. The load combinations of Section 1605 shall not apply to this requirement. Instead, design shall be based on 0.7 times nominal earthquake loads, 0.6 times nominal wind loads, 1.0 times other nominal loads, and investigation with one or more of the variable loads set to zero. The safety factor against lateral sliding shall be taken as the available soil resistance at the base of the retaining wall foundation divided by the net 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. Where wind loads are included, the minimum safety factor for retaining wall sliding and overturning shall be 1.3.

Reason: The intent of this provision applies to retaining walls where the designer includes wind loading in the analysis. For example, this would apply to retaining walls that include a freestanding wall, fence or other light structure, subject to wind loading, atop and supported by the retaining wall. The section requires a 0.7 factor be applied to seismic loads and a 1.0 factor for all other nominal loads. The 0.7 factor is to convert the ASCE 7-10 strength level seismic loading to service level loading. Likewise the proposed 0.6 factor is to convert the ASCE 7-10 strength level wind loading to service level loading. The 1.3 minimum factor of safety for wind loads is consistent with the long standing historical practice of allowable stress increases for earthquake and wind loads and should be less than the 1.5 factor of safety for other nominal loads.

Cost Impact: Will not increase the cost of construction

The costs associated with wall construction material and labor quantities would be reduced due to the reduction in wind loading and factor of safety when the wind loading case governs the design of the retaining wall and foundation.

Committee Reason: This proponent did not provide an explanation of the basis for the proposed load factors and safety factors. There is concern on whether the reference to "nominal" wind load is correct.

Public Hearing Results

Committee Action: Disapproved

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Kevin McOsker, representing Southern Nevada Chapter of ICC (ktm@ClarkCountyNV.gov) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

1807.2.3 Safety factor. Retaining walls shall be designed to resist the lateral action of soil to produce sliding and overturning with a minimum safety factor of 1.5 in each case. The load combinations of Section 1605 shall not apply to this requirement. Instead, design shall be based on 0.7 times nominal earthquake loads, 0.6 times nominal wind loads, 1.0 times other nominal loads, and investigation with one or more of the variable loads set to zero. The safety factor against lateral sliding shall be taken as the available soil resistance at the base of the retaining wall foundation divided by the net 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. Where wind loads are included, the minimum safety factor for retaining wall sliding and overturning shall be 1.3.

Exception: Where earthquake or wind loads are included, the minimum safety factor for retaining wall sliding and overturning shall be 1.1.
Commenter’s Reason: The committee disapproved this code proposal based on two issues: 1) the term “nominal” in reference to the wind load requirements; and 2) the proponent did not provide an explanation of the load and safety factors when wind load is a design consideration. The committee overlooked that both earthquake and other loads in this section are described as “nominal” loads, and for consistency we used the term “nominal” in the same context. In addition, Chapter 2 (Definitions) the term “nominal” refers to loads determined from Chapter 16; so it would appear that the use of the term “nominal” when used for wind loads is consistent with the definition and other uses in the code. It should be noted that both earthquake and wind loads are required by Chapter 16 to be based on ASCE 7, and these are strength level loads. Based on the committee’s comments, we are revising our original proposal to include the wind load safety factor to be the same as earthquake loads. The 1.1 minimum factor of safety for wind loads is consistent with the long standing practice of considering earthquake and wind loads interchangeably as short-term loads. The 0.6 factor on wind loads modifies the strength level design load to the service level design load, consistent with both sections 1605.3.1 and 1605.3.2; these sections also applies the 0.7 factor for earthquake loads. Without this code change, wind load governed designs would be considerably more conservative than necessary. The code as currently written does not address the possibility of wind loads on retaining wall structures. Where retaining walls include a free standing wall, fence or other structures; the code user needs adequate guidance when wind loads governs the design.
Committee Action: Disapproved

Assembly Action: None

2015 International Building Code

Revise as follows:

1809.5 Frost protection. Except where otherwise protected from frost, foundations, exterior landings as required by Sections 1010.1.5 and 1010.1.6, and other permanent supports of buildings and structures shall be protected from frost by one or more of the following methods:

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

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

1. Assigned to Risk Category I.
2. Area of 600 square feet (56 m²) or less for light-frame construction or 400 square feet (37 m²) or less for other than light-frame construction.
3. Eave height of 10 feet (3048 mm) or less.

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

Reason: The proposed change is to help make it clear that the exterior landings at exit doors in locations subject to frost conditions be provided with frost protection the same as the foundations of the building being exited. Adding “exterior landings” in this section will make it clear that the landing areas immediately outside a required exit door must be provided with the same frost protected foundation as that of the building. In cold climate areas, this would help prevent the typical concrete landing (the exit discharge), that is in most cases level from inside to outside, from heaving and potentially compromising the required operability of the exit door itself. The reference to both 1010.1.5 and 1010.1.6 was so it was clear that the protected area need not extend to the entire area of a landing or platform, only the landing area that is required by code and immediately outside of the required exit door.

Cost Impact: Will increase the cost of construction

The frost protection will increase the cost of construction, but given the limited number of foundations associated with the exits in comparison to the overall cost of a buildings foundation, that increase should be minimal when compared to the overall cost. Also, a reasonable payback to the costs should exist when considering decreased maintenance costs and the significant costs of a repair in those situations where the heave has been significant enough block an exit.

Public Hearing Results

Committee Action: Disapproved

Committee Reason: This code change does not consider that heave occurs over an extended period of time so it can be readily addressed. In a lot of jurisdictions this could mean an onerous required depth to resist frost heave.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent : James Smith, representing ICC Region III Code Development Committee (jsmith@awc.org) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

1809.5 Frost protection. Except where otherwise protected from frost, foundations, exterior landings as at required exits as specified by Sections 1010.1.5 and 1010.1.6, and other permanent supports of buildings and structures shall be protected...
from frost by one or more of the following methods:

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

- **Exception:** Free-standing buildings meeting all of the following conditions shall not be required to be protected:
  1. Assigned to Risk Category I.
  2. Area of 600 square feet ($56 \text{ m}^2$) or less for light-frame construction or 400 square feet ($37 \text{ m}^2$) or less for other than light-frame construction.
  3. Eave height of 10 feet ($3048 \text{ mm}$) or less.

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

**Commenter's Reason:** Although we felt the original proposal was clear that the added area of frost protection was only being proposed at the exterior landings of the means of egress doors as referenced by IBC s. 1010.1.5 and only of the size required by 1010.1.6, at least one of the committee members commented about the lack of those details. Accordingly we have modified the text to be clear that the proposal is only calling for the frost protection at the landings for the required means of egress doors. Another of the committee comments inferred that this change would mandate footings extending below frost line at all the landings, but the existing text begins with the qualification “Except where otherwise provided”. It is not uncommon in Wisconsin and other areas of the upper Midwest for designers to “otherwise provide” a resistant design where it is more cost effective than installing frost footings so that should not be a reason for disapproving this proposal with public comment.

Another of the committee reasons for disapproval was that the proposal did not consider that heave occurs over an extended period of time so it can be readily addressed. Correcting the effects of heave after it has occurred was the impetus for proposing this change. Although correcting the effects of heave after it occurs at required exits must always be addressed, the fact that it may or may not be "readily addressed" does little to mitigate the reason for the heave itself . . . the lack of a including a resistant design from the beginning. This proposal is addressing the frost heave issue when it should be addressed, during the design of the buildings egress system, not as a corrective measure after the fact.
Proposed Change as Submitted

Proponent : S M Schilder, representing self (smsengr@consolidated.net)

2015 International Building Code

Add new definition as follows:

SECTION 202 DEFINITIONS

SEGMENTED PILES. Piles consist of precast concrete segments, usually manufactured cylinders, which are installed one by one on top of one another, pressed into the ground by hydraulically jacking against the underside of the existing structure. The weight of the structure is used to create the reactive force that allows the pile segments to be driven into the soil. These piles may be categorized as driven displacement piles, which displace and force aside the surrounding soil as they are driven. The piles transfer load to the foundation soils primarily through skin friction along the length of the pile, although some end-bearing load transfer also occurs.

Add new text as follows:

1705.10 Segmented pile foundations. Special inspections shall be performed periodically during the installation of segmented pile foundations by the registered design professional in responsible charge, or a designated agent. The information recorded shall include installation equipment used, pile dimension, segment quantities, final depth and other pertinent installation data as required by the registered design professional in responsible charge.

Revise as follows:

1802.1 Definitions. The following words and terms are defined in Chapter 2:

DEEP FOUNDATION.
DRILLED SHAFT.
Socketed drilled shaft.
HELICAL PILE.
MICROPILE.
SEGMENTED PILE.
SHALLOW FOUNDATION.

Add new text as follows:

1810.3.1.6 Segmented piles. Segmented piles shall be designed and manufactured in accordance with accepted engineering practice to resist stresses induced by installation into the ground and services loads.

1810.3.5.4 Segmented concrete piles. The diameter of precast segmented concrete piles shall be not less than 4 inches.

1810.4.13 Concrete segmented piles. Install segmented piles to the required minimum depth as specified by the plans, if applicable. The contractor shall provide segmented piles capable of withstanding the segmented pile driving stresses and design loads, and capable of being driven to refusal at or below a minimum design depth, when specified by the engineer. Segmented piles that reach refusal before attaining the minimum required depth as specified shall be subject to the following:

1. Terminate pile at refusal depth obtained with approval of engineer, or
2. Replace pile with pile having a smaller cross sectional area, installed not less than three pile diameters from the terminated pile, or
3. Implement water jetting, or
4. Abandon the pile and pre-drill an adjacent pile not less than three pile diameters from the terminated pile.

Reason: This product is currently in use as a common method of repair for residential and other lightly loaded foundations. Methodologies for the product were established in committee (http://www.foundationperformance.org/projects/FPA-SC-08-1.pdf) to provide guidance for the use and installation.

References

Committee Action: Disapproved
Assembly Action: None

Public Hearing Results

Committee Reason: The proposed definition would not not cover all types of segmented piles. There are no prescriptive requirements contained in the proposed text. The reason refers to "lightly loaded" foundations, but the proposed text does not state that as a limitation.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:
Proponent: S M Schilder, representing Foundation Performance Association (smsengr@consolidated.net) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

SECTION 202 DEFINITIONS

Lightly loaded foundations. Lightly loaded foundations are defined as one and two family dwellings, and one to four story commercial buildings.

SEGMENTED PILES. Piles consist of precast concrete segments, usually manufactured cylinders, which are installed one by one on top of one another, pressed into the ground by hydraulically jacking against the underside of the existing structure. The weight of the structure is used to create the reactive force that allows the pile segments to be driven into the soil. These piles may be categorized as driven displacement piles, which displace and force aside the surrounding soil as they are driven. The piles transfer load to the foundation soils primarily through skin friction along the length of the pile, although some end-bearing load transfer also occurs.

Segmented Piles are precast units that are typically cylindrical in shape, approximately twelve inches long and six inches in diameter, although other shapes and lengths are permitted. Maximum size is that length and diameter that can be manhandled without the use of equipment. The segmented piles noted in this section are limited to lightly loaded foundations.

1810.4.13.1 Exterior Segmented Piles. Exterior Segmented Pilings shall have a maximum spacing as follows:

1. Eight feet center to center on a one-story wood framed structure with or without a masonry veneer
2. Seven Feet center to center on a two-story structure with masonry veneer on the first story only and a one and a half story when the wall supports the second floor joists
3. Six feet center to center on two-story structures with a masonry veneer
4. Structures with three of more stories should have an engineered underpinning design
5. Where the existing foundation does not have a structurally adequate exterior grade beam to span between the Segmented Piles, then a structural member designed by an Engineer shall be installed at the Pile under the foundation in order to reduce the point load on the foundation and to increase the Pile driving force.

1810.4.13.2 Interior Segmented Piles. Interior Segmented Piles shall have a maximum spacing as follows:

1. In a stiffened slab foundation, Piles shall be placed at grade beam intersections, and below grade beams at a maximum of ten feet center to center.
2. Eight feet center to center under walls of one story structures
3. Seven feet center to center under walls of two-story structures
4. Structures of three of more stories shall have an underpinning plan designed by an Engineer
5. Where the measurement across a room is twelve feet or greater, a Pile or Piles shall be placed near the middle of the room. The location of the Piles will be no greater than eight feet from a Pile supported wall or adjacent Pile.

6. Where there are no interior grade beams present then a structural member designed by an Engineer shall be installed at the Pile under the foundation in order to reduce the point load on the foundation and to increase the Pile driving force.

**Commenter's Reason:** These modifications should answer and complete the committe's issues with the submittal such that the submittal as modified is acceptable

S203-16
Proposed Change as Submitted

**Proponent:** Dale Biggers, P.E., Boh Bros. Construction Co., L.L.C., representing GeoCoalition (dbiggers@bohbros.com); Lori Simpson, P.E., G.E., representing GeoCoalition; Daniel Stevenson, P.E., representing GeoCoalition; E. Anna Sellountou, PhD, PE, representing GeoCoalition

**2015 International Building Code**

Revise as follows:

1810.3.2.6 **Allowable axial stresses.** No change to text.

**TABLE 1810.3.2.6**

<table>
<thead>
<tr>
<th>MATERIAL TYPE AND CONDITION</th>
<th>MAXIMUM ALLOWABLE AXIAL STRESS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Concrete or grout in compression&lt;sup&gt;b&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>A. Cast-in-place with a mandrel-driven permanent casing in accordance with Section 1810.3.2.7</td>
<td>0.4 $f_c$</td>
</tr>
<tr>
<td>B. Mandrel-driven elements not meeting Section 1810.3.2.7</td>
<td>0.33$f_c$</td>
</tr>
<tr>
<td>C. Cast-in-place in a driven pipe or tube in accordance with Section 1810.3.5.3.2 or other suitable permanent casing or in rock</td>
<td>0.43$f_c$</td>
</tr>
<tr>
<td>D. Micropiles cased length</td>
<td>0.4 $f_c$</td>
</tr>
</tbody>
</table>
| E. Cast-in-place without a permanent casing  
  With verification of area versus depth<sup>c</sup> | 0.3$f_c$ |
| Without verification of area versus depth<sup>c</sup> | 0.25 $f_c$ |
| F. Precast non-prestressed | 0.33$f_c$ |
| G. Precast prestressed | 0.33$f_c$ - 0.27$f_{pc}$ |

2. Nonprestressed reinforcement in compression

<table>
<thead>
<tr>
<th>MATERIAL TYPE AND CONDITION</th>
<th>MAXIMUM ALLOWABLE AXIAL STRESS</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Within Micropiles with rock sockets where strain compatibility is checked by the engineer</td>
<td>0.4 $f_y$ ≤ 30,000 psi</td>
</tr>
<tr>
<td>0.4 $f_y$ ≤ 48,000 psi</td>
<td></td>
</tr>
</tbody>
</table>

3. Steel in compression<sup>c</sup>

<table>
<thead>
<tr>
<th>MATERIAL TYPE AND CONDITION</th>
<th>MAXIMUM ALLOWABLE AXIAL STRESS</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Cores within concrete-filled pipes or tubes</td>
<td>0.5 $F_y$ ≤ 32,000 psi</td>
</tr>
<tr>
<td>B. Pipes, tubes (including cased sections of Micropiles) or H-piles, where justified in accordance with Section 1810.3.2.8</td>
<td>0.5 $F_y$ ≤ 32,000 psi</td>
</tr>
<tr>
<td>C. Pipes or tubes for micropiles</td>
<td>0.4 $F_y$ ≤ 32,000 psi</td>
</tr>
<tr>
<td>D. Other pipes, tubes or H-piles</td>
<td>0.35 $F_y$ ≤ 16 24,000 psi</td>
</tr>
<tr>
<td>E. Helical piles</td>
<td>0.55 $F_y$ ≤ 32,000 psi; 0.5 $F_y$ ≤ 32,000 psi</td>
</tr>
</tbody>
</table>

4. Nonprestressed reinforcement in tension<sup>d</sup>

<table>
<thead>
<tr>
<th>MATERIAL TYPE AND CONDITION</th>
<th>MAXIMUM ALLOWABLE AXIAL STRESS</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Within micropiles</td>
<td>0.6 $f_y$ ≤ 72,000 psi</td>
</tr>
<tr>
<td>B. Other conditions</td>
<td>0.5 $f_y$ ≤ 4460,000 psi</td>
</tr>
</tbody>
</table>

5. Steel in tension<sup>d</sup>

<table>
<thead>
<tr>
<th>MATERIAL TYPE AND CONDITION</th>
<th>MAXIMUM ALLOWABLE AXIAL STRESS</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8</td>
<td>0.5 $F_y$ ≤ 32,000 psi</td>
</tr>
<tr>
<td>B. Other pipes, tubes or H-piles</td>
<td>0.35 $F_y$ ≤ 1624,000 psi</td>
</tr>
<tr>
<td>C. Helical piles</td>
<td>0.605 $F_y$ ≤ 32,000 psi; 0.6 $F_y$ ≤ 32,000 psi</td>
</tr>
</tbody>
</table>

6. Timber

In accordance with the AWC 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 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.

c. Strength of shaft depends on material strength and cross-sectional area. Area can be verified by visual, mechanical or non-destructive testing methods.

d. A maximum tensile stress in the table is permitted where the resulting crack widths are computed to be less than 0.013 inches or where corrosion protection is provided.

e. The splices should be considered and may be the limiting factor in tension.
Reason: To clarify in the section title that this section is only referring to axial stresses.

• Title: The word “Axial” is added in the title to clarify the intent of the table.

• Capital letters A, B, C etc. are added for convenience for discussing individual reasons for each change (the following reasons are coded to correspond to the table designations). These capital letters may be removed if desired once the reasons are understood.

• 1.A.: The word “mandrel-driven” is added because section 1810.3.2.7 covers only mandrel-driven piles.

• 1.B.: To differentiate mandrel-driven elements from “other suitable permanent casing” in paragraph 1.C. Stress limit for 1.B. should be lower compared to 1.A and 1.C.

• 1.C.: Concrete cast in a driven pipe or tube (in accordance with Section 1810.3.5.3.2) or other suitable permanent casing or in rock is as good or better than mandrel-driven piles. The commentary for 1810.3.2.7 uses confinement as a basis for higher allowable stresses.

1810.3.2.7 says “Confinement is the technical qualification that permits the use of increased allowable compressive stresses”. The shape is known prior to placing concrete and contamination from soil is not possible. “Suitable” means able to withstand installation and avoid collapse for drilled shafts or micropiles.

• 1.D.: A new section for the cased length of micropiles is added. FHWA micropile manual (ASD) chapter 5 (equation 5-1) has allowed 0.4 for over a decade.

• 1.E.: A distinction has been inserted to consider if the area has been verified. Footnote (c) is added for further explanation. For uncased shafts in soil there is uncertainty in the cross-sectional area. Previous versions (2003) had 0.25fc’ for all cases and is more appropriate for unstable soils and difficult construction situations. Strength of shaft depends on both material strength and cross-sectional area. Area can be verified by visual, mechanical or non-destructive testing methods.

• 1.F.: Editorial hyphen added.

• 2.A: This case is added for a specific micropile condition. Please refer to "Grout confinement influence on strain compatibility in micropiles" report, dated Nov. 21, 2006, from DFI-ADSC.

• 3: Footnote (e) added for all options as splices or joints can be the weak link.

• 3.B.: Added a clarification that cased sections of micropiles fall under this category.

• 3.D.: Higher strength steel (oil casing 70ksi or above) has become commonplace. H-piles below 50ksi are no longer common. This change allows more economic use of the higher strength steels.

• 3.E.: The fixed limit provides consistency with the other pile types. The specific limit was chosen based on the available material limits at the time of this code proposal. The multiplier for Fy has been lowered to be consistent with other steel sections and to ensure a safety factor of 2.

• 4.: General comments are added for both options. Footnote (d) is provided to ensure corrosion protection. The value of 0.013 inches in Footnote (d) comes from ACI recommendation (ACI 224R “Control of Cracking in Concrete Structures”). Footnote (e) added as splices or joints can be the weak link in tension.

• 4.A.: A 150ksi bar, which has a yield strength of 120ksi, is commonly available. We added a specific upper limit to be consistent with the rest of the table. High tensile strains in steel reinforcement can lead into cracking of the surrounding concrete and corrosion of the steel reinforcement.

• 4.B.: Limit increased to allow use of 150 ksi bars. A 150ksi bar, which has a yield strength of 120ksi, is commonly available.

• 5.: Footnote (e) is added as a general statement for all options. Splices or joints can be the weak link in tension.

• 5.B.: Upper limit is increased. Higher strength steel (oil casing 70ksi or above) is commonly available and has been used successfully in many projects. H-piles below 50ksi are not commonly available.

• 5.C.: Changed the upper limit to a specific value to provide consistency with the other pile types. The multiplier for Fy has been lowered to be consistent with other steel sections.

Click here to view the members of the GeoCoalition who developed this proposal (http://www.piledrivers.org/files/8e1f877fe55e-438e-9896-97d962b138b3--8e2ea8a9-bafa-4bd8-9d45-16f92e11d37c/members-of-the-geocoalition.pdf)

• FHWA micropile manual (ASD) chapter 5 (equation 5-1), FHWA NHI-05-039.

• Grout confinement influence on strain compatibility in micropiles” report, dated Nov. 21, 2006, from DFI-ADSC

• ACI 224R “Control of Cracking in Concrete Structures”.

Cost Impact: Will not increase the cost of construction

This change in the section title added for clarity, but has no effect on costs.

Most of the code change proposals will not increase the cost of construction, and could result in reduced costs since higher allowable stresses (justified by increased available material strengths) can be used. Changes to 1E (but only if the area is not verified), 3E and 5C might increase the costs but will enhance the reliability (and reduce the long-term cost).
Committee Action: Disapproved

Committee Reason: The basis for these changes to allowable stresses is not from a consensus process. There needs to be more detailed technical justification for all these proposed changes.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Daniel Stevenson, P.E., representing GeoCoalition requests Approve as Modified by this Public Comment.

Replace Proposal as Follows:

2015 International Building Code

### TABLE 1810.3.2.6
ALLOWABLE STRESSES FOR MATERIALS USED IN DEEP FOUNDATION ELEMENTS

<table>
<thead>
<tr>
<th>MATERIAL TYPE AND CONDITION</th>
<th>MAXIMUM ALLOWABLE STRESS&lt;sup&gt;a&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Concrete or grout in compression&lt;sup&gt;b&lt;/sup&gt;</td>
<td></td>
</tr>
<tr>
<td>Cast-in-place with a permanent casing in accordance with Section 1810.3.2.7</td>
<td>0.4 ( f_c )</td>
</tr>
<tr>
<td>Cast-in-place in a pipe, tube, other permanent casing or rock</td>
<td>0.33 ( 0.4 f_c )</td>
</tr>
<tr>
<td>Cast-in-place without a permanent casing</td>
<td>0.3 ( f_c )</td>
</tr>
<tr>
<td>Precast non prestressed</td>
<td>0.33 ( f_c )</td>
</tr>
<tr>
<td>Precast prestressed</td>
<td>0.33 ( f_c ) - 0.27 ( f_{pc} )</td>
</tr>
<tr>
<td>2. Non prestressed reinforcement in compression</td>
<td>0.4 ( f_y ) ≤ 30,000 psi</td>
</tr>
<tr>
<td>3. Steel in compression</td>
<td></td>
</tr>
<tr>
<td>Cores within concrete-filled pipes or tubes</td>
<td>0.5 ( f_y ) ≤ 32,000 psi</td>
</tr>
<tr>
<td>Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8</td>
<td>0.5 ( f_y ) ≤ 32,000 psi</td>
</tr>
<tr>
<td>Pipes or tubes for micropiles</td>
<td>0.4 ( f_y ) ≤ 32,000 psi</td>
</tr>
<tr>
<td>Other pipes, tubes or H-piles</td>
<td>0.35 ( f_y ) ≤ 16,000 psi</td>
</tr>
<tr>
<td>Helical piles</td>
<td>0.6 ( f_y ) ≤ 0.5 ( F_U )</td>
</tr>
<tr>
<td>4. Non prestressed reinforcement in tension</td>
<td>0.6 ( f_y )</td>
</tr>
<tr>
<td>Within micropiles</td>
<td></td>
</tr>
<tr>
<td>Other conditions</td>
<td>0.5 ( f_y ) ≤ 24,000 psi</td>
</tr>
<tr>
<td>5. Steel in tension</td>
<td></td>
</tr>
<tr>
<td>Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8</td>
<td>0.5 ( f_y ) ≤ 32,000 psi</td>
</tr>
<tr>
<td>Other pipes, tubes or H-piles</td>
<td>0.35 ( f_y ) ≤ 16,000 psi</td>
</tr>
<tr>
<td>Helical piles</td>
<td>0.6 ( F_Y ) ≤ 0.5 ( F_U )</td>
</tr>
<tr>
<td>6. Timber</td>
<td>In accordance with the ANSI/AWC NDS</td>
</tr>
</tbody>
</table>

---

<sup>a</sup> \( f_c \) 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 steel; \( F_U \) is the specified minimum tensile stress of structural steel.

<sup>b</sup> 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.

Commenter's Reason:

1. The minimum material thickness for pipes, tubes, and permanent casing required by this code exceeds the minimum material thickness required by 1810.3.2.7 ("Increased allowable compressive stress for cased cast-in-place elements"). Current commentary for code section 1810.3.2.7 uses confinement of the concrete for justification of the higher allowable stress. The confinement provided by pipes, tubes, and permanent casing will exceed that required by 1810.3.2.7, therefore the allowable stress of 0.40 \( f_c \) should be permitted for these deep foundation types.
2. For concrete or grout cast in a rock socket, the rock will also provide sufficient confinement to justify the higher allowable stress. FHWA publication NHI-05-039 "Micropile Design and Construction" recommends an allowable stress of 0.40 $f_c$ for uncased elements in rock.

**Cost Impact:** will not increase cost of construction. It will decrease costs in some cases since it may allow higher design loads where the geotechnical capacity is sufficiently higher than the structural strength.

Click here to view the 35 members of the GeoCoalition. (http://www.piledrivers.org/geocoalition-members/)

These 35 GeoCoalition members are structural and geotechnical engineers and contractors from across the country. They are in leadership positions of more than nine organizations including:

- DFI – Deep Foundations Institute
- PDCA – Pile Driving Contractors Association
- ADSC – Association of Drilled Shaft Contractors
- ASCE – American Society of Civil Engineers
- ASTM – American Society of Testing Materials
- ACI – American Concrete Institute
- SAME – Society of American Military Engineers
- NCSEA – National Council of Structural Engineers Associations
- GBA – Geoprofessional Business Association (formerly ASFE)

Leadership positions held include:
- Chair DFI Soil Nailing and Tiebacks Committee
- Chair PDCA Technical Committee
- President of Geo-Institute
- Chair ADSC-DFI Joint Micropile Committee
- President of PDCA
- Chair Earth Retaining Structures of ASCE/G-I
- Manager DFI Technical Committees
- Received five ASTM Standards Development Awards
- Chair DFI Helical Pile Committee
- Chair DFI Codes and Standards Committee
- Director of GBA
- President of DFI

**Bibliography:** FHWA NHI-05-039 "Micropile Design and Construction", Sabatini et al., 2005, pg 5-17

**Public Comment 2:**

**Proponent:** Daniel Stevenson, P.E., representing GeoCoalition (dstevenson@berkelapg.com) requests Approve as Modified by this Public Comment.

**Replace Proposal as Follows:**

**2015 International Building Code**

<table>
<thead>
<tr>
<th>TABLE 1810.3.2.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>ALLOWSABLE STRESSES FOR MATERIALS USED IN DEEP FOUNDATION ELEMENTS</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>MATERIAL TYPE AND CONDITION</th>
<th>MAXIMUM ALLOWABLE STRESS$^a$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Concrete or grout in compression$^b$</td>
<td>$0.4 f_c$</td>
</tr>
<tr>
<td>Cast-in-place with a permanent casing in accordance with Section 1810.3.2.7</td>
<td>$0.33 f_c$</td>
</tr>
<tr>
<td>Cast-in-place in a pipe, tube, other permanent casing or rock</td>
<td>$0.33 f_c$</td>
</tr>
<tr>
<td>Cast-in-place without a permanent casing</td>
<td>$0.3f_c$</td>
</tr>
<tr>
<td>Precast nonprestressed</td>
<td>$0.33 f_c$</td>
</tr>
<tr>
<td>Precast prestressed</td>
<td>$0.33 f_c - 0.27 f_{pc}$</td>
</tr>
<tr>
<td>2. Nonprestressed reinforcement in compression</td>
<td>$0.4 f_y \leq 30,000$ psi</td>
</tr>
<tr>
<td>3. Steel in compression</td>
<td></td>
</tr>
</tbody>
</table>
Cores within concrete-filled pipes or tubes | $0.5 \frac{F_y}{F} \leq 32,000 \text{ psi}$
---|---
Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8 | $0.5 \frac{F_y}{F} \leq 32,000 \text{ psi}$
Pipes or tubes for micropiles | $0.4 \frac{F_y}{F} \leq 32,000 \text{ psi}$
Other pipes, tubes or H-piles | $0.35 \frac{F_y}{F} \leq 16,000 \text{ psi}$
Helical piles | $0.6 \frac{F_y}{F} \leq 0.5 \frac{F_u}{F}$

4. Non prestressed reinforcement in tension

Within micropiles | $0.6 f_y$
Other conditions | $0.5 \frac{f_y}{F} \leq 24,000 \text{ psi}$

5. Steel in tension

Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8 | $0.5 \frac{F_y}{F} \leq 32,000 \text{ psi}$
Other pipes, tubes or H-piles | $0.35 \frac{F_y}{F} \leq 16,000 \text{ psi}$
Helical piles | $0.6 \frac{F_y}{F} \leq 0.5 \frac{F_u}{F}$

6. Timber | In accordance with the ANSI/AWC 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 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.

Commenter's Reason:

1. The upper stress level of 24,000 psi for non prestressed reinforcing in tension is inconsistent with ACI 318 and ACI 543R. In most cases ACI 318 design methods will result in a service level tension stress between 0.56 $f_y$ and 0.64 $f_y$, depending on the load factors required. The code's current allowable stress of 0.50 $f_y$ is slightly conservative but reasonable. For non prestressed reinforcing, ACI 318 requires that the upper limit for $f_y$ to be used in calculations for non prestressed reinforcing shall not exceed 80,000 psi. Setting the upper limit at 40,000 psi (50% of 80,000 psi) provides consistency between the allowable stress of 0.50 $f_y$ currently in the code and the requirements of ACI 318.
2. There is no historical basis for the allowable stress limit of 24,000 psi for non prestressed reinforcing. Prior to the 2009 IBC, there was no upper stress limit for non prestressed reinforcing in tension in the Codes. There was nothing in the 2009 reason statement of Code Change Proposal S160-07/08 to justify the imposed 24,000 psi stress limit.
3. A drastic difference in the allowable tension stress for reinforcing steel used in micropiles compared to other deep foundation types is not justifiable. The Code currently allows up to 0.60 $f_y$ with no upper limit for non prestressed steel in tension for micropiles. Steel bars with yield stress of 120,000 psi are often used in micropiles. Per the current code these bars will have an allowable tension stress of 72,000 psi in micropiles, compared to 24,000 psi for other deep foundation types.

Cost Impact: The proposal will not increase costs. The introduction of the 24,000 psi stress limit has led to a significant increase in cost of construction on some building projects. Increasing the upper limit to a level consistent with that allowed by ACI 318 will result in a reduction of construction cost.

Click here to view the 35 members of the GeoCoalition. (http://www.piledrivers.org/geocoalition-members/)

These 35 GeoCoalition members are structural and geotechnical engineers and contractors from across the country. They are in leadership positions of more than nine organizations including:
- DFI – Deep Foundations Institute
- PDCA – Pile Driving Contractors Association
- ADSC – Association of Drilled Shaft Contractors
- ASCE – American Society of Civil Engineers
- ASTM – American Society of Testing Materials
- ACI – American Concrete Institute
- SAME – Society of American Military Engineers
- NCSEA – National Council of Structural Engineers Associations
- GBA – Geoprofessional Business Association (formerly ASFE)

Leadership positions held include:
- Chair DFI Soil Nailing and Tiebacks Committee
- Chair PDCA Technical Committee
- President of Geo-Institute
- Chair ADSC-DFI Joint Micropile Committee
- President of PDCA
Public Comment 3:

Proponent: Daniel Stevenson, P.E., representing GeoCoalition requests Approve as Modified by this Public Comment.

Replace Proposal as Follows:

2015 International Building Code

<table>
<thead>
<tr>
<th>MATERIAL TYPE AND CONDITION</th>
<th>MAXIMUM ALLOWABLE STRESS[^1]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Concrete or grout in compression[^2]</td>
<td>(0.4 f_c)</td>
</tr>
<tr>
<td>Cast-in-place with a permanent casing in accordance with Section 1810.3.2.7</td>
<td>(0.4 f_c)</td>
</tr>
<tr>
<td>Cast-in-place in a pipe, tube, other permanent casing or rock</td>
<td>(0.33 f_c)</td>
</tr>
<tr>
<td>Cast-in-place without a permanent casing</td>
<td>(0.3 f_c)</td>
</tr>
<tr>
<td>Precast non prestressed</td>
<td>(0.33 f_c)</td>
</tr>
<tr>
<td>Precast prestressed</td>
<td>(0.33 f_c - 0.27 f_{pc})</td>
</tr>
<tr>
<td>2. Non prestressed reinforcement in compression</td>
<td>(0.4 f_y \leq 30,000) psi</td>
</tr>
<tr>
<td>3. Steel in compression</td>
<td></td>
</tr>
<tr>
<td>Cores within concrete filled pipes or tubes</td>
<td>(0.5 f_y \leq 32,000) psi</td>
</tr>
<tr>
<td>Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8</td>
<td>(0.5 f_y \leq 32,000) psi</td>
</tr>
<tr>
<td>Pipes or tubes for micropiles</td>
<td>(0.4 f_y \leq 32,000) psi</td>
</tr>
<tr>
<td>Other pipes, tubes or H-piles</td>
<td>(0.35 f_y \leq 16,240,000) psi</td>
</tr>
<tr>
<td>Helical piles</td>
<td>(0.6 f_y \leq 0.5 F_u)</td>
</tr>
<tr>
<td>4. Non prestressed reinforcement in tension</td>
<td></td>
</tr>
<tr>
<td>Within micropiles</td>
<td>(0.6 f_y)</td>
</tr>
<tr>
<td>Other conditions</td>
<td>(0.5 f_y \leq 24,000) psi</td>
</tr>
<tr>
<td>5. Steel in tension</td>
<td></td>
</tr>
<tr>
<td>Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8</td>
<td>(0.5 f_y \leq 32,000) psi</td>
</tr>
<tr>
<td>Other pipes, tubes or H-piles</td>
<td>(0.35 f_y \leq 16,240,000) psi</td>
</tr>
<tr>
<td>Helical piles</td>
<td>(0.6 f_y \leq 0.5 F_u)</td>
</tr>
<tr>
<td>6. Timber</td>
<td>In accordance with the ANSI/AWC NDS</td>
</tr>
</tbody>
</table>

[^1]: \(f_c\) 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 steel; \(F_u\) is the specified minimum tensile stress of structural steel.

[^2]: 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.

**Commenter’s Reason:**

1. The upper stress level of 16,000 psi for driven piles is not consistent with maximum allowable stress of 0.35 \(f_y\) and the material standards listed in section 1810.3.2.3. The most common grades of steel currently in use for HP sections that match the material standards required by the code have a yield strength of 50,000 psi. Steel with a yield strength of 65,000 psi is also currently available. Setting the upper limit at 24,000 will create consistency between the allowable stress of 0.35 \(f_y\) and the material standards currently listed in the code.

2. There is no historical basis for the upper level of allowable stress of 16,000 psi for pipes, tubes or H-piles. Prior to the 2009 IBC, there was no upper limit for pipes, tubes or H-piles specified in the Code. The upper stress level of 16,000 psi was first introduced into the IBC in the 2009 edition by Code Change Proposal S160-07/08. There was nothing presented in the reason statement of CCP S160-07/08 to justify the 16,000 psi stress limit.

**Cost impact:** The introduction of the 16,000 psi stress limit has led to an increase in cost of construction on some building projects. Increasing the upper limit to a level consistent with the material standards currently listed in the code will result in a
reduction of construction cost.

Click here to view the 35 members of the GeoCoalition. (http://www.piledrivers.org/geocoalition-members/)

These 35 GeoCoalition members are structural and geotechnical engineers and contractors from across the country. They are in leadership positions of more than nine organizations including:

- DFI – Deep Foundations Institute
- PDCA – Pile Driving Contractors Association
- ADSC – Association of Drilled Shaft Contractors
- ASCE – American Society of Civil Engineers
- ASTM – American Society of Testing Materials
- ACI – American Concrete Institute
- SAME – Society of American Military Engineers
- NCSEA – National Council of Structural Engineers Associations
- GBA – Geoprofessional Business Association (formerly ASFE)

Leadership positions held include:

- Chair DFI Soil Nailing and Tiebacks Committee
- Chair PDCA Technical Committee
- President of Geo-Institute
- Chair ADSC-DFI Joint Micropile Committee
- President of PDCA
- Chair Earth Retaining Structures of ASCE/G-I
- Manager DFI Technical Committees
- Received five ASTM Standards Development Awards
- Chair DFI Helical Pile Committee
- Chair DFI Codes and Standards Committee
- Director of GBA
- President of DFI
S209-16
IBC: 1810.3.3.1.

Proposed Change as Submitted

Proponent: Lori Simpson, P.E., G.E., representing GeoCoalition; Dale Biggers, P.E., representing GeoCoalition (dbiggers@bobhros.com); Daniel Stevenson, P.E., representing GeoCoalition; E. Anna Sellountou, PhD, PE, representing GeoCoalition

2015 International Building Code
Revise as follows:

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 1810.3.3.1.9, where applicable. The load test shall not be required when waived by the registered design professional based on applicable experience and data or a factor of safety of not less than three for the geotechnical capacity. The balance of deep foundation elements shall be deemed to have capacities equal to that of the test elements where such elements are of the same type, size and relative lengths 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 of such elements is not more than that of the test elements driven with the same hammer through a comparable driving distance.

Reason: The phrase “where applicable” was added because some of the referenced sections are not applicable to some deep foundation types. The load test waiver provision was added to cover the case where extensive local knowledge and experience exist, backed up by nearby load test data.

The last sentence (see original sentence below) is moved from 1810.3.3.1.2 “Compression Load Tests” to this section herein, as it applied not only to compression load tests, but to tension tests as well. The bold phrases in the original sentence below were revised for clarity.

Original from section 1810.3.3.1.2: 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.

Click here to view the members of the GeoCoalition who developed this proposal (http://www.piledrivers.org/files/8e1f877f-e55e-438e-9896-97d962b138b3--8e2ea8a9-bafa-4bd8-9d45-16f92e11d37c/members-of-the-geocoalition.pdf)

Cost Impact: Will not increase the cost of construction
The code change proposal will not increase the cost of construction because it does not add any new requirements. It may actually decrease the cost of construction if the load test can be waived based on previous experience.

Public Hearing Results

Committee Action: Disapproved
Committee Reason: The language that is proposed would allow a registered design professional to waive requirements and take the building official out of the equation. It is difficult to see how this revised section would work with the other section. The current text of this section does not address load test and adding such language is not appropriate.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:
Proponent: E. Anna Sellountou, PhD, PE, representing GeoCoalition (asellountou@pile.com); Dale Biggers, P.E. (dbiggers@bohbros.com); Daniel Stevenson, P.E. (dstevenson@berkelapg.com) requests Approve as Modified by this Public Comment.

Replace Proposal as Follows:

2015 International Building Code

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 1810.3.3.1.9. The load test requirement shall not apply where approved by the building official based on applicable local experience and data.

Commenter's Reason: The load test waiver provision is at the discretion of the building official and was added to cover the case where extensive local knowledge and experience exist, backed up by nearby load test data.

Cost - The code change proposal will not increase the cost of construction. It may actually decrease the cost of construction if the load test can be waived based on previous experience.

Click here to view the 35 members of the GeoCoalition. (http://www.piledrivers.org/geocoalition-members/)

These 35 GeoCoalition members are structural and geotechnical engineers and contractors from across the country. They are in leadership positions of more than nine organizations including:

- DFI – Deep Foundations Institute
- PDCA – Pile Driving Contractors Association
- ADSC – Association of Drilled Shaft Contractors
- ASCE – American Society of Civil Engineers
- ASTM – American Society of Testing Materials
- ACI – American Concrete Institute
- SAME – Society of American Military Engineers
- NCSEA – National Council of Structural Engineers Associations
- GBA – Geoprofessional Business Association (formerly ASFE)

Leadership positions held include:

- Chair DFI Soil Nailing and Tiebacks Committee
- Chair PDCA Technical Committee
- President of Geo-Institute
- Chair ADSC-DFI Joint Micropile Committee
- President of PDCA
- Chair Earth Retaining Structures of ASCE/G-I
- Manager DFI Technical Committees
- Received five ASTM Standards Development Awards
- Chair DFI Helical Pile Committee
- Chair DFI Codes and Standards Committee
- Director of GBA
- President of GBA
S218-16
IBC: 1810.3.3.1.9.

Proposed Change as Submitted

Proponent: Lori Simpson, P.E., G.E., representing GeoCoalition; Dale Biggers, P.E., representing GeoCoalition (dbiggers@bohbros.com); Daniel Stevenson, P.E., representing GeoCoalition; E. Anna Sellountou, PhD, PE, representing GeoCoalition

2015 International Building Code

Revise as follows:

1810.3.3.1.9 Helical piles. The allowable axial design load, $P_A$, of helical piles shall be not exceed the allowable resistance of the pile's structural elements including the pile shaft, pile shaft couplings, and the helical bearing plates. The allowable axial design load, $P_A$, also shall not exceed the allowable geotechnical resistance determined through the load testing requirements of 1810.3.3, and as follows:

$$P_A = 0.5 P_U \quad \text{(Equation 18-4)}$$

where $P_U$ is the least value of:

1. Sum of the areas of the helical bearing plates times the ultimate bearing capacity of the soil or rock comprising the bearing stratum.
2. Ultimate capacity determined from well-documented correlations with installation torque.
3. Base resistance plus shaft resistance of the helical piles, where the base resistance is equal to the sum of the areas of the helical bearing plates times the ultimate bearing resistance of the soil or rock comprising the bearing stratum, and shaft resistance is equal to the frictional resistance of the soil multiplied by the shaft area above the helical bearing plates.
4. Ultimate capacity determined from load test well-documented correlations with installation torque. The installation torque shall not exceed the manufacturer's rated torque of the shaft, couplers, and helix plates.
5. Ultimate axial capacity of pile shaft.
6. Ultimate axial capacity of pile shaft couplings.
7. Sum of the ultimate axial capacity of helical bearing plates affixed to pile.

Reason: The additional language added to the first sentence covers the requirements of the listed items 4, 5, and 6. Reorganizing this section in this way allows the geotechnical capacity limits of items 1 and 2 to be separated from the mechanical (structural) limits of items 4, 5, and 6. The mechanical axial limits are already covered in Table 1810.3.2.6, and combined stresses and buckling limits are covered by other design guides and methods. This reorganization allows this provision to focus on the geotechnical capacity which is what it is really intended to do. Furthermore, the existing language can be confusing since the term “ultimate capacity” is one that seems to be avoided in many structural design guides that seem to prefer terms such as nominal resistance, design strength, and allowable resistance. Item 3 has also been stricken from the listed items and incorporated into the first sentence since load testing is already covered in the load testing requirements of 1810.3.3.

Item 1 has been revised to include the effects of skin friction along the length of the pile shaft. Helical piles are end bearing elements and most often, any contribution of skin friction is neglected. In circumstances where there is down drag, neglecting this skin friction is unconservative and should be included in the design. There are also some circumstances with larger diameter helical piles where the designer may desire to include the effects of skin friction and the addition of this language allows for the designer to utilize that skin friction at their option.

Item 2 has been revised to include the limits of the manufacturer’s rated torque. This is not a new requirement and already appears in 1810.4.11 and has been added here again for additional clarity.

Cost Impact: Will not increase the cost of construction
The code change proposal will not increase the cost of construction since it is a clarification to the code.

2016 ICC PUBLIC COMMENT AGENDA
Committee Action: Disapproved

Committee Reason: There is concern that the proposed language does not use allowable capacity, but instead refers to resistance could be in conflict with other code sections. In removing the itemized list it appears to ignore limitation on axial capacity of these elements, only referring to their torsional capacity. This would require testing of each of these piles to establish limits of axial capacity.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Dale Biggers, P.E., representing GeoCoalition (dbiggers@bohbros.com) requests Approve as Modified by this Public Comment.

Replace Proposal as Follows:

2015 International Building Code

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

\[ P_a = 0.5 P_u \]  

(Equation 18-4)

where \( P_u \) is the least value of:

1. Sum of the areas of the helical bearing plates times the ultimate bearing capacity of the soil or rock comprising the bearing stratum.
2. Base resistance plus shaft resistance of the helical pile, where the base resistance is equal to the sum of the areas of the helical bearing plates times the ultimate bearing resistance of the soil or rock comprising the bearing stratum, and shaft resistance is equal to the frictional resistance of the soil times the shaft area above the helical bearing plates.
3. Ultimate capacity determined from well-documented correlations with installation torque.
4. Ultimate capacity determined from load tests when required by Section 1810.3.3.
5. Ultimate axial capacity of pile shaft.
6. Ultimate axial capacity of pile shaft couplings.
7. Sum of the ultimate axial capacity of helical bearing plates affixed to pile.

Commenter’s Reason:

- Item 1: Larger helical pile elements are ever more common and shaft friction can play an important role for larger shaft diameters. This addition allows for the shaft friction to be taken into account.
- Item 3: This item has been misinterpreted to require load tests. Load testing is costly for small residential projects where helical piles are often used. The requirements for load testing of all piles is covered in Section 1810.3.3.

COST IMPACT: will not add costs

This change will reduce the cost of construction for those helical piles where shaft friction can be taken into account and for those situations which do not require a load test.

Click here to view the 35 members of the GeoCoalition. (http://www.piledrivers.org/geocoalition-members/)

These 35 GeoCoalition members are structural and geotechnical engineers and contractors from across the country. They are in leadership positions of more than nine organizations including:

- DFI – Deep Foundations Institute
- PDCA – Pile Driving Contractors Association
- ADS – Association of Drilled Shaft Contractors
- ASCE – American Society of Civil Engineers
- ASTM – American Society of Testing Materials
- ACI – American Concrete Institute
- SAME – Society of American Military Engineers
- NCSEA – National Council of Structural Engineers Associations
- GBA – Geoprofessional Business Association (formerly ASFE)

Leadership positions held include:
Proponent: Amy Cerato, University of Oklahoma, representing University of Oklahoma (acerato@ou.edu) requests Approve as Submitted.

Commenter’s Reason: Will improve quality.

Proponent: Nicholas Farkas, representing Grip-Tite Mfg. Co., LLC (njfarkas@aol.com) requests Approve as Submitted.

Commenter’s Reason: This proposal will lead to improved quality and better end-user costs for the industry.

Proponent: David Frink, representing Ideal Manufacturing requests Approve as Submitted.

Commenter’s Reason: Will provide for more accurate, cost-effective design.

Proponent: Howard Perko, Magnum Piering, Inc., representing Magnum Piering, Inc. (hperko@magnumgeo.com) requests Approve as Submitted.

Commenter’s Reason: The proposed model code amendment is important for two reasons: 1.) It allows shaft resistance to be added to the total resistance of the pile which will result in overall construction cost reduction for large commercial helical pile projects. 2.) It attempts to clarify that load testing is only required per 1810.3.3.

Proponent: Gary Seider, Hubbell Power Systems, Inc., representing Deep Foundations Institute - Helical Piles & Tiebacks Committee (glseider@hubbell.com) requests Approve as Submitted.

Commenter’s Reason: Reduce overall cost of helical piles for public benefit. It will also improve quality.
Committee Action: Disapproved

Assembly Action: None

S220-16
IBC: 1810.3.4.

Proposed Change as Submitted

Proponent: Daniel Stevenson, P.E., representing GeoCoalition; E. Anna Sellountou, PhD,PE, representing GeoCoalition; Lori Simpson, P.E., G.E., representing GeoCoalition; Dale Biggers, P.E., representing GeoCoalition (dbiggers@bohbros.com)

2015 International Building Code

Revise as follows:

1810.3.4 Subsiding soils. Where deep foundation elements are installed through subsiding soils (native soil, fill, or other subsiding strata manmade material) and derive support from underlying firmer materials, consideration shall be given to the downward frictional forces that may be imposed on the elements by the subsiding upper strata.

Where the influence of subsiding soils is considered as imposing loads on the element, the allowable stresses specified in this chapter shall be permitted to be increased near the neutral plane, where satisfactory substantiating data are analysis is submitted by the registered design professional.

Reason:

- Subsiding material includes more than just fill.
- The neutral plane is where the increase is applicable.
- **Definition of Neutral Plane:** The location where equilibrium exists between the sum of sustained compression load plus drag force and the sum of mobilized positive shaft resistance and mobilized toe resistance. The neutral plane is also where relative movement between the pile and soil is zero.
- The substantiation of increase should be done by the registered design professional.

Commentary Suggestion: “Foundation Engineering Handbook” by Fang 1991, for example, uses a maximum of 0.7 of the pile strength for the allowable stress limit near the neutral plane. The registered design professional needs to substantiate the actual allowable stress, which may be a factor less than 0.7.

Click here to view the members of the GeoCoalition who developed this proposal (http://www.piledrivers.org/files/8e1f877f-e55e-438b-9896-0d9d2b138b3--8e2ea8a9-bafa-4bd8-9d45-16f92e11d37c/members-of-the-geocoalition.pdf)

Cost Impact: Will not increase the cost of construction

The change has no cost impact because it is only a clarification of the code.

S220-16:
1810.3.4 -
STEVENSON10930

Public Hearing Results

Committee Reason: The proposal needs to include a definition or other means to apply the phrase “near the neutral” which is not established by the proposed test. Adding “by the registered design professional” is considered redundant. A definition referred to in the proponent’s reason statement could possible by used.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Lori Simpson, P.E., G.E., representing GeoCoalition (lsimpson@langan.com); Garland Likins (glikins@pile.com); E. Anna Sellountou, PhD,PE (asellountou@pile.com); Daniel Stevenson, P.E. (dstevenson@berkelapg.com); Dale Biggers, P.E. (dbiggers@bohbros.com) requests Approve as Modified by this Public Comment.

Replace Proposal as Follows:

2015 International Building Code
1810.3.4 Subsiding soils. Where deep foundation elements are installed through subsiding fills, soils (native soil, fill, or other subsiding strata, manmade material) and derive support from underlying firmer materials, consideration shall be given to the downward frictional forces that may be imposed on the elements by the subsiding upper strata.

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

Commenter's Reason:
1. The title of the section is Subsiding soils - changes are made to match the title.
2. In addition, subsiding materials includes more than just fill - the changes include native soil and manmade material that can also subside.
3. This change eliminates the previous proposal to include evaluation of the neutral plane.

Cost impact: Will not increase the cost of construction. The change has no cost impact because it is only a clarification of the code.

Click here to view the 35 members of the GeoCoalition. (http://www.piledrivers.org/geocoalition-members/)

These 35 GeoCoalition members are structural and geotechnical engineers and contractors from across the country. They are in leadership positions of more than nine organizations including:

- DFI – Deep Foundations Institute
- PDCA – Pile Driving Contractors Association
- ADSC – Association of Drilled Shaft Contractors
- ASCE – American Society of Civil Engineers
- ASTM – American Society of Testing Materials
- ACI – American Concrete Institute
- SAME – Society of American Military Engineers
- NCSEA – National Council of Structural Engineers Associations
- GBA – Geoprofessional Business Association (formerly ASFE)

Leadership positions held include:
- Chair DFI Soil Nailing and Tiebacks Committee
- Chair PDCA Technical Committee
- President of Geo-Institute
- Chair ADSC-DFI Joint Micropile Committee
- President of PDCA
- Chair Earth Retaining Structures of ASCE/G-I
- Manager DFI Technical Committees
- Received five ASTM Standards Development Awards
- Chair DFI Helical Pile Committee
- Chair DFI Codes and Standards Committee
- Director of GBA
- President of DFI
Proposed Change as Submitted

Proponent: Daniel Stevenson, P.E., representing GeoCoalition; Lori Simpson, P.E., G.E., representing GeoCoalition; Dale Biggers, P.E., representing GeoCoalition (dbiggers@bohbros.com); E. Anna Sellountou, PhD, PE, representing GeoCoalition

2015 International Building Code

Revise as follows:

1810.3.5.3.4 Steel pipes and tubes. Steel pipes and tubes used as deep foundation elements shall have a nominal outside diameter of not less than 6 inches (152 mm), unless approved by the registered design professional. When filling is required for diameters less than 8 inches (203 mm), grout shall be used. Where steel pipes or tubes are driven open ended, they shall have a minimum of 0.34 square inch (219 mm²) of steel in cross section to resist each 1,000 foot-pounds (1356 Nm) of pile hammer rated 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 steel 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, caisson piles shall have a nominal outside diameter of not less than 18 inches (457 mm), a wall thickness of not less than 5 / 8 inch (9.5 mm) and a suitable steel driving shoe welded to the bottom shall be provided; the diameter of the rock socket shall be approximately equal to the inside diameter of the casing or as determined by the registered design professional.

Exceptions:
1. There is no minimum diameter for steel pipes or tubes used in micropiles or helical piles.
2. For mandrel-driven pipes or tubes (not including corrugated pile shells), the minimum wall thickness shall be 1 / 10 inch (2.5 mm).

Reason: The purpose of this change is to clarify the Code and to substitute revised material for current provisions of the Code.

- Smaller (6-inch-diameter) piles are often used for lightly loaded structures.
- For lightly loaded deep foundation elements, pipe diameters smaller than 6 inches may be sufficient.
- When small diameter pipes are filled, grout shall be used rather than concrete to prevent bridging.
- "Rated energy" (as opposed to kinetic energy or energy transferred to the pile) is added for clarity.
- Hyphen is added to "open-ended" for correct punctuation.
- "Steel shoe" is a more generic term vs. cutting shoe or driving shoe. It also adds consistency.
- The term "caisson piles" used to eliminate confusion with drilled shafts (to differentiate this particular type of foundation from an ordinary drilled shaft).
- Rock sockets may have a diameter that is different than the inside diameter of the casing.
- Diameters for helical piles are covered in section 1810.3.5.3.5.
- Mandrel-driven corrugated shells are covered in section 1810.3.2.7.

Click here to view the members of the GeoCoalition who developed this proposal (http://www.piledrivers.org/files/8e1f877f-e55e-438e-9896-97d962b138b3--8e2ea8a9-bafa-4bd8-9d45-16f92e11d37c/members-of-the-geocoalition.pdf)

Cost Impact: Will not increase the cost of construction

The code change proposal will not increase the cost of construction as it is mainly a clarification. The change of the minimum diameter will reduce costs in construction by allowing smaller diameter piles on lightly loaded structures.

Public Hearing Results

Committee Action: Disapproved

Committee Reason: There is concern with using the hammer's "rated" energy since the code should be less concerned about what it is rated and more about what it delivers to the pile. Also caisson is not defined and would like to see it defined before adding it to this section.
Public Comment 1:

Proponent: Dale Biggers, P.E., representing GeoCoalition (dbiggers@bohbros.com) requests Approve as Modified by this Public Comment.

Replace Proposal as Follows:

**2015 International Building Code**

1810.3.5.3.4 Steel pipes and tubes. Steel pipes and tubes used as deep foundation elements shall have a nominal outside diameter of not less than 6 inches (203 mm). Where steel pipes or tubes are driven open ended, they shall have a minimum of 0.34 square inch (219 mm²) 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 \( \frac{3}{16} \) 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 \( \frac{3}{8} \) 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. For mandrel-driven pipes or tubes, the minimum wall thickness shall be \( \frac{1}{10} \) inch (2.5 mm).

**Commenter's Reason:** Smaller (6-inch) pipe piles are often appropriate for lightly loaded structures.

Cost Impact: Will not increase the cost of construction.

This will reduce the cost by allowing a smaller diameter on lightly loaded structures.

Click here to view the 35 members of the GeoCoalition (http://www.piledrivers.org/geocoalition-members/).

These 35 GeoCoalition members are structural and geotechnical engineers and contractors from across the country. They are in leadership positions of more than nine organizations including:

- DFI – Deep Foundations Institute
- PDCA – Pile Driving Contractors Association
- ADSC – Association of Drilled Shaft Contractors
- ASCE – American Society of Civil Engineers
- ASTM – American Society of Testing Materials
- ACI – American Concrete Institute
- SAME – Society of American Military Engineers
- NCSEA – National Council of Structural Engineers Associations
- GBA – Geoprofessional Business Association (formerly ASFE)

Leadership positions held include:

- Chair DFI Soil Nailing and Tiebacks Committee
- Chair PDCA Technical Committee
- President of Geo-Institute
- Chair ADSC-DFI Joint Micropile Committee
- President of PDCA
- Chair Earth Retaining Structures of ASCE/G-I
- Manager DFI Technical Committees
- Received five ASTM Standards Development Awards
- Chair DFI Helical Pile Committee
- Chair DFI Codes and Standards Committee
- Director of GBA
- President of DFI
Proposed Change as Submitted

Proponent: Daniel Stevenson, P.E., representing GeoCoalition; Lori Simpson, P.E., G.E., representing GeoCoalition; Dale Biggers, P.E., representing GeoCoalition (dbiggers@bohbros.com); E. Anna Sellountou, PhD, PE, representing GeoCoalition

2015 International Building Code

Revise as follows:

1810.3.6 Splices. Splices shall be designed or selected by the registered design professional to resist the axial and shear forces and moments occurring at the location of the splice during installation and for the design load combinations. Splices shall be constructed so as to provide and maintain true alignment and position of the component parts of the deep foundation element during installation and subsequent thereto and shall be designed to resist the axial and shear forces and moments occurring at the location of the splice during driving and for design load combinations. Where deep foundation elements of the same type are being spliced, splices shall develop not less than 50 percent of the bending strength of the weaker section. Where deep foundation elements of different materials or different types are being spliced, splices shall develop the full compressive strength and not less than 50 percent of the tension and bending strength of the weaker section. Where structural steel cores are to be spliced, the ends shall be milled or ground to provide full contact and shall be full-depth welded. Splices occurring in the upper 10 feet (3048 mm) of the embedded portion of an element shall be designed to resist at allowable stresses the moment and shear that would result from an assumed eccentricity of the axial load of 3 inches (76 mm), or the element shall be braced in accordance with Section 1810.2.2 to other deep foundation elements that do not have splices in the upper 10 feet (3048 mm) of embedment.

Reason: The proposed code change replaces an arbitrary criteria requirement with a requirement that has a rational basis. The first sentence is rearranged to make the registered design professional responsible for the design of the splice. It also replaces “driving” with “installation” for consistency. The requirements that splices develop not less than 50 per cent of the bending strength or not less than 50 per cent of the tension and bending strength are arbitrary and unnecessarily restrictive. The revised first two sentences will result in sufficient splice strength as appropriate for the application service loads and depth of the splice, as determined by the registered design professional. The current specification precludes commonly available splices that would be acceptable in many design situations, such as a splice located at significant depth. The design professional will analyze the pile and soil strength conditions for lateral and tension or compression axial loadings to determine the requirements for a splice. The existing Commentary describes a drive-fit splice for pipe piles and wood-to-wood splices that do not satisfy the current code. The last phrase is overly prescriptive and is removed because the bracing requirements are covered adequately by Section 1810.2.2, and will be further evaluated by the registered design professional.

Click here to view the members of the GeoCoalition who developed this proposal (http://www.piledrivers.org/files/8e1f877f-e55e-438e-9896-97d962b138b3--8e2ea8a9-bafa-4bd8-9d45-16f92e11d37c/members-of-the-geocoalition.pdf)

Cost Impact: Will not increase the cost of construction
The proposed code change will not increase the cost of construction. In certain cases, it will decrease the cost by allowing more economical splices that still satisfy the service loading.

Committee Action: Disapproved

Committee Reason: The committee feels it is not appropriate to remove the minimum splice provision. Perhaps consider a public comment that allows designed splices, but doesn't remove the minimum criteria. The proposed wording would require all splices to be designed. In addition it specifically requires design for forces "at the location of the splice" but typically you do not known where splices will occur.

Assembly Action: None

Individual Consideration Agenda
Public Comment 1:

Proponent: Dale Biggers, P.E., representing GeoCoalition (dbiggers@bohbros.com) requests Approve as Modified by this Public Comment.

Replace Proposal as Follows:

2015 International Building Code

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 designed to resist the axial and shear forces and moments occurring at the location of the splice during driving and for design load combinations. Where deep foundation elements of the same type are being spliced, splices shall develop not less than 50 percent of the bending strength of the weaker section. Where deep foundation elements of different materials or different types are being spliced, splices shall develop the full compressive strength and not less than 50 percent of the tension and bending strength of the weaker section. Use of splices with strengths less than 50 percent shall be permitted where supporting data justifying such lesser strengths is approved by the building official. Such substantiating data shall be as required by Section 1810.3.2.8. 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 designed to resist at allowable stresses the moment and shear that would result from an assumed eccentricity of the axial load of 3 inches (76 mm), or the element shall be braced in accordance with Section 1810.2.2 to other deep foundation elements that do not have splices in the upper 10 feet (3048 mm) of embedment.

Commenter’s Reason:

1.) The requirements that splices develop 50 percent are arbitrary and unnecessarily restrictive.

2.) The current specification precludes commonly available splices that are acceptable in many situations, such as splices located at significant depth.

3.) In locations (such as Louisiana) where even 240-foot long piles are friction piles, the depth of the splice is known in advance.

4.) The piling cost increased $1,510,000 for the VA Hospital and $520,000 for the Prison only because the current code requires a more expensive splice than the splice used successfully on the New Orleans Superdome, 52-story Shell Square, 50-story Sheraton Hotel, and many other structures.

5.) Section 1810.3.2.8 is referenced because it gives requirements, including geotechnical investigations and load tests, which must be submitted when requesting the approval of the building official.

6.) In splices with tension, seismic, or other considerations, there will be no change from the current splice strength requirements.

COST IMPACT: Will not increase the cost of construction.

The proposed change will greatly decrease the cost of construction in some areas of the country.

Click here to view the 35 members of the GeoCoalition. (http://www.piledrivers.org/geocoalition-members/)

These 35 GeoCoalition members are structural and geotechnical engineers and contractors from across the country. They are in leadership positions of more than nine organizations including:

- DFI – Deep Foundations Institute
- PDCA – Pile Driving Contractors Association
- ADSC – Association of Drilled Shaft Contractors
- ASCE – American Society of Civil Engineers
- ASTM – American Society of Testing Materials
- ACI – American Concrete Institute
- SAME – Society of American Military Engineers
- NCSEA – National Council of Structural Engineers Associations
- GBA – Geoprofessional Business Association (formerly ASFE)

Leadership positions held include:

- Chair DFI Soil Nailing and Tiebacks Committee
- Chair PDCA Technical Committee
- President of Geo-Institute
o Chair ADSC-DFI Joint Micropile Committee
o President of PDCA
o Chair Earth Retaining Structures of ASCE/G-I
o Manager DFI Technical Committees
o Received five ASTM Standards Development Awards
o Chair DFI Helical Pile Committee
o Chair DFI Codes and Standards Committee
o Director of GBA
o President of DFI
Committee Action: Disapproved

Assembly Action: None

S231-16

IBC: 1810.4.1.2.

Proposed Change as Submitted

Proponent: E. Anna Sellountou, PhD, PE, representing GeoCoalition; Lori Simpson, P.E., G.E., representing GeoCoalition; Dale Biggers, P.E., Boh Bros. Construction Co., L.L.C., representing GeoCoalition (dbiggers@bohbros.com); Daniel Stevenson, P.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

1810.4.1.2 Casing Shafts in unstable soils. 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 stabilized prior to placing the concrete. Where by a casing, suitable slurry, or other method approved by the registered design professional. Where a 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.

Reason:
• The title is changed because the focus of the section is shafts in unstable soils.
• The words “concrete is placed in an open drilled hole” are redundant since the sentence begins with the context of a cast-in-place element, and furthermore the hole is not really an open hole if a casing or slurry is in place.
• There are other commonly used means of stabilizing unstable soils besides casing, such as the use of drilling slurry.
• Driven casings are almost never mandrel driven. Mandrel driven piles are covered elsewhere.

Click here to view the members of the GeoCoalition who developed this proposal (http://www.piledrivers.org/files/8e1f877f-e55e-438e-9896-97d962b138b3--8e2ea8a9-bafa-4bd8-9d45-16f92e11d37c/members-of-the-geocoalition.pdf)

Cost Impact: Will not increase the cost of construction

The change will not increase costs and in some cases will reduce costs by allowing alternate construction techniques.

2016 ICC PUBLIC COMMENT AGENDA

Public Hearing Results

Committee Reason: This proposal would allow approval by the registered design professional, removing the building official from the entire process. There would be some benefit to the rest of the proposal that provides suitable methods for stabilizing the hole.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Lori Simpson, P.E., G.E., representing GeoCoalition (lsimpson@langan.com); Dale Biggers, P.E. (dbiggers@bohbros.com); Daniel Stevenson, P.E. (dstevenson@berkelapq.com); E. Anna Sellountou, PhD, PE (asellountou@pile.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

1810.4.1.2 Shafts in unstable soils. Where cast-in-place deep foundation elements are formed through unstable soils, the open hole shall be stabilized prior to placing the concrete by a casing, suitable slurry, or other method approved by the registered design professional building official. Where a 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.

Commenter's Reason:
1. There are other commonly used means of stabilizing unstable soils besides casing, such as the use of drilling slurry. The Louisville panel recognized the benefit of this proposal in their “Committee Reason”.
2. Based on the Louisville “Committee Reason”, the “registered design professional” was replaced by the “building official”.
3. The title is changed to “Shafts in unstable soils” because the focus of this section is stabilizing shafts in unstable soils.
Previous title was “Casings”, but there are other ways to stabilize the hole as this proposal suggests.

4. The current code version says in the first sentence “concrete is placed in an open drilled hole”. Those words are redundant, and thus removed, because the sentence begins with the context of a cast-in-place element.

5. The last sentence of the current code version was removed because casings are never mandrel driven. Mandrel driven piles are covered elsewhere.

Costs
The change will not increase costs and in some cases will reduce costs by allowing alternate construction techniques.

Committee Reason: This proposal would allow approval by the registered design professional, removing the building official from the entire process. There would be some benefit to the rest of the proposal that provides suitable methods for stabilizing the hole.

Click here to view the 35 members of the GeoCoalition. (http://www.piledrivers.org/geocoalition-members/)

These 35 GeoCoalition members are structural and geotechnical engineers and contractors from across the country. They are in leadership positions of more than nine organizations including:

- DFI – Deep Foundations Institute
- PDCA – Pile Driving Contractors Association
- ADSC – Association of Drilled Shaft Contractors
- ASCE – American Society of Civil Engineers
- ASTM – American Society of Testing Materials
- ACI – American Concrete Institute
- SAME – Society of American Military Engineers
- NCSEA – National Council of Structural Engineers Associations
- GBA – Geoprofessional Business Association (formerly ASFE)

Leadership positions held include:

- Chair DFI Soil Nailing and Tiebacks Committee
- Chair PDCA Technical Committee
- President of Geo-Institute
- Chair ADSC-DFI Joint Micropile Committee
- President of PDCA
- Chair Earth Retaining Structures of ASCE/G-I
- Manager DFI Technical Committees
- Received five ASTM Standards Development Awards
- Chair DFI Helical Pile Committee
- Chair DFI Codes and Standards Committee
- Director of GBA
- President of DFI
Committee Action: Disapproved

Assembly Action: None

S232-16
IBC: 1810.4.1.3.

Proposed Change as Submitted

Proponent: E. Anna Sellountou, PhD, PE, representing GeoCoalition; Lori Simpson, P.E., G.E., representing GeoCoalition; Dale Biggers, P.E., Boh Bros. Construction Co., L.L.C., representing GeoCoalition (dbiggers@bohbros.com); Daniel Stevenson, P.E., representing GeoCoalition

2015 International Building Code
Revise as follows:

1810.4.1.3 Driving near uncased concrete. Deep foundation elements shall not be driven within less than six element diameters center to center in granular soils or within less than one-half the element length in cohesive soils of an uncased element filled with concrete less than 48 hours old unless approved by the building official. If the concrete surface or grout level in any completed element rises is observed to rise or drops, drop, bubble, bleed water, etc. due to installation of an adjacent element, the completed element shall be replaced. Driven uncased deep foundation elements shall not be installed in soils that could cause heave unless considered by the registered design professional.

Reason: The changes in wording only clarify the distance; they do not change the distance. There are other possible areas of concern in addition to a change of elevation. This delineates who is responsible for consideration of heave and the determination of how much heave is acceptable. It is not clear what a driven uncased element is.

Cost Impact: Will not increase the cost of construction
This proposal will not increase the cost of construction. It does not change the current guidelines and it only calls attention to conditions that should already be under consideration.

Committee Reason: This proposal contains permissive language rather than mandatory requirements. The proposed wording contains useful information if it were rewritten, but as drafted it reads like a design guide more than code language.

Assembly Action: None

Public Hearing Results

Public Comment 1:

Proponent: E. Anna Sellountou, PhD, PE, representing GeoCoalition (asellountou@pile.com); Garland Likins, representing GeoCoalition (glikins@pile.com); Dale Biggers, P.E. (dbiggers@bohbros.com); Lori Simpson, P.E., G.E. (lsimpson@langan.com); Daniel Stevenson, P.E. (dstevenson@berkelapg.com) requests Approve as Modified by this Public Comment.

Replace Proposal as Follows:

2015 International Building Code

1810.4.1.3 Driving near uncased concrete. Deep foundation elements shall not be driven within six element diameters center...
to center in granular soils or within one-half the element length in cohesive soils of an uncased element filled with concrete less than 48 hours old unless approved by the building official. If the concrete or grout surface in any completed element rises is observed to rise, drop, bubble, or drops bleed water due to installation of an adjacent element, the completed element shall be replaced. Driven uncased deep foundation elements shall not be installed in soils that could cause heave.

Commenter's Reason:
1. There are other possible areas of concern in addition to a change of elevation of the top surface of a completed element.
2. It does not change the current guidelines and it only calls attention to conditions that should already be under consideration.
3. The proposal also clarifies that the completed element is the one to be replaced.
4. Some uncased concrete elements, like augercast piles, are constructed with grout and not concrete.

Cost - This change will not increase the cost of construction. In good construction practice this would have been done anyway.

Click here to view the 35 members of the GeoCoalition. (http://www.piledrivers.org/geocoalition-members/)

These 35 GeoCoalition members are structural and geotechnical engineers and contractors from across the country. They are in leadership positions of more than nine organizations including:

- DFI – Deep Foundations Institute
- PDCA – Pile Driving Contractors Association
- ADSC – Association of Drilled Shaft Contractors
- ASCE – American Society of Civil Engineers
- ASTM – American Society of Testing Materials
- ACI – American Concrete Institute
- SAME – Society of American Military Engineers
- NCSEA – National Council of Structural Engineers Associations
- GBA – Geoprofessional Business Association (formerly ASFE)

Leadership positions held include:

- Chair DFI Soil Nailing and Tiebacks Committee
- Chair PDCA Technical Committee
- President of Geo-Institute
- Chair ADSC-DFI Joint Micropile Committee
- President of PDCA
- Chair Earth Retaining Structures of ASCE/G-I
- Manager DFI Technical Committees
- Received five ASTM Standards Development Awards
- Chair DFI Helical Pile Committee
- Chair DFI Codes and Standards Committee
- Director of GBA
- President of DFI
Committee Action: Disapproved

Assembly Action: None

S235-16
IBC: 1810.4.11.

Proposed Change as Submitted

Proponent: E. Anna Sellountou, Ph.D., P.E., representing GeoCoalition; Lori Simpson, P.E., G.E., representing GeoCoalition; Dale Biggers, P.E., Boh Bros. Construction Co., L.L.C., representing GeoCoalition (dbiggers@bohbros.com); Daniel Stevenson, P.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

1810.4.11 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 manufacturer's rated installation torque of the helical pile as established in accordance with criteria acceptable to the registered design professional.

Reason: The term "manufacturer's rated installation torque" is the common term used in the industry and is more consistent with the language used in a manufacturer's evaluation report (ESR) and in the acceptance criteria AC358 published by the ICC-ES. The requirements for determining the maximum installation torque are contained in AC358. These documents prefer not to use the term "allowable". It also clarifies that the source of the maximum installation torque comes from the testing requirements used to generate the ESR as opposed to something that needs to be specified by the registered design professional.

Click here to view the members of the GeoCoalition who developed this proposal (http://www.piledrivers.org/files/8e1f877f-e55e-438e-9896-97d962b138b3--8e2ea8a9-bafa-4bd8-9d45-16f92e11d37c/members-of-the-geocoalition.pdf)

Cost Impact: Will not increase the cost of construction

The code change proposal will not increase the cost of construction since it is a clarification to the code.

Public Hearing Results

Committee Action: Disapproved

Committee Reason: This proposal causes some confusion with the intent of replacing "maximum allowable" with "manufacturer's rated" torque. The wording added at the end is considered ambiguous and unnecessary. There was a some feeling that this change to the manufacturer's rated torque would be akin to driven piles going up to 90 percent of yield and if the wording can be corrected in a public comment it would be a useful change.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Dale Biggers, P.E., representing GeoCoalition (dbiggers@bohbros.com) requests Approve as Modified by this Public Comment.

Replace Proposal as Follows:

2015 International Building Code

1810.4.11 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 manufacturer's rated installation torque of the helical pile.

Commenter's Reason: The term "manufacturer's rated installation torque" is consistent with the language used in the acceptance criteria AC 358 published by the ICC-ES.

COST IMPACT: Will not increase the cost of construction.

This is just a clarification of the code.

Click here to view the 35 members of the GeoCoalition. (http://www.piledrivers.org/geocoalition-members/)
These 35 GeoCoalition members are structural and geotechnical engineers and contractors from across the country. They are in leadership positions of more than nine organizations including:

- DFI – Deep Foundations Institute
- PDCA – Pile Driving Contractors Association
- ADSC – Association of Drilled Shaft Contractors
- ASCE – American Society of Civil Engineers
- ASTM – American Society of Testing Materials
- ACI – American Concrete Institute
- SAME – Society of American Military Engineers
- NCSEA – National Council of Structural Engineers Associations
- GBA – Geoprofessional Business Association (formerly ASFE)

Leadership positions held include:
- Chair DFI Soil Nailing and Tiebacks Committee
- Chair PDCA Technical Committee
- President of Geo-Institute
- Chair ADSC-DFI Joint Micropile Committee
- President of PDCA
- Chair Earth Retaining Structures of ASCE/G-I
- Manager DFI Technical Committees
- Received five ASTM Standards Development Awards
- Chair DFI Helical Pile Committee
- Chair DFI Codes and Standards Committee
- Director of GBA
- President of DFI

Proponent: Amy Cerato, representing University of Oklahoma (acerato@ou.edu) requests Approve as Submitted.

Commenter's Reason: Will improve quality.

Proponent: Nicholas Farkas, representing Grip-Tite Mfg. Co., LLC (njfarkas@aol.com) requests Approve as Submitted.

Commenter's Reason: This proposal will create improved quality of products and services and reduce costs in the industry.

Proponent: David Frink, representing Ideal Manufacturing requests Approve as Submitted.

Commenter's Reason: Reduces possibility of confusion on intent of code.

Proponent: Howard Perko, representing Magnum Piering, Inc. (hperko@magnumgeo.com) requests Approve as Submitted.

Commenter's Reason: The proposed code amendment clarifies that helical piles can be installed to the maximum manufacturer's rated torque which is consistent with current practice. The words maximum allowable torque are confusing. The change will reduce construction cost because it will allow piles to be installed to higher, more appropriate, maximum rated torque.

Proponent: Michael Perlow Jr, EKMLLC, representing self (mike@perlowmp.com) requests Approve as Submitted.

Commenter's Reason: Use of helical piles is increasing as a result of adoption of the ICC AC358 Acceptance Criteria and inclusion in the IBC2009. The industry through the Deep Foundations Institute (DFI) Helical Pile Tieback Committee (HPTC) efforts has moved forward on the development of helical pile standard specifications, a design guide, and applicable ASTM test standards. Most significantly, the DFI through the efforts of the HPTC has funded an industry sponsored seismic study of helical pile performance. A DFI Helical Piles & Tiebacks Seminar will be held on August 10-12 at the Ontario Airport Hotel and Conference Center (near Los Angeles) on the "Use of Helical Piles and Tiebacks in Seismic & Lateral Load Conditions". One
of the technical presentations will be given by Dr. Amy Cerato, Rapp Foundation Presidential Professor, School of Civil Engineering and Environmental Science University of Oklahoma. Dr. Cerato will be making a presentation on the seismic loading tests that were conducted in February of 2016 at the University of California San Diego shake table facility - the largest outdoor shake table in the world. At the August seminar, Dr. Cerato will be showing a high resolution video of tests that include amazing close up footage of the loaded piles reacting to the seismic forces. She will also be presenting some of the results of pile performance and pile condition from the testing. Preliminary results indicate that helical piles can be very effective in providing foundation support under seismic loading. See - https://vimeo.com/167773622. Completion of the DFI HPTC seismic research study will involve instrumented piles in active seismic areas and a comprehensive research report that could greatly expand the use of helical piles as a mitigation-retrofit foundation system.

The recommended changes are a critical building block to improving the IBC requirements and standard of practice for the increasing use of helical piles as a drilled deep foundation system.

**Proponent:** Gary Seider, representing Deep Foundations Institute - Helical Piles & Tiebacks Committee (glseider@hubbell.com) requests Approve as Submitted.

**Commenter's Reason:** Proposed modification will eliminate confusion of terms regarding installation torque. This change can reduce cost of construction.
S236-16
IBC: 1810.4.13 (New)

Proposed Change as Submitted

Proponent: E. Anna Sellountou, Ph.D., PE, representing GeoCoalition; Lori Simpson, P.E., G.E., representing GeoCoalition; Dale Biggers, P.E., Boh Bros. Construction Co., L.L.C., representing GeoCoalition (dbiggers@bohbros.com); Daniel Stevenson, P.E., representing GeoCoalition

2015 International Building Code

Add new text as follows:

1810.4.13 Integrity testing. Where required by a registered design professional, a representative number of deep foundation elements shall be tested for structural integrity in accordance with either ASTM D4945, ASTM D5882, ASTM D6760, ASTM D7949, or by a generally accepted method approved by the building official.

Reference standards type: This reference standard is new to the ICC Code Books

Add new standard(s) as follows:
ASTM D6760-14 Standard Test Method for Integrity Testing of Concrete Deep Foundations by Ultrasonic Crosshole Testing
ASTM D7949-14 Standard Test Methods for Thermal Integrity Profiling of Concrete Deep Foundations

Reason: To aid the registered design professional and/or the project team by highlighting that integrity testing is current practice to reduce the risk of structural failure. Some foundation types in certain soil conditions involve uncertainties in installation, and the allowable stress table 1810.3.2.6 would suggest some verification is appropriate. Some foundation types or circumstances require no integrity testing and the registered design professional is well equipped to distinguish when such testing is needed.

ASTM D6760 is “Standard Test Method for Integrity Testing of Concrete Deep Foundations by Ultrasonic Crosshole Testing.”
ASTM D7949 is “Standard Test Methods for Thermal Integrity Profiling of Concrete Deep Foundations.”

Click here to view the members of the GeoCoalition who developed this proposal (http://www.piledrivers.org/files/8e1f877fe55e-438e-9896-97d962b138b3--8e2ea8a9-bafa-4bd8-9d45-16f92e11d37c/members-of-the-geocoalition.pdf)

Cost Impact: Will not increase the cost of construction

The code change proposal will not increase the cost of construction as it is current practice to perform integrity testing on some projects.

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM D5882, ASTM D6760 & ASTM D7949, with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 1, 2016.

Public Hearing Results

Committee Action: Disapproved

Committee Reason: This proposal hasn’t established what integrity testing is or what purpose it serves in the context of the code. If it is going into the code we must establish the parameters for its use. There are concerns over wording such as “generally accepted methods” and “representative number”. This is not a requirement. If the building official can’t enforce, it doesn’t belong in the code. The registered design professional can put integrity testing requirements in the specifications.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:
Modify as Follows:

2015 International Building Code

1810.4.13 Structural Integrity Testing. Where required by a registered design professional, the building official, a representative number of the deep foundation elements shall be tested for structural integrity defects in accordance with either ASTM D4945, ASTM D5882, ASTM D6760, ASTM D7949, or by a generally accepted method approved by the building official.

Commenter's Reason: This is a proposed new section to the code with the purpose to add a requirement to the code to evaluate structural integrity (e.g., absence of defects that significantly detract from the structural strength of the element). Based on committee's verbal comments during the hearings, we understand that they viewed adding a section for integrity testing would be an improvement of the code.

1. We replaced the term "integrity testing" with the commonly known term "structural integrity" so that a definition of "integrity testing" is not needed. "Structural integrity" is further elaborated in the modified proposal by the words "the absence of structural defects".
2. We have established the parameters of its use; i.e., when the building official deems it necessary (e.g., because the integrity of the deep foundation element is in doubt due to difficult soil conditions, due to lack of redundancy, because of the importance of the structure, to approve the construction procedures, etc).
3. The building official can now require and enforce it when structural integrity is in doubt.
4. The term "generally accepted methods" is eliminated; The decision of which structural integrity evaluation method to be used resides with the building official.
5. The term "representative number" is eliminated. The building official can decide the amount of testing based on the importance of the structure, soil conditions, redundancy, and experience with the contractor's construction procedures.

ASTM D7949 is "Standard Test Methods for Thermal Integrity Profiling of Concrete Deep Foundations."

Cost - This proposal will not increase the cost of construction because it is current practice to perform integrity testing on some projects. Assuring that the foundation element has no defects will prevent subsequent remediation costs for elements that would have failed due to undetected defects, and thus produce an overall savings to the project in time and money.

Click here to view the 35 members of the GeoCoalition. (http://www.piledrivers.org/geocoalition-members/)

These 35 GeoCoalition members are structural and geotechnical engineers and contractors from across the country. They are in leadership positions of more than nine organizations including:

• DFI – Deep Foundations Institute
• PDCA – Pile Driving Contractors Association
• ADSC – Association of Drilled Shaft Contractors
• ASCE – American Society of Civil Engineers
• ASTM – American Society of Testing Materials
• ACI – American Concrete Institute
• SAME – Society of American Military Engineers
• NCSEA – National Council of Structural Engineers Associations
• GBA – Geoprofessional Business Association (formerly ASFE)

Leadership positions held include:
• Chair DFI Soil Nailing and Tiebacks Committee
• Chair PDCA Technical Committee
• President of Geo-Institute
• Chair ADSC–DFI Joint Micropile Committee
• President of PDCA
• Chair Earth Retaining Structures of ASCE/G-I
• Manager DFI Technical Committees
・Received five ASTM Standards Development Awards
・Chair DFI Helical Pile Committee
・Chair DFI Codes and Standards Committee
・Director of GBA
・President of DFI
Committee Action: Disapproved
Assembly Action: None

S238-16
IBC: 1810.4.5.

Proposed Change as Submitted

Proponent: E. Anna Sellountou, PhD,PE, representing GeoCoalition; Lori Simpson, P.E., G.E., representing GeoCoalition; Dale Biggers, P.E., Boh Bros. Construction Co., L.L.C., representing GeoCoalition (dbiggers@bohbros.com); Daniel Stevenson, P.E., representing GeoCoalition

2015 International Building Code
Revise as follows:

1810.4.5 Vibratory driving. Vibratory drivers shall only be used to fully install deep foundation elements where the element load capacity is verified by load tests in accordance with Section 1810.3.3.1.2, unless the piles are used only for lateral resistance. The installation of production elements shall be controlled according to power consumption, rate of penetration or other approved means that ensure element capacities equal or exceed those of the test elements.

Reason: Piles that are installed partially with vibratory hammer and then completed with an impact hammer have other means of verifying capacity. Piles that are installed only and fully with a vibratory hammer will still be required to have an axial load test, when axial loads are required. A load test for axial capacity is not needed for piles used for lateral resistance and would not confirm lateral performance of the pile. Lateral load testing requirements, if needed, are specified in Section 1810.3.3.2.

Cost Impact: Will not increase the cost of construction
The code change proposal will not increase the cost of construction. It can decrease the cost of construction if the piles are used only for lateral resistance.

Committee Reason: The proposed wording to “fully” install is confusing in the context of foundation element installation.

Assembly Action: None

Public Hearing Results

Public Comment 1:

Proponent: Lori Simpson, P.E., G.E., representing GeoCoalition (lsimpson@langan.com); Garland Likins (glikins@pile.com); E. Anna Sellountou, PhD,PE (asellountou@pile.com); Daniel Stevenson, P.E. (dstevenson@berkelapg.com); Dale Biggers, P.E. (dbiggers@bohbros.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

1810.4.5 Vibratory driving. Vibratory drivers shall only be used to fully install deep foundation elements where the element load capacity is verified by load tests in accordance with Section 1810.3.3.1.2, unless the pile installation is completed with an impact hammer or piles are used only for lateral resistance. The installation of production elements shall be controlled according to power consumption, rate of penetration or other approved means that ensure element capacities equal or exceed those of the test elements.

Commenter’s Reason:
1. The committee did not prefer the word “fully” so this is removed.
2. This public comment adds “unless the pile installation is completed using an impact hammer” because piles that are started using a vibratory hammer but completed using an impact hammer should be treated as piles that are installed with...
an impact hammer - the impact hammer can be used to evaluate capacity.

3. The previously submitted language “unless...the piles are used only for lateral resistance” remains as proposed language because a load test for axial capacity is not needed for piles used for lateral resistance and would not confirm lateral performance of the pile.

Cost Impact: Will not increase the cost of construction as it is only a clarification of the code. In fact, it will likely decrease cost as an axial load test will not be required where piles are only used for lateral resistance or where the pile installation is completed using an impact hammer.

Click here to view the 35 members of the GeoCoalition. (http://www.piledrivers.org/geocoalition-members/)

These 35 GeoCoalition members are structural and geotechnical engineers and contractors from across the country. They are in leadership positions of more than nine organizations including:

- DFI – Deep Foundations Institute
- PDCA – Pile Driving Contractors Association
- ADSC – Association of Drilled Shaft Contractors
- ASCE – American Society of Civil Engineers
- ASTM – American Society of Testing Materials
- ACI – American Concrete Institute
- SAME – Society of American Military Engineers
- NCSEA – National Council of Structural Engineers Associations
- GBA – Geoprofessional Business Association (formerly ASFE)

Leadership positions held include:

- Chair DFI Soil Nailing and Tiebacks Committee
- Chair PDCA Technical Committee
- President of Geo-Institute
- Chair ADSC-DFI Joint Micropile Committee
- President of PDCA
- Chair Earth Retaining Structures of ASCE/G-I
- Manager DFI Technical Committees
- Received five ASTM Standards Development Awards
- Chair DFI Helical Pile Committee
- Chair DFI Codes and Standards Committee
- Director of GBA
- President of DFI
Proposed Change as Submitted

Proponent: theodore Maynard, representing GeoCoalition (trmagm@aol.com); Lori Simpson, P.E., G.E., representing GeoCoalition

2015 International Building Code

Add new text as follows:

1804.2 Temporary earth retention system. Where a temporary earth retention system is selected to protect or support existing foundations, structures or infrastructure elements, the temporary earth retention system shall be designed, installed and removed in accordance with the provisions of Section 1811.

SECTION 1811 TEMPORARY EARTH RETENTION SYSTEMS

1811.1 Definition. A temporary earth retention system is a retaining structure used to provide support of excavation sidewalls for areas of excavation where the need for support will be eliminated within a 3 year period, unless otherwise approved by the building official.

1811.1.1 Underpinning relationship. Requirements for underpinning adjacent structures prior to excavation or for the installation of temporary earth retention systems shall be determined in accordance with Section 1803.5.7 by a registered design professional. Temporary earth retention systems are permitted in conjunction with underpinning of adjacent structures.

1811.1.2 Depth of excavation requirements. Excavations deeper than 5 feet with side slopes to steep to prevent ground movements or collapse, as determined by a registered design professional, shall be provided with a temporary earth retention system.

1811.2 Temporary Earth Retention System Design. Temporary earth retention systems shall be designed and installed in accordance with the provisions of Chapters 16 and 18. The design shall be performed and the installation/removal procedures determined by a registered design professional. The system shall be designed to limit ground movements when adjacent property may be affected.

1811.3 Temporary Earth Retention System Monitoring. Where determined by the registered design professional responsible for the temporary earth retention system design, monitoring of the system shall be performed during installation, for the life of the system, and during removal of the system. Monitoring shall be performed under the supervision of a registered design professional, using generally accepted methods, by personnel trained and experienced in the chosen methods. Monitoring shall be performed at intervals sufficient to provide information in time to mitigate impending excessive movements and protect adjacent structures and infrastructure elements.

1811.4 Wall Support Removal. Temporary earth retention system wall supports may only be removed when corresponding replacement support is provided by backfill or by the new structure. The loads on the temporary earth retention system shall only be transferred to the new structure once the new structure is capable of sustaining these loads, as determined by the registered design professional for the new structure.

1811.4.1 Retention Wall Removal. Removal shall be performed in such a manner as to protect the new structure, adjacent structures and adjacent infrastructure elements.

Reason: This proposal provides a general definition of a temporary earth retention system, a concept introduced in our proposal to add a new Section 1804.2. Temporary earth retention systems are commonly used to support walls of excavated areas during building construction to protect adjacent property and are not presently covered, as such, in the current version of the Code. The proposal describes the relationship between temporary earth retention systems and underpinning. Both concepts are described in Section 1804, as revised by our proposals, as methods for protecting structures which are adjacent to excavations required for new construction. The proposal indicates when the use of temporary earth retention systems need to be determined by a registered design professional. The proposal provides general requirements for the design, installation, monitoring and removal of temporary earth retention systems. This proposal defines the various stages requiring action during the implementation of schemes used to support walls of excavated areas where there is a need to contain earth movements in order to protect adjacent structures and public infrastructure elements as well as provide for the safety and well being of the public at large.

This proposal introduces the term "temporary earth retention system" and refers to the proposed new Section 1811 which describes the basic requirements for this type of system. It is much more common to use temporary earth retention systems to support the walls of an excavation and to use these systems in lieu of underpinning to prevent detrimental movements to existing property. The present version of the Code does not cover this type of system adequately.
Cost Impact: Will not increase the cost of construction

This code change proposal provides a general description for the use of temporary earth retention systems. The installation of temporary earth retention systems to support walls of excavated areas during building construction in order to protect both the new construction and adjacent property reflects typical current practice. For projects where it is recognized that there is a need to install a temporary earth retention system, the proposed code change will not increase the cost of construction. If a temporary earth retention system is not used and was needed, the resulting damage which may occur to either the new construction or to adjacent structures and surrounding infrastructure elements can be extensive and very expensive to repair.

<table>
<thead>
<tr>
<th>S241-16 : 1811 (NEW) MAYNARD12713</th>
</tr>
</thead>
</table>

Committee Action: Disapproved

Committee Reason: There are concerns over whether the building code is the appropriate place for temporary earth retention systems and, if so, where should they be located. Those feeling these provisions should be available suggest possibly Chapter 33 for protection of other property or even in an appendix. It was pointed out the possible confusion with the definition of "temporary" in other code applications being limited to 180 days. There are questions about the proposed section on removal and whether that would allow detensioning tiebacks.

Assembly Action: None

Public Hearing Results

Public Comment 1:

Proponent: theodore Maynard, representing GeoCoalition (trmagm@aol.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

SECTION 1811 TEMPORARY EARTH RETENTION SYSTEMS

1811.1.2 Depth of excavation requirements. Excavations deeper than 5 feet with side slopes to steep to prevent ground movements or collapse, as determined by a registered design professional, shall be provided with a temporary earth retention system.

1811.4.1 Retention Wall Removal. Removal shall be performed in such a manner as to protect the new structure, adjacent structures and adjacent infrastructure elements.

1804.2 Temporary earth Excavation retention system. Where a temporary earth excavation retention system is selected to protect or support existing foundations provide protection for adjacent property, structures or infrastructure elements, it shall conform to the temporary earth retention system shall be designed, installed and removed in accordance with the provisions requirements of Section 1811 Chapter33.

1811.1.3 3307.2 Definition Excavation retention systems. A temporary earth retention system shall conform to the following requirements building official.

1811.1.4 3307.2.1 Underpinning relationship. Requirements for underpinning adjacent structures prior to or excavation for the installation of temporary earth retention systems shall be determined in accordance with Section 1803.5.7 by a registered design professional. Temporary earth Excavation retention systems are permitted in conjunction with underpinning or in lieu of adjacent structures underpinning.

1811.2 3307.2.2 Temporary Earth Retention System Design Excavation retention system design. Temporary earth Excavation retention systems shall be designed and installed in accordance with the provisions of Chapters Chapter 16 and 18. The design shall be performed and When providing protection for adjacent property, the installation/ removal procedures...
determined by a registered design professional. The system shall be designed to limit ground movements when adjacent property may be affected.

1811.3 3307.2.3 Temporary Earth Retention System Monitoring Excavation retention system monitoring Where determined by the registered design professional responsible for the temporary earth retention system design, monitoring Monitoring of the retention system shall be performed during installation; for the life of the system until backfilling is complete; and, if required by the building official, during removal of the system. Monitoring shall be performed under the supervision of a registered design professional, using generally accepted methods, by personnel trained and experienced in the chosen methods. Monitoring shall be performed at intervals sufficient to provide information of impending excessive movements in time to mitigate impending excessive movements the situation and protect adjacent structures and infrastructure elements property.

1811.4 3307.2.4 Wall Support Removal Retention system removal. Temporary earth retention Retention system wall supports may only be removed when corresponding adequate replacement support is provided by backfill or by the new structure. The loads on the temporary earth retention system Removal shall only be transferred performed in such a manner to the new structure once the new structure is capable of sustaining these loads, as determined by the registered design professional for the new structure protect adjacent property.

Commenter’s Reason: This proposal has been significantly revised to incorporate the reviewing committee’s comments. While the proposers would prefer this information in Chapter 18, it is agreed that Chapter 33 is an acceptable vehicle as long as the new Section 1804.2 is also approved. This version does not contain any temporary provisions and is silent about the removal of tie-backs as long as it doesn't adversely affect adjacent property. An excavation retention system is usually necessary for major buildings with deep basements and, depending on soil conditions, even could be needed in relatively shallow excavations where adjacent structures are in close proximity. The current version of the Code is silent on the existence of this type of retaining system. The Code presently refers to underpinning as a means of protecting adjacent foundations. Excavation retaining systems are much more common with underpinning considered a last resort. This proposal indicates the relationship between retention systems and underpinning. The need for and type of excavation retention is the first item which has to be addressed in new building construction and the Code should contain some general requirements such as proposed herein.
Proposed Change as Submitted

Proponent: Satyendra Ghosh, S. K. Ghosh Associates Inc., representing Federal Emergency Management Agency - National Institute of Building Sciences Building Seismic Safety Council and Jennifer Goupil, American Society of Civil Engineers, representing the American Society of Civil Engineers (skghoshinc@gmail.com)

2015 International Building Code

Revise as follows:

1613.1 Scope. Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7, excluding Chapters 11, 12, 13, 15, 17 and Appendix 11A, as applicable. The seismic design category for a structure is permitted to be determined in accordance with Section 1613 or ASCE 7.

Exceptions:

1. Detached one- and two-family dwellings, assigned to Seismic Design Category A, B or C, or located where the mapped short-period spectral response acceleration, $S$, is less than 0.4 g.
2. The seismic force-resisting system of wood-frame buildings that conform to the provisions of Section 2308 are not required to be analyzed as specified in this section.
3. Agricultural storage structures intended only for incidental human occupancy.
4. Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances and nuclear reactors.

1901.2 Plain and reinforced concrete. Structural concrete shall be designed and constructed in accordance with the requirements of this chapter and ACI 318 as amended in Section 1905 of this code. Except for the provisions of Sections 1904 and 1907, the design and construction of slabs on grade shall not be governed by this chapter unless they transmit vertical loads or lateral forces from other parts of the structure to the soil. Precast concrete diaphragms in buildings assigned to Seismic Design Category C, D, E, or F shall be designed in accordance with the requirements of ASCE 7 Section 14.2.4.

Reason: Seismic design of diaphragms is addressed in Sections 12.10.1 and 12.10.2 of ASCE 7-16. These sections are essentially the same as Sections 12.10.1 and 12.10.2 of ASCE 7-10. Based on significant work done by Issue Team 6 on Diaphragms of the Building Seismic Safety Council (BSSC) Provisions Update Committee (PUC), an alternative seismic design force level for diaphragms has been included in new Section 12.10.3 of ASCE 7-16. The alternative design force level is mandated for precast concrete diaphragms in buildings assigned to Seismic Design Category (SDC) C and above. It is permitted for other precast concrete diaphragms, cast-in-place concrete diaphragms, and wood diaphragms. At the same time, new precast diaphragm design provisions have been included in new Section 14.2.4 of ASCE 7-16, which goes hand-in-hand with the alternative diaphragm design force level in Section 12.10.3 of ASCE 7-16. The Section 14.2.4 requirements are based on multi-year, multi-million-dollar research, known as DSDM (Diaphragm Seismic Design Methodology) research, sponsored by the National Science Foundation (NSF), the Precast/Prestressed Concrete Institute (PCI), and the Pankow Foundation.

An integral part of the precast diaphragm design procedure of ASCE 7-16 Section 14.2.4 is a connector qualification methodology that was also developed in the course of DSDM research. ASCE 7-16 Section 12.10.3 will automatically be part of the 2018 IBC, presuming it adopts ASCE 7-16; however, Section 14.2.4 will not be, because 2015 IBC Section 1613 excludes Section 14.2 from the adoption of ASCE 7. This code change is meant to take care of this problem and make ASCE 7-16 Section 14.2.4 a part of the 2018 IBC.

Appendix 11A is no longer part of ASCE 7-16. Instead of excluding any particular chapter(s), this proposed change calls out the primary ASCE 7 chapters that charge specific parts of the design process. These chapters, in turn, reference other ASCE 7 sections, other ASCE 7 chapters and other standards for portions of the requirements. All needed parts of ASCE 7 are therefore incorporated, including the ground motions.

Cost Impact: Will increase the cost of construction

The cost of precast concrete diaphragms will go up - not so much because of this proposal, but because the higher design force level for precast concrete diaphragms in Section 12.10.3 of ACE 7-16, which is mandated to be used with the proposed design procedure. The required use of high-deformability connectors with the Reduced Design Option may also contribute to an increase in cost. Finally, the required use of moderate-deformability connectors with the Basic Design Option may result in modest cost increases.
Committee Action: Approved as Modified

Modification:

2015 International Building Code

1613.1 Scope. Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7, Chapters 11, 12, 13, 15, 17 and 18, as applicable. The seismic design category for a structure is permitted to be determined in accordance with Section 1613 or ASCE 7.

- Exceptions:
  1. Detached one- and two-family dwellings, assigned to Seismic Design Category A, B or C, or located where the mapped short-period spectral response acceleration, $S_S$, is less than 0.4 g.
  2. The seismic force-resisting system of wood-frame buildings that conform to the provisions of Section 2308 are not required to be analyzed as specified in this section.
  3. Agricultural storage structures intended only for incidental human occupancy.
  4. Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances and nuclear reactors.
  5. Reference in ASCE 7 to Chapter 14 shall not apply, except as specifically required herein.

Committee Reason: This proposal updates IBC provisions for coordination with the latest edition of the referenced standard, ASCE 7, which was updated in ADM94-16. The modification reinstates the exclusion of Chapter 14 in ASCE 7.

Assembly Action: None

Individual Consideration Agenda

Proponent: Ed Berkel, representing ICC Code Correlation Committee (ccc@icc safe.org) requests Disapprove.

Commenter's Reason: The Code Correlation Committee requests Disapproval of this code change proposal in order to bring a correlation issue to the attention of the full membership at the Public Comment Hearings and to allow the membership to coordinate action on this code change proposal with action taken on Code Change Proposal ADM94-16.

ADM94-16 is the administrative update to referenced standards in the I-Codes. One of these standards, ASCE7, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, was proposed for update to ASCE7-16. However, a successful assembly motion requests that the referenced ASCE7 remain at ASCE7-10 as it presently is referenced in the 2015 I-Codes. This code change proposal coordinates with and relies upon reference to ASCE7-16.

The Code Correlation Committee is a standing committee of the International Code Council whose objectives, procedures and organization are set forth in Council Policy CP#44-13. The objective of the Code Correlation Committee is to maintain technical and editorial consistency among the International Codes and to assist staff in the evaluation and processing of code change proposals and comments that are exclusively editorial.
Proposed Change as Submitted

Proponent: Jason Thompson, Masonry Alliance for Codes and Standards (MACS), representing Masonry Alliance for Codes and Standards (jthompson@ncma.org)

2015 International Building Code

Revise as follows:

2103.1 Masonry units. Concrete masonry units, clay or shale masonry units, stone masonry units, glass unit masonry and AAC masonry units shall comply with Article 2.3 of TMS 602/ACI 503.1/ASCE 6. Architectural cast stone shall conform to ASTM C 1364. Adhered manufactured stone masonry veneer units shall conform to ASTM C1670.

Exception: Structural clay tile for nonstructural use in fireproofing of structural members and in wall furring shall not be required to meet the compressive strength specifications. The fire-resistance rating shall be determined in accordance with ASTM E 119 or UL 263 and shall comply with the requirements of Table 602.

Reference standards type: This reference standard is new to the ICC Code Books

Add new standard(s) as follows:

ASTM C1670-15 Standard Specification for Adhered Manufactured Stone Masonry Veneer Units

Reason: While commonly used as a cladding material, adhered manufactured stone masonry has historically not had a national, consensus-based specification governing the minimum properties for these products; which in turn has been a source of performance issues in the field. Topics covered by ASTM C1670 include:
1) Minimum requirements for constituent materials.
2) Sampling and testing criteria.
3) Minimum compressive strength, maximum absorption, minimum freeze-thaw durability, minimum bond strength, and maximum drying shrinkage requirements.

Cost Impact: Will not increase the cost of construction

Adoption of this standard establishes minimum physical requirements for manufactured stone veneer units consistent with existing industry practices.

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM C1670, with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 1, 2016.

Public Hearing Results

Part I

Committee Action: Approved as Submitted

Committee Reason: Agreement with the proponent's reason which indicates that adding the proposed referenced standard for adhered manufactured stone masonry veneer establishes minimum physical requirements for this material.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Jason Thompson, Masonry Alliance for Codes and Standards, representing Masonry Alliance for Codes and Standards (jthompson@ncma.org) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

2103.1 Masonry units. Concrete masonry units, clay or shale masonry units, stone masonry units, glass unit masonry and AAC masonry units shall comply with Article 2.3 of TMS 602/ACI 503.1/ASCE 6. Architectural cast stone shall conform to ASTM
C 1364. Adhered manufactured stone masonry veneer units shall conform to ASTM C1670.

- Exception: Structural clay tile for nonstructural use in fireproofing of structural members and in wall furring shall not be required to meet the compressive strength specifications. The fire-resistance rating shall be determined in accordance with ASTM E 119 or UL 263 and shall comply with the requirements of Table 602.

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:

- ASTM C1670-16 Standard Specification for Adhered Manufactured Stone Masonry Veneer Units

Commenter's Reason: Since the Committee hearings in April, a new version of ASTM C1670 has been published by ASTM. This modification simply captures the very latest information by updating the reference standard to the 2016 edition.

Analysis: The proposed modification to this code change proposal includes update of the year edition of standard ASTM C1670 from -15 to -16. CP28, Section 3.6.3.1 and newly referenced standard “shall be completed and readily available prior to the Public Comment Hearing based on the cycle of code development which includes the code change proposal.” Therefore, the proponent is required to provide information verifying that the standard ASTM C1670-16 is completed and readily available at the time of the public comment hearings.
S245-16 Part II
IRC: R606.2.6 (New).

Proposed Change as Submitted

Proponent: Jason Thompson, Masonry Alliance for Codes and Standards (MACS), representing Masonry Alliance for Codes and Standards (jthompson@ncma.org)

2015 International Residential Code

Add new text as follows:

R606.2.6 Adhered manufactured stone masonry veneer units. Adhered manufactured stone masonry veneer units shall conform to ASTM C1670.

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
ASTM C1670-15 Standard Specification for Adhered Manufactured Stone Masonry Veneer Units

Reason: While commonly used as a cladding material, adhered manufactured stone masonry has historically not had a national, consensus-based specification governing the minimum properties for these products; which in turn has been a source of performance issues in the field. Topics covered by ASTM C1670 include:
1) Minimum requirements for constituent materials.
2) Sampling and testing criteria.
3) Minimum compressive strength, maximum absorption, minimum freeze-thaw durability, minimum bond strength, and maximum drying shrinkage requirements.

Cost Impact: Will not increase the cost of construction
Adoption of this standard establishes minimum physical requirements for manufactured stone veneer units consistent with existing industry practices.

Public Hearing Results

Part II

Committee Action: Approved as Submitted

Committee Reason: This proposal adds a new section for adhered manufactured stone masonry veneer and brings the current standard for design and installation of the product into the IRC.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Jason Thompson, representing Masonry Alliance for Codes and Standards (jthompson@ncma.org) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Residential Code

R606.2.6 Adhered manufactured stone masonry veneer units. Adhered manufactured stone masonry veneer units shall conform to ASTM C1670.

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:

ASTM C1670-15 Standard Specification for Adhered Manufactured Stone Masonry Veneer Units

Commenter's Reason: Since the Committee hearings in April, a new version of ASTM C1670 has been published by ASTM.
This modification simply captures the very latest information by updating the reference standard to the 2016 edition.

**Analysis**: The proposed modification to this code change proposal includes update of the year edition of standard ASTM E2925 from -14 to -16. CP28, Section 3.6.3.1 and newly referenced standard “shall be completed and readily available prior to the Public Comment Hearing based on the cycle of code development which includes the code change proposal.” Therefore, the proponent is required to provide information verifying that the standard ASTM E2925-16 is completed and readily available at the time of the public comment hearings.
Proposed Change as Submitted

Proponent: Jason Thompson, Masonry Alliance for Codes and Standards (MACS), representing Masonry Alliance for Codes and Standards (jthompson@ncma.org); Phillip Samblanet, representing The Masonry Society (psamblanet@masonrysociety.org)

2015 International Building Code

Revise as follows:

2109.1.1 Limitations. The use of empirical design of masonry shall be limited as noted in Section A.1.2 of TMS 402/ACI 530/ASCE 5. The use of dry-stacked, surface-bonded masonry shall be prohibited in Risk Category IV structures. In buildings that exceed one or more of the limitations of Section A.1.2 of TMS 402/ACI 530/ASCE 5, masonry shall be designed in accordance with the engineered design provisions of Section 2101.2 or the foundation wall provisions of Section 1807.1.5.

Section A.1.2.2 of TMS 402/ACI 530/ASCE 5 shall be modified as follows:

A.1.2.2 Wind. Empirical requirements shall not apply to the design or construction of masonry for buildings, parts of buildings, or other structures to be located in areas where \( V_{\text{asd}} \), as determined in accordance with Section 1609.3.1 of the International Building Code, exceeds 110 mph.

Reason: There are a few issues with this modification to the empirical design provisions of TMS 402:

1) By reference to Section 1609.3.1 one could interpret that Risk Category IV structures could be empirically designed; which isn't permitted by the reference standard or the intent of the empirical design provisions.

2) TMS 402 caps the maximum wind speed at 125 mph (ultimate), which is considerably lower than the 110/142 mph (allowable/ultimate) wind speed trigger of Section 2109.1.1.

3) TMS 402 places additional wind speed limitations on the use of empirical design based upon building height and masonry assembly use (loadbearing versus nonloadbearing).

By maintaining this modification to the reference standard has the net result of considerably reducing the stringency of the wind speed limitations originally intended for the empirical design provisions of TMS 402.

Cost Impact: Will increase the cost of construction

The net impact of removing this modification will increase the stringency, and therefore more than likely the cost of construction.

Public Hearing Results

Committee Action: Disapproved

Committee Reason: Based on the committee's action on S249-16, this proposal is considered unnecessary.

Assembly Action: None

Individual Consideration Agenda

Proponent: Martin Hammer, Martin Hammer, Architect, representing Martin Hammer, Architect (mfhammer@pacbell.net) requests Approve as Submitted.

Commenter's Reason: During the IBC-S Committee Action Hearings in Louisville, there was confusion by this commenter who testified in opposition to S250-16, and among committee members in their pre-vote discussion, whether approving S250-16 would create a conflict with the preceding proposal S249-16, which was approved with a floor modification. After subsequent communication with the original proponent, and after discussions with colleagues and more thorough investigation, this commenter sees that the original proposal should be approved as submitted for the following reasons:

Approving S250-16, and therefore striking the existing code language in Section 2109.1 Limitations, as proposed, will create no conflict with the approved S249-16. Leaving the language in place will cause two problems (if the committee decision to disapprove S250-16 stands). One, it causes confusion because the existing code language's required modification of Section A.1.2.2 in TMS402/ACI530/ASCE5 is in terms of basic wind speed \( V_{\text{asd}} \) whereas Section A.1.2.2 is in terms of ultimate design
wind speed $V_{ult}$. Two, the required modification of Section A.1.2.2 in TMS402/ACI530/ASCE5 would allow use of empirical design for adobe structures up to a higher wind speed threshold ($V_{ult}$ of 142 mph) compared with the threshold ($V_{ult}$ of 125 mph) in place for all other masonry structures in TMS402/ACI530/ASCE5.

There is no basis for adobe structures to use empirical design up to a higher wind speed threshold than the threshold used for all other masonry materials and systems.

For these reasons the IBC Building Code Committee's decision to disapprove S250-16 should be overturned, and it should be approved as submitted.
Proposed Change as Submitted

Proponent: Bonnie Manley, AISI, representing American Iron and Steel Institute (bmanley@steel.org)

2015 International Building Code

Revise as follows:

2203.1 Identification. Identification of structural steel elements shall be in accordance with AISC 360. Identification of cold-formed steel members shall be in accordance with AISI S100. Identification of cold-formed steel light-frame construction shall also comply with the requirements contained in AISI S200 S240 or AISI S220, as applicable. Other steel furnished for structural load-carrying purposes shall be properly identified for conformity to the ordered grade in accordance with the specified ASTM standard or other specification and the provisions of this chapter. Steel that is not readily identifiable as to grade from marking and test records shall be tested to determine conformity to such standards.

2203.2 Protection. Painting of structural steel elements shall be in accordance with AISC 360. Painting of open-web steel joists and joist girders shall be in accordance with SJI CJ, SJI JG, SJI K and SJI LH/DLH. Individual structural members and assembled panels of cold-formed steel construction shall be protected against corrosion in accordance with the requirements contained in AISI S100. Protection of cold-formed steel light-frame construction shall be in accordance with AISI S200 S240 or AISI S220, as applicable.

2210.2 Seismic requirements for cold-formed steel structures. Where a response modification coefficient, R, in accordance with ASCE 7, Table 12.2-1, is used for the design of cold-formed steel structures, the structures shall be designed and detailed in accordance with the requirements of AISI S100, ASCE 8, or, for cold-formed steel special-bolted moment frames, AISI S140 S400.

2211.1 General Structural framing. The

For cold-formed steel light-frame construction, the design and installation of the following structural framing systems, including their members and nonstructural members utilized in cold-formed steel light-frame construction where the specified minimum base steel thickness is not greater than 0.1180 inches (2.997 mm) connections, shall be in accordance with AISI S200 S240, and Sections 2211.2 2211.1.1 through 2211.7, or AISI S220 2211.1.3, as applicable:

1. Floor and roof systems,
2. Structural walls,
3. Shear walls, strap braced walls and diaphragms to resist in-plane lateral loads, and
4. Trusses.

Add new text as follows:

2211.1 Seismic requirements for cold-formed steel structural systems. The design of cold-formed steel light frame construction to resist seismic forces shall be in accordance with the provisions of Section 2211.1.1.1 or 2211.1.1.2, as applicable.

2211.1.1 Seismic Design Categories B and C. Where a response modification coefficient, R, in accordance with ASCE 7, Table 12.2-1 is used for the design of cold-formed steel light frame construction assigned to Seismic Design Category B or C, the seismic force-resisting system shall be designed and detailed in accordance with the requirements of AISI S400.

Exception: The response modification coefficient, R, designated for “Steel systems not specifically detailed for seismic resistance, excluding cantilever column systems” in ASCE 7 Table 12.2-1 shall be permitted for systems designed and detailed in accordance with AISI S240 and need not be designed and detailed in accordance with AISI S400.

2211.1.2 Seismic Design Categories D through F. In cold-formed steel light frame construction assigned to Seismic Design Category D, E, or F, the seismic force-resisting system shall be designed and detailed in accordance with AISI S400.

Revise as follows:

2211.2 2211.1.2 Prescriptive framing. No change to text.

2211.3 2211.1.3 Truss design. Cold-formed steel trusses shall be designed in accordance with AISI S200, the additional provisions of Sections 2211.3.1 2211.1.3.1 through 2211.1.3.4 and accepted engineering practice 2211.1.3.3.

2211.3.1 2211.1.3.1 Truss design drawings. The truss design drawings shall conform to the requirements of Section 2211.1.1.1 of AISI S240 S202 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 2211.1.1.1 of AISI S240 S202 where these methods are utilized to provide restraint/bracing.

2211.3.2 2211.1.3.2 Trusses spanning 60 feet or greater. No change to text.
2211.3.4 Truss quality assurance. Trusses not part of a manufacturing process that provides requirements for quality control done under the supervision of a third-party quality control agency in accordance with AISI S240 Chapter D, shall be manufactured in compliance with Sections 1704.2.5 and 1705.2, as applicable.

Delete without substitution:

2211.2 Header design. Headers, including box and back-to-back headers, and double and single L-headers shall be designed in accordance with AISI S212 or AISI S100.

Add new text as follows:

2211.2 Nonstructural Members. For cold-formed steel light frame construction, the design and installation of nonstructural members and connections shall be in accordance with AISI S220.

Delete without substitution:

2211.3.2 Deferred submittals. AISI S214 Section B4.2 shall be deleted.

2211.4 Structural wall stud design. Structural wall studs shall be designed in accordance with either AISI S211 or AISI S100.

2211.5 Floor and roof system design. Framing for floor and roof systems in buildings shall be designed in accordance with either AISI S210 or AISI S100.

2211.6 Lateral design. Light-frame shear walls, diagonal strap bracing that is part of a structural wall and diaphragms used to resist wind, seismic and other in-plane lateral loads shall be designed in accordance with AISI S213.

Reference standards type: This contains both new and updated standards

Add new standard(s) as follows:

AISI S200—12, North American Standard for Cold-Formed Steel Framing-General Provisions, 2012, 2203.1, 2203.2, 2203.12.1, Table 2603.12.2
AISI S210—07(2012), North American Standard for Cold-Formed Steel Framing—Floor and Roof System Design, 2007 (Reaffirmed 2012), 2211.5
AISI S211—07/S1-12(2012), North American Standard for Cold-Formed Steel Framing—Wall Stud Design, 2007 including Supplement 1, dated 2012 (Reaffirmed 2012), 2211.4
AISI S212—07(2012), North American Standard for Cold-Formed Steel Framing—Header Design, 2007. (Reaffirmed 2012), 2211.2
AISI S214—12, North American Standard for Cold-Formed Steel Framing—Truss Design, 2012, 2211.3, 2211.3.1, 2211.3.2
AISI S202, Code of Standard Practice for Cold-Formed Steel Structural Framing, 2015
AISI S240, North American Standard for Cold-Formed Steel Structural Framing, 2015
AISI S400, North American Standard for Seismic Design of Cold-Formed Steel Structural Systems, 2015

Reason: This proposal is one in a series adopting the latest generation of AISI standards for cold-formed steel. This particular proposal focuses on Chapter 22 by incorporating references to three new cold-formed steel standards -- AISI S240, AISI S400, and AISI S202. All three standards are published and available for a free download at: www.aisistandards.org.

AISI S240, North American Standard for Cold-Formed Steel Structural Framing, addresses requirements for construction with cold-formed steel structural framing that are common to prescriptive and engineered light frame construction. This comprehensive standard was formed by merging the following AISI standards:

- AISI S200, North American Standard for Cold-Formed Steel Framing-General Provisions
- AISI S210, North American Standard for Cold-Formed Steel Framing—Floor and Roof System Design
- AISI S211, North American Standard for Cold-Formed Steel Framing—Wall Stud Design
- AISI S212, North American Standard for Cold-Formed Steel Framing—Header Design
- AISI S213, North American Standard for Cold-Formed Steel Framing—Lateral Design
- AISI S214, North American Standard for Cold-Formed Steel Framing—Truss Design

Consequently, AISI S240 supersedes all previous editions of the above mentioned individual AISI standards. Additionally, the standard builds upon this foundation by adding the first comprehensive chapter on quality control and quality assurance for cold-formed steel light frame construction.

AISI S400, North American Standard for Seismic Design of Cold-Formed Steel Structural Systems, addresses the design and construction of cold-formed steel structural members and connections used in the seismic force-resisting systems in buildings and other structures. This first edition primarily represents a merging of the requirements from AISI S110, Standard for Seismic Design of Cold-Formed Steel Structural Systems – Special Bolted Moment Frame, 2007 with Supplement No. 1-09, and the
seismic portions of AISI S213, *North American Standard for Cold-Formed Steel Framing – Lateral Design*, 2007 with Supplement No. 1-09. The layout and many of the seismic design requirements are drawn from ANSI/AISC 341-10, *Seismic Provisions for Structural Steel Buildings*, which is developed by the American Institute of Steel Construction (AISC). AISI S400 supersedes AISI S110 and the seismic design provisions of AISI S213 and is intended to be applied in conjunction with both AISI S100 and AISI S240, as applicable.

**AISI S202, Code of Standard Practice for Cold-formed Steel Structural Framing**, is intended to service as a state-of-the-art mandatory document for establishing contractual relationships between various parties in a construction project where cold-formed steel structural materials, components and assemblies are used. While it is not specifically intended to be a direct reference in the building code, portions of AISI S202 are recommended for adoption in this proposal to establish the minimum requirements for cold-formed steel truss design drawings.

Modifications specific to Chapter 22 include the following:

- **Section 2203:** Requirements on identification and protection of cold-formed steel framing are now located in AISI S240.
- **Section 2210.2:** Requirements for the cold-formed steel special-bolted moment frame are now located in AISI S400.
- **Section 2211:** Requirements for cold-formed steel light-frame construction are now split into two major subsections – structural provisions are located in Section 2211.1 and nonstructural provisions are located in Section 2211.2.
- **Section 2211.1:** Reference to AISI S240 is made for the general design of cold-formed steel structural framing systems.
- **Section 2211.1.1:** Reference to AISI S400 is made for the design of cold-formed steel seismic force-resisting systems. Since the relationship between AISI S240 and AISI S400 is similar to that between AISC 360 and AISC 341, the charging language in IBC Section 2211.1.1 has been modified to parallel the language in Section 2205.2 for structural steel. It adopts AISI S400 and exempts seismic force-resisting systems only where the seismic design category is B or C and the seismic response modification coefficient, R, equals 3. This is done to recognize that ASCE 7, Table 12.2-1, Line H exempts steel systems from seismic detailing requirements as long as they are designed in accordance with AISI S240.
- **Section 2211.1.2:** No substantive changes are proposed for prescriptive framing.
- **Section 2211.1.3:** Requirements for cold-formed steel trusses are updated and streamlined to reflect changes in AISI S240. Additionally, in the process of merging the old AISI S214 into the new AISI S240, requirements for truss design drawings were relocated to AISI S202. Consequently, a direct pointer was added to Section 2211.1.3.1.

**Cost Impact:** Will increase the cost of construction  
This code change proposal adopts the latest industry standards for cold-formed steel. At this time, it is difficult to anticipate how cost of construction will be fully impacted, other than to note that some of the additional costs will be offset by new efficiencies in the design and installation of cold-formed steel.

**Analysis:** A review of the standard(s) proposed for inclusion in the code, AISI S202, AISI S240 & AISI S400 with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 1, 2016.

**Public Comment 1:**  
Proponent: Bonnie Manley, AISI, representing American Iron and Steel Institute (bmanley@steel.org) requests Approve as Modified by this Public Comment.

**Modify as Follows:**

**2015 International Building Code**

**2211.1 Structural framing.** For cold-formed steel light-frame construction, the design and installation of the following structural framing systems, including their members and connections, shall be in accordance with AISI S240, and Sections
2211.1 through 2211.1.3, as applicable:
1. Floor and roof systems,
2. Structural walls,
3. Shear walls, strap braced walls and diaphragms to resist in-plane lateral loads, and
4. Trusses.

Reference standards type: This reference standard is new to the ICC Code Books

Add new standard(s) as follows:

Commenter's Reason: The AISI Committee on Framing Standards recently completed Supplement 1-16 to AISI S400-15. It addresses a public comment received on the standard from California's Division of the State Architect (DSA). This supplement revises the expected strength factors for cold-formed steel light-frame shear walls sheathed with wood structural panels, steel sheet sheathing, gypsum board, and fiberboard panel sheathing. The supplement is published and available for a free download at: www.aisistandards.org.

Analysis: The proposed modification to this code change proposal includes update of the year edition of standard AISI S400-15 to AISI S400-15/S1-16. CP28, Section 3.6.3.1 and newly referenced standard “shall be completed and readily available prior to the Public Comment Hearing based on the cycle of code development which includes the code change proposal.” Therefore, the proponent has, as required, provided information verifying that the standard AISI S400-15/S1-16 is completed and readily available at the time of the public comment hearings.

Public Comment 2:

Proponent : Bonnie Manley, AISI, representing American Iron and Steel Institute (bmanley@steel.org) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

2211.1.3.3 Truss quality assurance. Trusses that are not part of manufactured using a manufacturing process that provides requirements for overseen by a quality control done under the program with third-party supervision of a third party shall be subject to quality control agency assurance in accordance with AISI S240, Chapter D, shall be fabricated in compliance with Sections 1704.2.5 and 1705.2, as applicable.

Commenter's Reason: This public comment is intended to coordinate with a public comment submitted on Proposal S129-16 to bring back modifications to the cold-formed steel truss special inspection requirements. This public comment focuses on establishing a minimum quality assurance requirement for trusses that are not subject to third-party supervised quality control during manufacture. It institutes the minimum quality assurance provisions by referencing AISI S240, Chapter D, while retaining the reference to Section 1704.2.5 (approved fabricator) and Section 1705.2 (exception for simple fabrication processes). The references to Chapter 17 has been corrected from “fabricated in compliance”, since Chapter 17 does not address fabrication requirements; those reside in Chapter 22, the material chapter. With this public comment, the references to Chapter 17 now invoke special inspection/quality assurance of the fabrication process.
Committee Action: Disapproved

Committee Reason: The committee believes this proposal would add a new application to preservative treated wood that is not justified.

Assembly Action: None

Public Comment 1:
proponent: Paul Coats, PE CBO (pcoats@awc.org) requests Approve as Modified by this Public Comment.

Modify as follows:

2015 International Building Code

2303.1.9 Preservative-treated wood. Lumber, timber, plywood, piles and poles supporting permanent structures required by Section 2304.12 to be preservative treated shall conform to the requirements of the applicable AWPA Standard U1 and M4 for the species, product, preservative and end use. Preservatives shall be listed in Section 4 of AWPA U1. Lumber and plywood
used in wood foundation systems shall conform to Chapter 18.

2303.1.9 Identification. Wood required by Section 2304.12 to be preservative treated shall bear the quality mark of an inspection agency that maintains continuing supervision, testing and inspection over the quality of the preservative-treated wood. Inspection agencies for preservative-treated wood shall be listed by an accreditation body that complies with the requirements of the American Lumber Standards Treated Wood Program, or equivalent. The quality mark shall be on a stamp or label affixed to the preservative-treated wood, and shall include the following information:
1. Identification of treating manufacturer.
2. Type of preservative used.
3. Minimum preservative retention (pcf).
4. End use for which the product is treated.
5. AWPA standard to which the product was treated.
6. Identity of the accredited inspection agency.

2303.1.9.2 Moisture content. Where preservative-treated wood is used in enclosed locations where drying in service cannot readily occur, such wood shall be at a moisture content of 19 percent or less before being covered with insulation, interior wall finish, floor covering or other materials.

2303.1.9.3 Strength Adjustments. Design values for preservative-treated wood in accordance with Section 2303.1.9 need no adjustment for type of preservative used. Other adjustments are applicable except that the impact load in accordance with AWC NDS shall apply. Load duration factors for structural members pressure-treated with water-borne preservatives shall not exceed 1.6.

2303.2 Fire-retardant-treated wood. Fire-retardant-treated wood is any wood product which, when impregnated with chemicals by a pressure process or other means during manufacture, shall have, when tested in accordance with ASTM E 84 or UL 723, a listed flame spread index of 25 or less and show no evidence of significant progressive combustion when the test is continued for an additional 20-minute period. Additionally, the flame front shall not progress more than $10^{1/2}$ feet (3200 mm) beyond the centerline of the burners at any time during the test.

2303.2.1 Pressure process. For wood products impregnated with chemicals by a pressure process, the process shall be performed in closed vessels under pressures not less than 50 pounds per square inch gauge (psig) (345 kPa).

2303.2.2 Other means during manufacture. For wood products produced by other means during manufacture, the treatment shall be an integral part of the manufacturing process of the wood product. The treatment shall provide permanent protection to all surfaces of the wood product.

2303.2.3 Testing. For wood products produced by other means during manufacture, other than a pressure process, all sides of the wood product shall be tested in accordance with and produce the results required in Section 2303.2. Wood structural panels shall be permitted to test only the front and back faces.

2303.2.4 Labeling. Fire-retardant-treated lumber and wood structural panels shall be labeled. The label shall contain the following items:
1. The identification mark of an approved agency in accordance with Section 1703.5.
2. Identification of the treating manufacturer.
3. The name of the fire-retardant treatment.
4. The species of wood treated.
5. Flame spread and smoke-developed index.
7. Conformance with appropriate standards in accordance with Sections 2303.2.5 through 2303.2.8.
8. For fire-retardant-treated wood exposed to weather, damp or wet locations, include the words "No increase in the listed classification when subjected to the Standard Rain Test" (ASTM D 2898).

2303.2.5 Strength adjustments. Design values for untreated lumber and wood structural panels, as specified in Section 2303.1, shall be adjusted for fire-retardant-treated wood. Adjustments to design values, including fastener values, shall be based on an approved method of investigation that takes into consideration the effects of the anticipated temperature and humidity to which the fire-retardant-treated wood will be subjected, the type of treatment and redrying procedures. Other adjustments are applicable except that the impact load in accordance with AWC NDS shall apply. Load duration factors for structural members pressure-treated with fire retardant chemicals shall not exceed 1.6.

Commenter's Reason: The original proposal implements a consistent format for preservative treated and fire-retardant treated wood strength adjustments and removes redundant information on stress adjustments in Section 2306.1.3. This public comment proposal revises provisions for adjustment of strength of preservative treated and fire retardant treated wood as follows:
Section 2303.1.9.3. Sentence 1 is revised to replace “treatment” with “type of preservative used”. This revision clarifies that no adjustment is associated with the type of preservative used. In some cases, “treatment” involves a conditioning process or incising and adjustments of design values are applicable as well as other adjustments in accordance with NDS. The second sentence is revised to be consistent with AWC NDS Table 2.3.2 in which load duration adjustment is not to exceed 1.6 for structural members pressure treated with water-borne preservatives and is a smaller adjustment than the factor of 2.0 associated with impact load duration.

Section 2303.2.5. The second sentence is revised to be consistent with AWC NDS Table 2.3.2 in which load duration adjustment is not to exceed 1.6 for structural members pressure treated with fire-retardant chemicals and is a smaller adjustment than the factor of 2.0 associated with impact load duration.
Proposed Change as Submitted

Proponent: Marcelo Hirschler, representing GBH International (gbhint@aol.com)

2015 International Building Code

Revise as follows:

2303.2 Fire-retardant-treated wood. Fire-retardant-treated wood is any listed wood product which, when impregnated with chemicals by a pressure process or other means during manufacture, shall comply with one of the following:

1. It shall have, when tested in accordance with ASTM E 84 or UL 723, a listed flame spread index of 25 or less and show no evidence of significant progressive combustion when the test is continued for an additional 20-minute period. Additionally, the flame front shall not progress more than $10^{1/2}$ feet (3200 mm) beyond the centerline of the burners at any time during the test.

2. It shall be listed to both comply with all of the requirements of ASTM E2768, and also show no evidence of significant progressive combustion during the 30 minute test, when tested on all sides with a ripped or cut longitudinal gap of 1/8 inch (3.2 mm).

Reference standards type: This is an update to reference standard(s) already in the ICC Code Books

Add new standard(s) as follows:


Reason: ASTM E2768 was developed specifically to represent an ASTM E84 test extended to 30 minutes and requiring a flame spread index of 25 or less and a flame front that does not progress more than 10.5 ft (3.2 m) beyond the centerline of the burners at any time during the 30 min test period.

There have been multiple discussions since ASTM E2768 was developed in 2011 at various code hearings as to whether the added requirement (no significant progressive combustion) is included or not in ASTM E2768. The wording of ASTM E2768 reads as follows: "13.1.2 The flame front shall not progress more than 10.5 ft (3.2 m) beyond the centerline of the burners at any time during the 30 min test period. This is considered evidence of no significant progressive combustion in this test method." In order to recognize this test method the present proposal does not enter this debate but it requires that, for a wood product to be accepted as "fire-retardant-treated wood" it must also have been listed, beyond compliance to ASTM E2768, to demonstrate "no evidence of significant progressive combustion".

Furthermore, this proposal also requires that a product tested to ASTM E2768 must have been tested on all sides and must have been tested with a longitudinal gap of 1/8 inch. The intent of this requirement is to ensure that a product that is coated and not impregnated cannot be accepted as "fire-retardant-treated wood". The requirements in this proposal can clearly not be met by a wood product coated with a flame retardant, because the flame retardant needs to have been impregnated in order to have the good fire performance when exposed to flame through the gap, and tested on all sides.

The IWUIC has accepted requirements with language similar to this and it is time for the IBC to accept it too. The IWUIC language for ignition resistant building materials reads as follows (note that the section covers materials that are not made of wood also):

503.2 Ignition-resistant building material. Ignition-resistant building materials shall comply with any one of the following:

1. Material shall be tested on all sides with the extended ASTM E 84 (UL 723) test or ASTM E 2768, except panel products shall be permitted to test only the front and back faces. Panel products shall be tested with a ripped or cut longitudinal gap of 1/8 inch (3.2 mm). Materials that, when tested in accordance with the test procedures set forth in ASTM E 84 or UL 723 for a test period of 30 minutes, or with ASTM E 2768, comply with the following:

1.1. Flame spread. Material shall exhibit a flame spread index not exceeding 25 and shall not show evidence of progressive combustion following the extended 30-minute test.

1.2. Flame front. Material shall exhibit a flame front that does not progress more than 101/2 feet (3200 mm) beyond the centerline of the burner at any time during the extended 30-minute test.

1.3. Weathering. Ignition-resistant building materials shall maintain their performance in accordance with this section under conditions of use. Materials shall meet the performance requirements for weathering (including exposure to temperature, moisture and ultraviolet radiation) contained in the following standards, as applicable to the materials and the conditions of use:


1.3.2. ASTM D 7032 for wood-plastic composite materials.
1.3.3. ASTM D 6662 for plastic lumber materials.

1.4. Identification. All materials shall bear identification showing the fire test results.

Exception: Materials comprised of a combustible core and a noncombustible exterior covering, comprised of either aluminum at a minimum 0.019 inch (0.48 mm) thickness or corrosion-resistant steel at a minimum 0.0149 inch (0.38 mm) thickness shall not be required to be tested with a ripped or cut longitudinal gap.

Cost Impact: Will not increase the cost of construction

ASTM E2768 is simply an alternate means for a wood product to be designated FRTW.

Analysis: The standard proposed for inclusion in this code, ASTM E2768, is referenced in the International Wildland Urban Interface Code.

Public Hearing Results

Committee Action: Disapproved

Committee Reason: The committee believes that the proposed details for testing of fire-retardant-treated wood should be added into the referenced standard and then it should be added into the code. The committee also noted the lack of an industry consensus on this issue.

Assembly Action: None

Public Comment 1:

Proponent: Tim Earl (tearl@gbhinternational.com); Marcelo Hirschler, representing GBH International (gbhint@aol.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

2303.2 Fire-retardant-treated wood. Fire-retardant-treated wood is any listed wood product which, when impregnated with chemicals by a pressure process or other means during manufacture, shall comply with one of the following:
1. It shall have, when tested in accordance with ASTM E 84 or UL 723, a listed flame spread index of 25 or less and show no evidence of significant progressive combustion when the test is continued for an additional 20-minute period. Additionally, the flame front shall not progress more than 10 1/2 feet (3200 mm) beyond the centerline of the burners at any time during the test when the test is continued for an additional 20-minute period.
2. It shall be listed to both comply with all of the requirements of ASTM E2768, and also show no evidence of significant progressive combustion during the 30 minute test, when tested on all both the front and back sides with a ripped or cut longitudinal gap of 1/8 inch (3.2 mm).

Commenter's Reason: This public comment eliminates the requirement for "no evidence of significant progressive combustion" which is unnecessary because it is undefined and there is no evidence as to how to measure it, which makes it very subjective and prone to misleading information. Fire testing labs have used as the corresponding criterion that the flame front in the ASTM E84 test does not progress more than 10.5 ft. beyond the centerline of the burners in either the 10 minute test or the continuation of the test for a total of 30 minutes.

The public comment also proposes to add testing to ASTM E2768 as an alternate option (which is fully equivalent).

The term fire-retardant-treated wood is used in the IBC, the IRC, the IFC, the IMC and the IWUI. It is also used in NFPA codes (NFPA 101, Life Safety Code and NFPA 5000, Building Code) and in NFPA 703 ("Standard for Fire Retardant–Treated Wood and Fire-Retardant Coatings for Building Materials"). It is also used in AC 66 (Acceptance Criteria for Fire-Retardant Treated Wood). However, neither in any of the ICC codes nor in any NFPA code or standard nor in AC 66 nor in ASTM E84 is there any description or guidance for what constitutes "no evidence of significant progressive combustion". However, there is one standard that contains the criterion for the assessment of "no evidence of significant progressive combustion". That standard is ASTM E2768 "Standard Test Method for Extended Duration Surface Burning Characteristics of Building Materials (30 min
It has been stated that AC 66 (Acceptance Criteria for Fire-Retardant Treated Wood) describes the way in which "no evidence of significant progressive combustion" is assessed and that it includes references to the ASTM E69, "Standard Test Method for Combustible Properties of Treated Wood by the Fire-Tube Apparatus" (fire tube test) and to large scale fire tests. With regard to fire testing, the sole reference in AC 66 other than to ASTM E84 or UL 723 is to ASTM E69. There is also no information in AC 66 as to how to assess "no evidence of significant progressive combustion".

The June 2012 edition of AC 66 states as follows:

"5.5.4 Chemical Verification: In all cases, chemical verification shall conform to requirements outlined in the approved quality documentation. Verification shall be by means of fire tube tests or an assay of borings by chemical analysis, using nationally recognized test methods or other methods that have been validated to relate to results of fire tests conducted in accordance with Sections 3.1.4 and 3.2.4 of this criteria. Three fire-tube tests (ASTM E 69, Procedure B) shall be conducted on specimens processed with each charge treated. In lieu of the actual species treated, a standard lumber species, such as Douglas fir, may be used for fire tube testing on each charge. The average final percentage weight loss of the treated wood samples, after flaming and glowing have ceased, and the maximum temperature, shall be equal to or less than that obtained on the qualification-test specimens. The final percentage weight loss of any individual specimen shall not exceed the qualification value by more than five percentage points. Alternately, an assay of borings, by chemical analysis, may be used to verify the treatment process. This analysis shall be conducted on a composite of 20 borings per species per charge, on a representative sampling of the treated lumber. The result of this analysis shall substantiate equivalency to the qualification analysis. When the treatment process is verified by methods other than fire tube tests or an assay of borings, the approved quality documentation shall include a description of the verification method and conditions of acceptance."

"5.7.3 When conducting the ASTM E 69 fire tube test on samples from a load of treated lumber, (i.e., the charge) the charge is acceptable if the first three samples tested meet the quality control requirements. If one of the first three samples fails, an additional three samples may be tested. If all of the three additional samples meet the requirements, the charge is acceptable. If the charge is not acceptable, it shall be retreated and retested.

5.7.4 The solution concentration shall be within the range specified in the quality documentation. If the solution concentration is low, the charge shall be retreated with the proper solution.

5.7.5 The analysis of solution sampled by the quality control agency shall confirm proper chemical composition and concentration. If nonconforming, appropriate action shall be taken by the plant to adjust the solution. Additional samples shall then be analyzed on a weekly basis until conformance has been demonstrated in two consecutive samples. All lumber and plywood found to have been treated with a nonconforming solution shall be segregated and labeled as nonconforming. A representative sampling of the nonconforming lumber and plywood selected by the quality control agency shall be tested, and shall meet the flame spread and strength requirements of the code before it may be released.

5.7.6 The charge retention shall be within the specified range of gage retention of fire-retardant chemical, as determined during qualification testing for the applicable material and species. If retention is below the minimum, the charge shall be retreated so that the total retention is within the minimum and maximum qualified values. If retention is above the maximum allowed, the lumber or plywood in the charge shall not be stamped."

For further information, sections 3.1.4 and 3.2.4 of AC 66 contain the same criteria that the IBC and other codes have, namely

"3.1.4 Surface Burning Characteristics The surface burning characteristics (flame spread and smoke-developed index) shall be determined in accordance with ASTM E 84 or UL 723. The flame spread index shall be 25 or less and there shall be no evidence of significant progressive combustion when the test is continued for an additional 20-minute period. Additionally, the flame front shall not progress more than 10½ feet (3200 mm) beyond the centerline of the burners at any time during the test. The smoke-developed index shall be 450 or less. For recognition of exterior use, tests shall be conducted both before and after durability tests conducted in accordance with Section 3.1.3. The FRT lumber shall meet the requirements of IBC Section 2303.2, IRC Section R802.1.3, UBC Section 207, SBC Section 202, or BNBC 2310.2, as applicable."

"3.2.4 Surface Burning Characteristics The surface burning characteristics (flame spread and smoke-developed index) shall be determined in accordance with ASTM E 84 or UL 723. The flame spread index shall be 25 or less and there shall be no evidence of significant progressive combustion when the test is continued for an additional 20-minute period. Additionally, the flame front shall not progress more than 10½ feet (3200 mm) beyond the centerline of the burners at any time during the test. The smoke-developed index shall be 450 or less. For recognition of exterior use, tests shall be conducted both before and after durability tests conducted in accordance with Section 3.2.3. The FRT plywood shall meet the requirements of Section 2303.2 of the IBC or Section R802.1.3 of the IRC, UBC Section 207, SBC Section 202, or BNBC Section 2310.2, as applicable."

No means has been proposed in any document other than in ASTM E2768 as to what constitutes "no evidence of significant progressive combustion". Fire testing labs have used as the corresponding criterion that the flame front in the ASTM E84 test does not progress more than 10.5 ft. beyond the centerline of the burners in either the 10 minute test of the continuation of the
test for an additional 20 minutes (for a total of 30 minutes). Details follow.

The scope of ASTM E2768 includes the following statement: "The purpose of this fire-test-response standard is to evaluate the ability of a product to limit the surface spread of flame when evaluated for 30 min. This fire-test-response standard uses the apparatus and procedure of Test Method E84 with the total test period extended to 30 min."

The conditions of classification of ASTM E2768 include the following criteria:

1. The flame spread index shall be 25 or less as determined for the initial 10 min test period,
2. The flame front shall not progress more than 10.5 ft (3.2 m) beyond the centerline of the burners at any time during the 30 min test period. This is considered evidence of no significant progressive combustion in this test method.
3. For materials or products that are not homogeneous or symmetrical about their longitudinal axis, only surfaces that have been individually tested shall be eligible to be classified and reported as meeting the conditions of classification of this standard.

Public Comment 2:

Proponent: Marcelo Hirschler, representing GBH International (gbhint@aol.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

2303.2 Fire-retardant-treated wood. Fire-retardant-treated wood is any listed wood product which, when impregnated with chemicals by a pressure process or other means during manufacture, shall comply with one of the following:

1. It shall have, when tested in accordance with ASTM E 84 or UL 723, a listed flame spread index of 25 or less and show no evidence of significant progressive combustion when the test is continued for an additional 20-minute period. Additionally, the flame front shall not progress more than 10.5 ft (3200 mm) beyond the centerline of the burners at any time during the test.

2. It shall be listed to both comply with all of the requirements of ASTM E2768, and also show no evidence of significant progressive combustion during the 30 minute test, when tested on all sides with a ripped or cut longitudinal gap of 1/8 inch (3.2 mm) the test is continued for an additional 20-minute period.

Commenter's Reason:

This public comment simply eliminates the requirement for “no evidence of significant progressive combustion” which is unnecessary because it is undefined and there is no evidence as to how to measure it, which makes it very subjective and prone to misleading information. Fire testing labs have used as the corresponding criterion that the flame front in the ASTM E84 test does not progress more than 10.5 ft beyond the centerline of the burners in either the 10 minute test or the continuation of the test for an additional 20 minutes (for a total of 30 minutes).

The term fire-retardant-treated wood is used in the IBC, the IRC, the IFC, the IMC and the IWUIC. It is also used in NFPA codes (NFPA 101, Life Safety Code and NFPA 5000, Building Code) and in NFPA 703 ("Standard for Fire Retardant–Treated Wood and Fire-Retardant Coatings for Building Materials"). It is also used in AC 66 (Acceptance Criteria for Fire-Retardant Treated Wood). However, neither in any of the ICC codes nor in any NFPA code or standard nor in AC 66 nor in ASTM E84 is there any description or guidance for what constitutes “no evidence of significant progressive combustion”. However, there is one standard that contains the criterion for the assessment of “no evidence of significant progressive combustion”. That standard is ASTM E2768 "Standard Test Method for Extended Duration Surface Burning Characteristics of Building Materials (30 min Tunnel Test)", dated 2011. It states, in the section on classification as follows: "The flame front shall not progress more than 10.5 ft (3.2 m) beyond the centerline of the burners at any time during the 30 min test period. This is considered evidence of no significant progressive combustion in this test method."

This public comment does not propose adding ASTM E2768 into this section of the code (but an alternate public comment does).

It has been stated that AC 66 (Acceptance Criteria for Fire-Retardant Treated Wood) describes the way in which "no evidence of significant progressive combustion" is assessed and that it includes references to the ASTM E69, “Standard Test Method for Combustible Properties of Treated Wood by the Fire-Tube Apparatus” (fire tube test) and to large scale fire tests. With regard to fire testing, the sole reference in AC 66 other than to ASTM E84 or UL 723 is to ASTM E69. There is also no information in AC 66 as to how to assess “no evidence of significant progressive combustion”.

The June 2012 edition of AC 66 states as follows:

"5.5.4 Chemical Verification: In all cases, chemical verification shall conform to requirements outlined in the approved quality documentation. Verification shall be by means of fire tube tests or an assay of borings by chemical analysis, using nationally recognized test methods or other methods that have been validated to relate to results of fire tests conducted in accordance
The conditions of classification of ASTM E2768 include the following criteria:

1. The flame spread index shall be 25 or less as determined for the initial 10 min test period.
2. The flame front shall not progress more than 10.5 ft (3.2 m) beyond the centerline of the burners at any time during the 30 min test period. This is considered evidence of no significant progressive combustion in this test method.
3. For materials or products that are not homogeneous or symmetrical about their longitudinal axis, only surfaces that have been individually tested shall be eligible to be classified and reported as meeting the conditions of classification of this standard.
Public Comment 3:

Proponent: Joseph Holland, Hoover Treated Wood Products, representing Hoover Treated Wood Products (j holland@ftw.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

2303.2 Fire-retardant-treated wood. Fire-retardant-treated wood is any listed wood product which, when impregnated with chemicals by a pressure process or other means during manufacture, shall comply with one of the following:
1. It shall have, when tested in accordance with ASTM E 84 or UL 723, a listed flame spread index of 25 or less and show no evidence of significant progressive combustion when the test is continued for an additional 20-minute period. Additionally, the flame front shall not progress more than 10/2 feet (3200 mm) beyond the centerline of the burners at any time during the test. 2. It shall be listed to comply with all of the requirements of ASTM E2768, and also show no evidence of significant progressive combustion during the 30 minute test. Wood structural panels shall be tested on all sides with a ripped or cut longitudinal gap of 1/8 inch (3.2 mm).

Commenter’s Reason: History:
Pressure impregnated fire-retardant-treated wood or wood where the impregnation occurs during manufacture are the only products required by the code to be tested using ASTM E84 for 30 minutes. The E84 test is a ten minute test used for interior finish. A member of the fire retardant pressure impregnation treating industry approached the D7 (Wood) Committee to create a standard for testing fire-retardant-treated wood in the Steiner Tunnel for 30 minutes as required by the building code. After consideration of such a standard, D7 approached E5 to develop a standard for a broad range of materials. ASTM E2768 is the product of that effort.

Concern:
- The standard will allow testing material for use in load bearing and nonload bearing applications.
- The ASTM E2768 standard only looks at the surface of a material.
- ASTM E2768 allows one to classify materials based on testing only one surface.

The testing of only the surface of a material as well as only one surface may be appropriate if one were testing for use only as an interior finish material where it is not concealed behind other construction. The NFPA 101, Chapter 10, Interior Finish, allows factory applied coating for use as an interior finish, a nonstructural application, when tested for 30 minutes. It is not appropriate for a material tested for 30 minutes used for load bearing applications in walls and roofs.

Codes and standards:
- Recognize surface treatment only for interior finish.
- Do not recognize surface treatments for load bearing applications.
- The only material tested for 30 minutes recognized by codes and standards for use in load bearing applications is fire-retardant-treated wood. Fire-retardant-treated wood must be impregnated by a pressure process or other means during manufacture.

Summary:
The code needs the 1/8 inch gap:
- to separate the materials tested for interior finish as spelled out in Section 10.2.6.2, NFPA 101, 2015 edition
- for material used in load bearing applications
- for material used in concealed spaces.

Public Comment 4:

Proponent: Manny Muniz, representing Myself (Mannymuniz.mm@gmail.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

2303.2 Fire-retardant-treated wood. Fire-retardant-treated wood is any listed wood product which, when impregnated with chemicals by a pressure process or other means during manufacture, shall comply with one of the following:
1. It shall have, when tested in accordance with ASTM E 84 or UL 723, a listed flame spread index of 25 or less and show no evidence of significant progressive combustion when the test is continued for an additional 20-minute period. Additionally, the flame front shall not progress more than 10/2 feet (3200 mm) beyond the centerline of the burners at any time during the test. 2. Lumber and wood structural panels impregnated with chemicals by a process other than a pressure process shall be listed
to both comply in accordance with all of the requirements of ASTM E2768, and also show no evidence of significant progressive combustion during the 30-minute test, when . Lumber shall be tested on all sides. Wood structural panels shall be tested with a ripped or cut longitudinal gap of 1/8 inch (3.2 mm) located between the burners.

Commenter's Reason: The intent of this public comment is to clarify that ASTM E2768 is appropriate for the testing and listing of lumber and wood structural panels impregnated with chemicals by a process other than a pressure process, that lumber must be tested on all sides and that wood structural panels that are impregnated with chemicals by other means must be tested with a 1/8 inch gap. The APA – The Engineered Wood Association specifically recommends a 1/8-inch space between panel edge and end joints. "Plywood and oriented strand board (OSB), like all wood products, will expand or shrink with changes in moisture content. If the wood structural panels are tightly butted, there is no room for expansion and buckling can occur."

S260-16
Committee Action: Disapproved

Assembly Action: None

---

**S263-16 Part I**

**IBC: 2303.2.3.**

**Proposed Change as Submitted**

**Proponent:** Tim Earl, representing GBH International (tearl@gbhinternational.com)

**2015 International Building Code**

Revise as follows:

**2303.2.3 Testing.** For wood products produced by other means during manufacture, other than a pressure process, all sides of the fire-retardant treated wood product shall be tested in accordance with and produce the results required in Section 2303.2. Wood structural panels shall be permitted to test only the front and back faces.

**Reason:** As currently written, this section requires additional testing for fire-retardant treated wood materials that have been manufactured by a process different than pressure treatment. All fire retardant treated wood products must meet the requirements of section 2303.2 (which contains the fire test requirements). Moreover, all fire retardant treated wood products, irrespective of how they are manufactured, must meet the requirements (also contained in section 2303.2) that they must be impregnated with chemicals. A wood material can only be fire retardant treated wood if it is impregnated with chemicals and that will differentiate it from fire retardant coated materials. Testing requirements should be a function of performance and not of the way a product is made. There is no reason that some fire retardant treated wood materials should be treated in a different fashion by the code as a function of how they are manufactured.

**Cost Impact:** Will not increase the cost of construction

This proposal will reduce the cost for manufacturers of some materials by eliminating overly burdensome testing requirements based solely on how they are manufactured.

---

**Public Hearing Results**

**Part I**

**Committee Action:** Disapproved

**Committee Reason:** This proposal would not address the issue that it is intended to solve. This change would increase current testing requirements and would lead to cost increases.

**Assembly Action:** None

---

**Individual Consideration Agenda**

**Public Comment 1:**

**Proponent:** Tim Earl, representing GBH International (tearl@gbhinternational.com) requests Approve as Modified by this Public Comment.

**Modify as Follows:**

**2015 International Building Code**

**2303.2.3 Testing.** All The front and back sides of the fire-retardant treated wood product shall be tested in accordance with and produce the results required in Section 2303.2. Wood structural panels shall be permitted to test only the front and back faces.

**Commenter's Reason:** There is no technical justification for requiring some fire-retardant treated wood products to be subjected to more testing due to the method of manufacture. During Committee Action Hearings, there was discussion of what exactly constitutes a side - do the ends have to be tested? That is actually not practical in the Steiner Tunnel, as you would have to fasten 864 small pieces together to make one specimen. If we assume that “side” excludes the ends (which is not at all obvious), the code still requires 4 times the amount of testing for products produced by other means than a pressure process, even though section 2303.2.2 states that all sides must be provided permanent protection. This modification would require all treated wood products to be tested on the front and back sides, leveling the playing field.
Proposed Change as Submitted

Proponent: Tim Earl, representing GBH International (tearl@gbhinternational.com)

2015 International Residential Code

Revise as follows:

R802.1.5.3 Testing. For wood products produced by other means during manufacture, other than a pressure process, all sides of the fire-retardant treated wood product shall be tested in accordance with and produce the results required in Section R802.1.5. Testing of only the front and back faces of wood structural panels shall be permitted.

Reason: As currently written, this section requires additional testing for fire-retardant treated wood materials that have been manufactured by a process different than pressure treatment. All fire retardant treated wood products must meet the requirements of section 2303.2 (which contains the fire test requirements). Moreover, all fire retardant wood products, irrespective of how they are manufactured, must meet the requirements (also contained in section 2303.2) that they must be impregnated with chemicals. A wood material can only be fire retardant treated wood if it is impregnated with chemicals and that will differentiate it from fire retardant coated materials. Testing requirements should be a function of performance and not of the way a product is made. There is no reason that some fire retardant treated wood materials should be treated in a different fashion by the code as a function of how they are manufactured.

A similar proposal is being made to IBC 2303.2.3.

Cost Impact: Will not increase the cost of construction
This proposal will reduce the cost for manufacturers of some materials by eliminating overly burdensome testing requirements based solely on how they are manufactured.

Public Hearing Results

Part II

Committee Action: Disapproved
Committee Reason: The committee felt the existing specified testing for fire-retardant treated wood is sufficient and additional testing is not needed.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Tim Earl, representing GBH International (tearl@gbhinternational.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Residential Code

R802.1.5.3 Testing. All the front and back sides of the fire-retardant treated wood product shall be tested in accordance with and produce the results required in Section R802.1.5. Testing of only the front and back faces of wood structural panels shall be permitted.

Commenter's Reason: There is no technical justification for requiring some fire-retardant treated wood products to be subjected to more testing due to the method of manufacture. During Committee Action Hearings, there was discussion of what exactly constitutes a side - do the ends have to be tested? That is actually not practical in the Steiner Tunnel, as you would have to fasten 864 small pieces together to make one specimen.
If we assume that "side" excludes the ends (which is not at all obvious), the code still requires 4 times the amount of testing for products produced by other means than a pressure process, even though section 2303.2.2 states that all sides must be provided permanent protection.

This modification would require all treated wood products to be tested on the front and back sides, leveling the playing field.
Committee Action: Disapproved

Assembly Action: None

**Proposed Change as Submitted**

Proponent: Marcelo Hirschler (gbhint@aol.com)

2015 International Building Code

Delete without substitution:

**2303.2.3 Testing.** For wood products produced by other means during manufacture, other than a pressure process, all sides of the wood product shall be tested in accordance with and produce the results required in Section 2303.2. Wood structural panels shall be permitted to test only the front and back faces.

Reason: This code section simply includes added testing requirements (and thus added burden) to fire-retardant treated wood materials that have been manufactured by a process different than pressure treatment. Note that all fire retardant treated wood products must meet the requirements of section 2303.2 (which contains the fire test requirements). Moreover, all fire retardant treated wood products, irrespective of how they are manufactured, must meet the requirements (also contained in section 2303.2) that they must be impregnated with chemicals. A wood material can only be fire retardant treated wood if it is impregnated with chemicals and that will differentiate it from fire retardant coated materials.

Testing requirements should be a function of performance and not of the way a product is made. There is no reason that some fire retardant treated wood materials should be treated in a different fashion by the code as a function of how they are manufactured.

If it is believed that it is important that all side of a fire retardant treated wood product be tested for fire safety, then 2303.2.3 can be rewritten as follows, in which case also all products are treated the same way, without differences as a function of how they are manufactured:

"2303.2.3 Testing. All sides of the fire retardant treated wood product shall be tested in accordance with and produce the results required in Section 2303.2. Wood structural panels shall be permitted to test only the front and back faces."

A proposal consistent with this proposal is also being made to IRC 802.1.5.

**Cost Impact:** Will not increase the cost of construction

This proposal will lower the excessive burden of fire testing for some materials as a function of how they are manufactured and not of their performance.

**Committee Reason:** The committee has a concern with the amount of conflicting testimony that is confusing this issue. A public comment is suggested with more written documentation to support this proposed change.

**Assembly Action:** None

**Individual Consideration Agenda**

Proponent: Marcelo Hirschler, representing GBH International (gbhint@aol.com) requests Approve as Submitted.

Commenter’s Reason: This section does nothing technically because the prior section already states that protection to all sides is required. There is no reason to have to test products in different ways as a function of how they are manufactured. This section does nothing other than add an excessive testing burden on manufacturers who choose a process other than pressure treatment for making fire retardant treated wood (FRTW). During the committee proposal stage there was a lot of discussion about the long history of FRTW: that is great and this proposal does not propose to deny that the product has been around for many years and performs well: the issue is not to provide a competitive advantage to one method of manufacturing FRTW. Testimony also discussed that there needs to be a distinction between products that are “impregnated” and products that are “coated”. This section does not do anything to address that issue because the test requirements can be “passed” without impregnating the...
wood, by just coating all sides. Moreover, since new test specimens are needed every time that a material is tested, if all sides are coated (or impregnated) how does a lab know which side has been tested and which side has not been tested? Top and bottom look the same (for panels) and the sides look the same (for products such as two by fours). Moreover, it is highly impractical (if not almost impossible) to test the ends of products such as two by fours in the ASTM E84 tunnel, since the sections would not be held in place but would fall into the tunnel.

This section serves no fire safety purpose as written.

The technical committee stated that the testimony was confusing and asked for further documentation.

Test specimens in the ASTM E84 test method must be 24 ft long by 20 to 24 inches wide and the maximum thickness is 4 inches. Test specimens can be provided in one of two ways: (1) a continuous, unbroken length; (2) sections that will be joined or butted end-to-end. Wood products are required to be tested using practice ASTM E2579, which (as stated in ASTM E84) applies to the following wood products: “solid board, lumber and timber products (including solid boards, lumber, timber, fingerjoined lumber, glulam, laminate wood, laminated veneer lumber and parallel strand lumber products), panel products (including fibreboard, hardboard, oriented strandboard, waferboard, and plywood panel products), decorative wood products (including fine woodwork, millwork and moulding) and shingles and shakes used as interior wall and ceiling finish and interior trim as well as to laminated products factory-produced with a wood substrate”, which means that it applies to all wood products. Therefore if we want to test ends of products they would have to be cut to 4 inch lengths and nailed or fastened or glued together and it would require a very large number (actually 864, for a 2 x 4) of small pieces, which is not practical.
Committee Action: Approved as Submitted

Assembly Action: None

---

Proposed Change as Submitted

Proponent: Joseph Holland, representing Hoover Treated Wood Products (jholland@frtw.com)

2015 International Building Code

Revise as follows:

2303.2.4 Labeling. Fire-retardant-treated lumber and wood structural panels shall be labeled in addition to the labels required in Section 2303.1.1 for sawn lumber and Section 2303.1.5 for wood structural panels each piece of fire-retardant-treated lumber and wood structural panels shall be labeled. The label shall contain the following items:

1. The identification mark of an approved agency in accordance with Section 1703.5.
2. Identification of the treating manufacturer.
3. The name of the fire-retardant treatment.
4. The species of wood treated.
5. Flame spread and smoke-developed index.
7. Conformance with appropriate standards in accordance with Sections 2303.2.5 through 2303.2.8.
8. For fire-retardant-treated wood exposed to weather, damp or wet locations, include the words “No increase in the listed classification when subjected to the Standard Rain Test” (ASTM D 2898).

Reason: There are products coming into the marketplace that have obscured the labels required by Section 2303.1.1 and 2303.1.5. This change clarifies that FRTW must have two labels: one for the grading of the wood the other for the treatment. There are also manufacturers making the claim for a lift of lumber or wood structural panel. The change clarifies each piece must be labeled with both marks.

Cost Impact: Will not increase the cost of construction

Manufacturer’s treating in accordance with the code requirement for pressure treatment or other means during manufacturer already mark each piece. The proposal clarifies, for others, what is already being done.

---

Public Hearing Results

Committee Action: Approved as Submitted

Committee Reason: This proposal provides a clarification of the labeling of fire-retardant-treated wood that aides verification in the field.

Assembly Action: None

---

Individual Consideration Agenda

Public Comment 1:

Proponent: Joseph Holland, Hoover Treated Wood Products, representing Hoover Treated Wood Products (jholland@frtw.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

2303.2.4 Labeling. Fire-retardant-treated lumber and wood structural panels shall be labeled in addition to the labels required in Section 2303.1.1 for sawn lumber and Section 2303.1.5 for wood structural panels each piece of fire-retardant-treated lumber and wood structural panels shall be labeled. The label shall contain the following items:

1. The identification mark of an approved agency in accordance with Section 1703.5.
2. Identification of the treating manufacturer.
3. The name of the fire-retardant treatment.
4. The species of wood treated.
5. Flame spread and smoke-developed index.
7. Conformance with appropriate standards in accordance with Sections 2303.2.5 through 2303.2.8.
8. For fire-retardant-treated wood exposed to weather, damp or wet locations, include the words "No increase in the listed classification when subjected to the Standard Rain Test" (ASTM D 2898).

**Commenter's Reason:** During the deliberation on this change the phrase "each piece of" was questioned as to the exact meaning. We agreed with the concern and stated we would fix during the comment stage. This modification better reflects the intent of the change. The intent was to clarify that there must be two stamps on fire-retardant-treated wood: the grade stamp and the treating stamp.

S265-16
Proposed Change as Submitted

Proponent: Edwin Huston, representing National Council of Structural Engineers' Associations (NCSEA)
(huston@smithhustoninc.com)

2015 International Building Code

Revise as follows:

2303.4.1.2 Permanent individual truss member restraint. Where permanent restraint of truss members is required 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.

Method 1 is not allowed where any of the following conditions occur:

1.1 The trusses are installed in locations where the ultimate design wind speed $V_{uH}$, as defined in Section 1609, is 120 miles per hour or higher.
1.2 The trusses are installed in locations where the ground snow loads $P_D$ is 30 pounds per square foot or higher.
1.3 Where more than one permanent restraint is required on any single truss web member.

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.

Reason: This code change will clarify that permanent individual truss member restraint / bracing must be designed and detailed by a registered design professional for trusses located in higher load / risk situations. Standard industry details, such as those in the Building Component Safety Information (BCSI) documents published by the Structural Building Components Association (SBCA) and the Truss Plate Institute (TPI), do not address restraint/bracing in high wind or high snow areas. The issue of wood trusses being installed without adequate permanent individual truss member restraint/bracing is a life safety concern. By relying on "standard industry details", the public is at risk if the restraint/bracing is not adequate or more importantly, is not installed correctly. This code change seeks to prevent a truss failure in higher load, higher risk situations as outlined.

Cost Impact: Will increase the cost of construction

This code change has no impact on a majority of the jurisdictions in the country, and therefore there is no cost implications in those areas. The cost of construction in the high risk areas will increase slightly by the amount of the cost of a registered design professional designing and detailing the bracing. This increased cost however is minimal and justified.

Committee Action: Disapproved

Committee Reason: The proposed wording is confusing in terms of the exception and the triggers set in this section were not substantiated. Furthermore, this wording would prohibit the use of standard industry truss bracing details. This is typically how it's done - this change would not allow it.

Assembly Action: None

Individual Consideration Agenda

Proponent: Scott Campbell, representing Portland Cement Association (scampbell@cement.org) requests Approve as Submitted.
Commenter's Reason: The proposed change requires a design professional to design permanent lateral bracing of truss members in a limited number of cases where life safety could be jeopardized by using industry standard designs that are not appropriate for the design conditions. It does not prohibit the use of standard designs, only requiring design professional input in those few cases where current industry standard designs are not applicable.
S268-16
IBC: 2303.4.6.

Proposed Change as Submitted

Proponent: Edwin Huston, representing National Council of Structural Engineers' Associations (NCSEA) (huston@smithhustoninc.com)

2015 International Building Code

Revise as follows:

2303.4.6 TPI 1 specifications. In addition to Sections 2303.4.1 through 2303.4.5, the design, manufacture and quality assurance of metal-plate-connected wood trusses shall be in accordance with TPI 1, modified as follows:

1. Job-site inspections shall Delete TPI 1 Section 2.3.3.2.
2. Replace TPI 1 Section 2.3.5.5(o) with the following:
   (o) Required permanent individual truss member restraint location and the method and details of restraint/bracing to be in compliance with Section 110.4, as applicable used.

Reason: This code change is needed to harmonize TPI 1-2014 with the current language in IBC Section 2303.4

The language regarding truss member restraint/bracing has been extensively discussed and coordinated through a consensus process over the last several Code cycles. The changes to TPI 1-2014 unacceptably alters both the language and the intent of the IBC Code requirements.

TPI 1 Section 2.3.3.2 Absence of Truss Restraining Method or Details, is intended for structures not required to be permitted under the International Building Code and thus should be deleted. IBC Section 2303.4.3 requires the method and details to be delivered at the job site as part of the truss submittal package. TPI 1 section 2.3.3.2 may be construed by some that they do not need to submit the method and details in the truss submittal package. There should never be an "absence" of restrain/bracing details. If the construction documents do not specify a restraint/bracing method, then the Truss Designer must select the method to be used, and provide details as required by IBC Section 2303.4.3.

TPI 1-2014 Section 2.3.5.5(o) altered the language from that in TPI 1-2007, and differs from IBC Section 2303.4.1.1, item #14. The proposed change revises the TPI 1 language to be the same as is in the IBC section.

Cost Impact: Will not increase the cost of construction

All of the changes are clarifications only and do no change the cost of construction, since they maintain the current status of the IBC.

Public Hearing Results

Committee Action: Disapproved
Committee Reason: The committee does not believe that the proposed modifications to the referenced standard, TPI 1, are needed.

Assembly Action: None

Individual Consideration Agenda

Proponent: Edwin Huston, representing National Council of Structural Engineers' Associations (NCSEA) (huston@smithhustoninc.com) requests Approve as Submitted.

Commenter's Reason: NCSEA strongly feels that the Committee misunderstood the intent of this Code Change Proposal and did not realize the importance of this change when they reviewed the text. As currently published, TPI 1-2014 has changed what the intent and wording of IBC Section 2303.4 states, and without this code change there will be confusion and disagreement.

The language regarding truss member restraint/bracing has been extensively discussed and coordinated through a consensus process over the last several Code cycles. The changes to TPI 1-2014 unilaterally and unacceptably alters both the language and the intent of the IBC Code requirements. The code change proposal is needed to harmonize TPI 1-2014 with the current
language in IBC Section 2303.4.

Important points to consider:

1) “TPI 1 Section 2.3.3.2 Absence of Truss Restrain/Bracing Method or Details”: this section is intended for structures not required to be permitted under the International Building Code and thus should be deleted from the IBC. IBC Section 2303.4.3 requires the method and details to be delivered to the job site as part of the truss submittal package. TPI 1 section 2.3.3.2 may be construed by some to mean that they do not need to submit the method and details in the truss submittal package. There should never be an “absence” of restraint/bracing details. If the construction documents do not specify a restraint/bracing method, then the Truss Designer must select the method to be used, and provide details as required by IBC Section 2303.4.3.

2) TPI 1 -2014 Section 2.3.5.5(o) altered the language from that in TPI 1-2007, and differs from IBC Section 2303.4.1.1, item #14. The proposed IBC Code change reverses the TPI 1 language to match the language in the IBC section. This discrepancy will be an issue for practicing structural engineers, if not corrected.

The IBC modifies other standards in similar ways, either by modifying a particular section, such as 1613.5, 1905.1, 2107.3, or 2108.2; or by not adopting a section, such as ASCE 7 Chapter 14.
S277-16

Proposed Change as Submitted

Proponent: Edward Keith, APA- The Engineered Wood Association, representing APA- The Engineered Wood Association (ed.keith@apawood.org)

2015 International Building Code

Revise as follows:

2304.12.1 Locations requiring water-borne preservatives or naturally durable wood. Wood used above ground in the locations specified in Sections 2304.12.1.1 through 2304.12.1.5, 2304.12.3 and 2304.12.5 shall be naturally durable wood or preservative-treated wood using water-borne preservatives, in accordance with AWPA U1 for above-ground use.

2304.12.2 Other locations. Wood used in the locations specified in Sections 2304.12.2.1 through 2304.12.2.5 shall be naturally durable wood or preservative-treated wood in accordance with AWPA U1. Preservative-treated wood used in interior locations shall be protected in accordance with two coats of urethane, shellac, latex epoxy or varnish unless water-borne preservatives are used in the treating company's recommendations. Prior to application of the protective finish, the wood shall be dried in accordance with the manufacturer's recommendations.

Reason: Preservative treatments recognized by AWPA standards are not limited to water-borne treatment. Therefore, it is proposed that Section 2304.12.1 be revised to recognize other treatments. With this change, Section 2304.12.2 is required to be updated as the specified protection finish may not be appropriate for interior locations for non-waterborne preservative treatments. A reference to the treatment company's recommendations is a common practice in this regard.

In Section 2304.12.2, the code provisions cover other locations that require protection of wood against decay or termites. However, the subsection of the existing 2304.12.2.4 specifies requirements for laminated timber. This seems to be out of place and makes it difficult for the user of the code to find the information. An appropriate place seems to be in a new Section 2304.12.1.6, where sleepers and sills, and wood siding are located. In fact, the requirements in the existing Section 2304.12.2.4 should be applied to all exposed wood members, but not limited to laminated timbers. Therefore, it is proposed that this section be revised as suggested.

Cost Impact: Will not increase the cost of construction

This code change will not increase the cost of construction as it simply recognizes the general intent of using preservative-treated wood under exposed conditions.

S277-16:
2304.12.1-
KEITH11102

Public Hearing Results

Committee Action: Disapproved

Committee Reason: The committee believed the proposal was more confusing and the proposed modifications did not clear that up. There was some sentiment for tabulating these requirements for preservative treatment. The committee suggests a public comment to clarify the proposal.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Borjen Yeh, representing APA - The Engineered Wood Association (borjen.yeh@apawood.org); Edward Keith (ed.keith@apawood.org) requests Approve as Modified by this Public Comment.

Replace Proposal as Follows:
2015 International Building Code

2304.12 Protection against decay and termites. Wood shall be protected from decay and termites in accordance with the applicable provisions of Sections 2304.12.1 through 2304.12.7. 2304.12.8.

2304.12.1 Locations requiring water-borne preservatives or naturally durable wood. Wood used above ground in the locations specified in Sections 2304.12.1.1 through 2304.12.1.5, 2304.12.3 and 2304.12.5 shall be naturally durable wood or preservative-treated wood using water-borne preservatives, in accordance with AWPA U1 for above-ground use.

2304.12.1.1 Joists, girders and subfloor. Wood joists or wood structural floors that are closer than 18 inches (457 mm) or wood girders that are closer than 12 inches (305 mm) to the exposed ground in crawl spaces or unexcavated areas located within the perimeter of the building foundation shall be of naturally durable or preservative-treated wood.

2304.12.1.2 Wood supported by exterior foundation walls. Wood framing members, including wood sheathing, that are in contact with exterior foundation walls and are less than 8 inches (203 mm) from exposed earth shall be of naturally durable or preservative-treated wood.

2304.12.1.3 Exterior walls below grade. Wood framing members and furring strips in direct contact with the interior of exterior masonry or concrete walls below grade shall be of naturally durable or preservative-treated wood.

2304.12.1.4 Sleepers and sills. Sleepers and sills on a concrete or masonry slab that is in direct contact with earth shall be of naturally durable or preservative-treated wood.

2304.12.1.5 Wood siding. Clearance between wood siding and earth on the exterior of a building shall not be less than 6 inches (152 mm) or less than 2 inches (51 mm) vertical from concrete steps, porch slabs, patio slabs and similar horizontal surfaces exposed to the weather except where siding, sheathing and wall framing are of naturally durable or preservative-treated wood.

2304.12.2 Other locations. Wood used in the locations specified in Sections 2304.12.2.1 through 2304.12.2.6 shall be naturally durable wood or preservative-treated wood in accordance with AWPA U1. Preservative-treated wood used in interior locations shall be protected with two coats of urethane, shellac, latex epoxy or varnish unless water-borne preservatives are used. Prior to application of the protective finish, the wood shall be dried in accordance with the manufacturer's recommendations.

2304.12.2.1 Girder ends. The ends of wood girders entering exterior masonry or concrete walls shall be provided with a 1/2-inch (12.7 mm) airspace on top, sides and end, unless naturally durable or preservative-treated wood is used.

2304.12.2.2 Posts or columns. Posts or columns supporting permanent structures and supported by a concrete or masonry slab or footing that is in direct contact with the earth shall be of naturally durable or preservative-treated wood.

• Exception: Posts or columns that are not exposed to the weather, are supported by concrete piers or metal pedestals projected at least 1 inch (25 mm) above the slab or deck and 8 inches (203 mm) above exposed earth and are separated by an impervious moisture barrier.

2304.12.2.3 Supporting member for permanent appurtenances. Naturally durable or preservative-treated wood shall be utilized for those portions of wood members that form the structural supports of buildings, balconies, porches or similar permanent building appurtenances where such members are exposed to the weather without adequate protection from a roof, eave, overhang or other covering to prevent moisture or water accumulation on the surface or at joints between members.

• Exception: When a building is located in a geographical region where experience has demonstrated that climatic conditions preclude the need to use durable materials where the structure is exposed to the weather.

2304.12.2.4 Supporting members for permeable floors and roofs. No change to text.

2304.12.3 Wood in contact with the ground or fresh water. Wood used in contact with exposed earth shall be naturally durable for both decay and termite resistance or preservative treated in accordance with AWPA U1 for soil or fresh water use.

• Exception: Untreated wood is permitted where such wood is continuously and entirely below the ground-water level or submerged in fresh water.

2304.12.3.1 Posts or columns. Posts and columns that are supporting permanent structures and embedded in concrete that is exposed to the weather or in direct contact with the earth shall be of preservative-treated wood.

2304.12.4 Termite protection. In geographical areas where hazard of termite damage is known to be very heavy, wood floor framing in the locations specified in Section 2304.12.2.1 and exposed framing of exterior decks or balconies shall be of naturally durable species (termite resistant) or preservative treated in accordance with AWPA U1 for the species, product preservative and end use or provided with approved methods of termite protection.

2304.12.5 Wood used in retaining walls and cribs. Wood installed in retaining or crib walls shall be preservative treated in accordance with AWPA U1 for soil and fresh water use.

2304.12.6 Laminted timbers. The portions of glued-laminated timbers that form the structural supports of a
building or other structure and are exposed to weather and not fully protected from moisture by a roof, eave or similar covering shall be pressure treated with preservative in accordance with AWPA U1 or be manufactured from naturally durable or preservative-treated wood. Preservative-treated wood used in interior locations shall be protected with two coats of urethane, shellac, latex epoxy or varnish unless water-borne preservatives are used. Prior to application of the protective finish, the wood shall be dried in accordance with the manufacturer's recommendations.

2304.12.6 2304.12.7 Attic ventilation. No change to text.

2304.12.7 2304.12.8 Under-floor ventilation (crawl space). No change to text.

Commenter's Reason: The original proposal S277-16 was intended to clarify that preservative treatments for glued-laminated timber (glulam) recognized by AWPA U1 are not limited to applications / locations specified in Section 2304.12.2 “Other Locations.” In fact, preservative treated glulams have been successfully used in locations specified in Section 2304.12.1, such as joists and girders, for so many years. However, as the current glulam requirements are specified as a subsection (2304.12.2.4) under 2304.12.2, it has caused confusion that glulam can only be used under 2304.12.2 “Other Locations” and cannot be used in those locations listed under 2304.12.1.

Based on comments received at the Committee Action Hearing, we believe the proposed changes in this public comment could address this issue without causing additional confusion by those changes as originally proposed (see the Committee Reason from the Committee Action). The proposed changes in this public comment achieve the following:

1) Move the glulam sub-section 2304.12.2.4 to 2304.12.6 (one level up) to make sure glulams are not limited to “Other Locations” listed under 2304.12.2.

2) Add the wording of “in accordance with AWPA U1” to the paragraph to cover all preservative treatment methods (water-borne or non-water-borne) recognized by AWPA U1 for glulams.

3) Add the same wording as that included in the current Section 2304.12.2 for the protection of non-water-borne treated wood when used in interior locations.

4) Re-number the rest of sections.

There will be no cost impact to construction due to this code change. Please support this public comment to clarify the confusion in the existing code.

S277-16
Proposed Change as Submitted

Proponent: Randy Shackelford, Simpson Strong-Tie, representing Simpson Strong-Tie (rshackelford@strongtie.com)

2015 International Building Code

Revise as follows:

2304.12.2.2 Posts or columns. Posts or columns supporting permanent structures and supported by a concrete or masonry slab or footing that is in direct contact with the earth shall be of naturally durable or preservative-treated wood.

Exception: Posts or columns that are not exposed to the weather without adequate protection as specified in Section 2304.12.2.3, or are located in basements or cellars, and are supported by concrete piers or metal pedestals projecting at least 1 inch (25 mm) above the slab or deck and 8 inches (203 mm) above exposed earth, and are separated therefrom by an impervious moisture barrier.

Reason: The purpose of this code change is to return the text of this section to be more closer to the text that existed in the 2000-2012 IBC, without creating a conflict with Section 2304.12.2.3.

For the 2015 IBC, the American Wood Council did a major re-write of 2304.12 on Protection against decay and termites. As part of that, they completely changed the meaning of this section by adding the word "not" to the first sentence of the exception.

From 2000 to 2102, this exception has read “Posts and columns that are either exposed to the weather or located in basements or cellars, supported by concrete piers or metal pedestals projected at least 1 inch (25 mm) above the slab or deck and 6 inches (152 mm) above exposed earth, and are separated therefrom by an impervious moisture barrier.”

2000 and 2003 IBC: Section 2304.11.2.6
2006, 2009, and 2012 IBC: Section 2304.11.2.7.

The AWC code change that was accepted was S268-12. Its only statement about this section was that “The first exception was worded incorrectly and would seem to exempt exposed wood from protection; the proposed wording is a fix.” I am not sure you can say definitively that this was worded incorrectly since it was exactly this way in 5 editions of the IBC from 2000 to 2012.

Another AWC code change that was disapproved, S271-12, made a similar change, and noted that as written this section conflicts with current 2304.12.2.3, Supporting member for permanent appurtenances. The only conflict is that section states that naturally durable or preservative-treated wood must be utilized “where such members are exposed to the weather without adequate protection from a roof, eave, overhang, or other covering to prevent moisture or water accumulation on the surface or at joints between members.”

This seems to be a reasonable requirement, and describes well what is considered to be “exposed”. However, the language in 2304.12.2.2 now simply says “not exposed to the weather”, which could easily be interpreted to exempt any outdoor wood member.

So this proposal attempts to better define exposed to the weather by referencing the clearer description in 2304.12.2.3.

A second modification changes “projected” to “projecting”, which sounds like it better describes the situation. Projected sounds like something you do to a film.

A third modification reinstates the word “therefrom” because it seems to improve the meaning.

Cost Impact: Will not increase the cost of construction
No cost impact. Possible cost savings.
This proposal may allow the use of non-treated or non-naturally durable wood where it is protected from moisture by a covering above to prevent moisture or water accumulation and is supported by a 1” pedestal. The option remains to use durable or treated wood and not use the base with 1” pedestal.

Committee Action: Approved as Modified

Modification:
2304.12.2.2 Posts or columns. Posts or columns supporting permanent structures and supported by a concrete or masonry slab or footing that is in direct contact with the earth shall be of naturally durable or preservative-treated wood.

- **Exception:** Posts or columns that meet all of the following:
  1. Are not exposed to the weather, or are protected by a roof, eave, overhang, or other covering if exposed to the weather, and
  2. Are supported by concrete piers or metal pedestals projecting at least 1 inch (25 mm) above the slab or deck and are separated from the concrete pier by an impervious moisture barrier.
  3. Are located at least 8 inches (203 mm) above exposed earth.

Committee Reason: The rewording of the exception for posts and columns is an improvement that explains when and how to provide protection for posts supported on concrete or masonry. The modification reformats the exception as a list so that will be easier to understand.

Assembly Action: None

---

**Individual Consideration Agenda**

**Proponent:** Scott Campbell, representing Portland Cement Association (scampbell@cement.org) requests Disapprove.

**Commenter's Reason:** Adequately protecting exposed wood elements from moisture is crucial to producing a structure that will provide for the public safety and also provide satisfactory property protection. Wood elements located under a roof, eave, overhang or other covering are not necessarily protected from the weather, potentially allowing water to accumulate at the base of wood elements. Adequately protecting such elements requires the use of naturally durable or preservative-treated wood as currently specified in the IBC.

S278-16
Proposed Change as Submitted

Proponent: Dennis Richardson, American Wood Council, representing American Wood Council (drichardson@awc.org)

2015 International Building Code

Revise as follows:

2304.12.2.5 Supporting members for permeable floors and roofs. Wood structural members that support moisture-permeable floors or roofs that are exposed to the weather, such as concrete or masonry slabs, shall be of naturally durable or preservative-treated wood unless separated from such floors or roofs by an impervious moisture barrier. The impervious moisture barrier system protecting the structure supporting floors shall include elements providing positive drainage of water that infiltrates the moisture-permeable floor topping.

Reason: A key functional requirement of impervious moisture barrier systems installed under a permeable floor system exposed to water are elements that provide for drainage of any water making its way through the permeable floor system. Without a properly functioning method to transport this water out, the floor assembly can stay saturated for very long periods of time possibly contributing to premature failure. This code proposal creates a requirement for impervious moisture barrier systems protecting the structure, supporting a floor, to provide a mechanism for the water to drain out.

Cost Impact: Will increase the cost of construction

Drainage elements between the permeable floor slab and impervious barrier are commonly called for and installed by many practitioners and will not change the cost of construction in those cases. However in cases where no method to provide positive drainage is currently provided, this proposal will increase the cost of construction.

Public Hearing Results

Committee Action: Disapproved

Committee Reason: The proposed language on impervious moisture barriers is not clear enough for the building official to enforce. The requirement for "elements providing positive drainage" should be clarified. The committee recognizes that this proposal would address a serious issue that needs to be dealt with and a public comment is encouraged to address the committee's concerns.

Assembly Motion: As Submitted

Online Vote Results: Failed
Support: 40.07% (107) Oppose: 59.93% (160)

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Dennis Richardson, representing American Wood Council (drichardson@awc.org) requests Approve as Modified by this Public Comment.

Modify as follows:

2015 International Building Code

2304.12.2.5 Supporting members for permeable floors and roofs. Wood structural members that support moisture-permeable floors or roofs that are exposed to the weather, such as concrete or masonry slabs, shall be of naturally durable or preservative-treated wood unless separated from such floors or roofs by an impervious moisture barrier. The impervious moisture barrier system protecting the structure supporting floors shall include elements providing positive drainage of water that infiltrates the moisture-permeable floor topping.

Commenter's Reason: This existing code section applies when wood (that is not preservative-treated or naturally durable) supports moisture-permeable floors or roofs exposed to weather such as concrete or masonry slabs.
When such assemblies are a roof, and there is a leak in the impervious barrier, the occupants typically know about it and repairs are made. When the assembly supports a walking surface such as a balcony, there may be no early warning of a leak or decay because any leak may be located over unoccupied areas outside of the structure building envelope so the leak remains undetected.

Balcony structure performance is critical because they may see substantial loading when the balcony is occupied by several persons and balconies can be located several stories above grade. Structural failure of a balcony can result in multiple serious injuries or deaths.

In this code section, the existing requirement calls for separation by an impervious moisture barrier when the supporting wood is not preservative-treated or naturally durable. The term “impervious moisture barrier” is not defined in the code but really describes the required performance of the barrier. One bit of testimony during the Committee Action Hearing was existing language in 2304.12.2.5 may be unclear as it currently exists.

Other code changes affecting balconies were approved at the Committee Action Hearing:
ADM77-16 requires detailing on plans of all elements of the impervious moisture barrier system (including manufacturer’s instructions when applicable) if the impervious moisture barrier option is used.
ADM87-16 requires inspection of all elements of the impervious moisture barrier system or special inspection can be utilized at the option of the code official.
S85-16 increased the live load for balconies to be consistent with live load requirements in ASCE-7.
S289-16 was disapproved on a close vote decided by the Chair. In their reason statement the Committee acknowledged this proposal would address a serious issue that needs to be dealt with and a public comment is encouraged to address the committee’s concerns.

Early initial approaches to this code change as well as ADM77-16 and ADM87-16 were to include a comprehensive list of the various elements that might make up an impervious moisture barrier system. The proponent of these code changes received substantial feedback not to include a laundry list of possible elements that commonly make up these systems as the elements are not always the same for different systems and configurations. That logic was supported by the committee with the approval of ADM 77-16 and ADM 87-16.

Since the initial Group B code change deadline, an article by Joseph Lstiburek has been published in the ASHRAE Journal. The unedited version can be found on the author’s website at the following link:

Two key concepts covered in this document is the need to provide slope, and when the traffic surface is permeable (like a concrete or masonry surface), then “it is critical that a drainage layer or space is provided immediately above the waterproofing layer.” The article gives additional emphasis to the word “critical”.

Without slope and a way for the water to get out, the impervious moisture barrier can be subject to constant attack by water that infiltrates the moisture permeable topping slab in a wet environment.
This concept is similar to a weep screed that provides a path for water to get out of the wood wall covered with plaster. Without an effective functioning weep screed there can be substantial water damage leading to the decay of the structural elements.

Because the overall code section is performance based, it is not possible to write a cookbook method to address this from a design standpoint. Articles such as the one linked to this reason statement do help the designer with some guidance as do manufacturer’s instructions and recommendations. The key point though is just as with a weep screed, there needs to be positive drainage for moisture to get out.

There may be time to fully to address concerns of the existing language found in Section 2304.12.2.5 for the 2021 IBC code cycle. That is outside of the scope of the public comment process. Since existing language will be in place for at least three more years, this public comment at least makes it clear to designers of the need to consider and provide positive drainage of water that infiltrates the moisture permeable floor topping.

As the committee said this is a serious issue in the code that needs to be dealt with.

Information on this and other code change proposals by American Wood Council may be found at the following web address:
Committee Action: Approved as Submitted

Assembly Action: None

2015 International Building Code

Revise as follows:

2407.1.1 Loads. The panels and their support system shall be designed to withstand the loads specified in Section 1607.8. A design, using a safety factor of four shall be used for safety.

Reason: The purpose of this proposal is to return the code language to well-recognized terms, and eliminate terms that have no meaning to the engineers that will be performing the designs of these panels and supports. In the last cycle, proposal S300-12 was Approved as Submitted at the Final Action Hearings. That proposal substituted the phrase, "design factor...for safety" for "safety factor." The latter is a well-recognized engineering term, whereas the former is not. Unfortunately, there was no opportunity at the FAH to make any changes to the proposal, editorial or otherwise.

This proposal does not change the meat of the code or the intent of S300-12, but is an editorial change that will be more understandable to the engineering community who will be responsible for these designs.

Cost Impact: Will not increase the cost of construction

This is an editorial change to clarify the code. It does not change any requirements of the code, and therefore, has no cost impact.

S296-16
2407.1.1-
SIU11093

Public Hearing Results

Committee Reason: approval is consistent with action taken on S295-16.

Assembly Reason: None

Individual Consideration Agenda

Proponent: Jonathan Siu, representing City of Seattle Department of Construction and Inspections (Jon.Siu@seattle.gov) requests Approve as Modified by this Public Comment.

Modify as follows:

2015 International Building Code

2407.1.1 Loads. The panels and their support system shall be designed to withstand the loads specified in Section 1607.8. Glass guard elements shall be designed using a safety factor of four.

Commenter's Reason: This public comment corrects the original proposal, and clarifies to which elements in a glass guard system the safety factor of 4 is to be applied. The proposed change was intended to be purely editorial. However, as written by the proponent and approved by the Committee, the proposal incorrectly made clear the safety factor was to be applied to the structural elements supporting the glass guard elements.

A safety factor of 4 is necessary for glass elements because it is known there can be an extreme variation in structural properties for glass. Any small defect can cause the glass to fail prematurely--something that is not desireable for an element that is supposed to keep a person from falling. However, it is not necessary to design the non-glass guard elements for same safety factor, since their structural properties are much more predictable. For those non-glass elements, normal safety factors built into their structural design parameters would be adequate. We do not believe it is the intent of the code to penalize other structural elements of guards, just because they are supporting glass. It is to be noted that if the supports for a glass guard system (top rail, connections, etc.) are required to be designed for a safety factor of 4, it will be extremely difficult to comply with the code.
Proposed Change as Submitted

Proponent: Bonnie Manley, AISI, representing American Iron and Steel Institute (bmanley@steel.org)

2015 International Building Code

Revise as follows:

### TABLE 2506.2

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>STANDARD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Accessories for gypsum board</td>
<td>ASTM C1047</td>
</tr>
<tr>
<td>Adhesives for fastening gypsum board</td>
<td>ASTM C557</td>
</tr>
<tr>
<td>Cold-formed steel studs and track, structural</td>
<td>AISI S240 and ASTM C 955, Section 8</td>
</tr>
<tr>
<td>Cold-formed steel studs and track, nonstructural</td>
<td>AISI S220 and ASTM C 645, Section 10</td>
</tr>
<tr>
<td>Elastomeric joint sealants</td>
<td>ASTM C 920</td>
</tr>
<tr>
<td>Fiber-reinforced gypsum panels</td>
<td>ASTM C 1278</td>
</tr>
<tr>
<td>Glass mat gypsum backing panel</td>
<td>ASTM C 1178</td>
</tr>
<tr>
<td>Glass mat gypsum panel 5</td>
<td>ASTM C 1658</td>
</tr>
<tr>
<td>Glass mat gypsum substrate</td>
<td>ASTM C 1177</td>
</tr>
<tr>
<td>Joint reinforcing tape and compound</td>
<td>ASTM C 474; C 475</td>
</tr>
<tr>
<td>Nails for gypsum boards</td>
<td>ASTM C 514, F 547, F 1667</td>
</tr>
<tr>
<td>Steel screws</td>
<td>ASTM C 954; C 1002</td>
</tr>
<tr>
<td>Standard specification for gypsum board</td>
<td>ASTM C 1396</td>
</tr>
<tr>
<td>Testing gypsum and gypsum products</td>
<td>ASTM C 22; C 472; C 473</td>
</tr>
</tbody>
</table>

### TABLE 2507.2

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>STANDARD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Accessories for gypsum veneer base</td>
<td>ASTM C1047</td>
</tr>
<tr>
<td>Blended cement</td>
<td>ASTM C595</td>
</tr>
<tr>
<td>Exterior plaster bonding compounds</td>
<td>ASTM C932</td>
</tr>
<tr>
<td>Cold-formed steel studs and track, structural</td>
<td>AISI S240 and ASTM C 955, Section 8</td>
</tr>
<tr>
<td>Cold-formed steel studs and track, nonstructural</td>
<td>AISI S220 and ASTM C 645, Section 10</td>
</tr>
<tr>
<td>Hydraulic cement</td>
<td>ASTM C 1157; C 1600</td>
</tr>
<tr>
<td>Gypsum casting and molding plaster</td>
<td>ASTM C 59</td>
</tr>
<tr>
<td>Gypsum Keene's cement</td>
<td>ASTM C 61</td>
</tr>
<tr>
<td>Gypsum plaster</td>
<td>ASTM C 28</td>
</tr>
<tr>
<td>Gypsum veneer plaster</td>
<td>ASTM C 587</td>
</tr>
<tr>
<td>Interior bonding compounds, gypsum</td>
<td>ASTM C 631</td>
</tr>
<tr>
<td>Lime plasters</td>
<td>ASTM C 5; C 206</td>
</tr>
<tr>
<td>Masonry cement</td>
<td>ASTM C 91</td>
</tr>
<tr>
<td>Metal lath</td>
<td>ASTM C 847</td>
</tr>
<tr>
<td>Plaster aggregates</td>
<td>ASTM C 35; C 897</td>
</tr>
<tr>
<td>Sand</td>
<td>ASTM C 35</td>
</tr>
<tr>
<td>Perlite</td>
<td>ASTM C 35</td>
</tr>
<tr>
<td>Vermiculite</td>
<td>ASTM C 35</td>
</tr>
<tr>
<td>Plastic cement</td>
<td>ASTM C 1328</td>
</tr>
<tr>
<td>Portland cement</td>
<td>ASTM C 150</td>
</tr>
<tr>
<td>Steel screws</td>
<td>ASTM C 1002; C 954</td>
</tr>
<tr>
<td>Welded wire lath</td>
<td>ASTM C 933</td>
</tr>
<tr>
<td>Woven wire plaster base</td>
<td>ASTM C 1032</td>
</tr>
</tbody>
</table>

**Reference standards type:** This contains both new and updated standards

**Add new standard(s) as follows:**

- AISI S200—12, North American Standard for Cold-Formed Steel Framing-General Provisions, 2012, 2203.1, 2203.2, 2211.1, Table 2603.12.1, Table 2603.12.2
- AISI S240, North American Standard for Cold-Formed Steel Structural Framing, 2015

**Update the following existing reference:**

- AISI S220 1415, North American Standard for Cold-Formed Steel Framing -- Nonstructural Members, 2015
Reason: This proposal is one in a series adopting the latest generation of AISI standards for cold-formed steel. This particular proposal focuses on Chapter 25 by incorporating a reference to the new cold-formed steel structural framing standard – AISI S240. Additionally, it amends existing language to reflect updates made to the existing cold-formed steel nonstructural framing standard – AISI S220. The update to AISI S220 is being handled through the administrative update process. Both standards are published and available for a free download at: www.aisistandards.org.

The new standard, **AISI S240, North American Standard for Cold-Formed Steel Structural Framing**, addresses requirements for construction with cold-formed steel structural framing that are common to prescriptive and engineered light frame construction. This comprehensive standard was formed by merging the following AISI standards:

- **AISI S200, North American Standard for Cold-Formed Steel Framing-General Provisions**
- **AISI S210, North American Standard for Cold-Formed Steel Framing–Floor and Roof System Design**
- **AISI S211, North American Standard for Cold-Formed Steel Framing–Wall Stud Design**
- **AISI S212, North American Standard for Cold-Formed Steel Framing–Header Design**
- **AISI S213, North American Standard for Cold-Formed Steel Framing–Lateral Design**
- **AISI S214, North American Standard for Cold-Formed Steel Framing–Truss Design**

Consequently, AISI S240 supersedes all previous editions of the above mentioned individual AISI standards.

The updated 2015 edition of **AISI S220, North American Standard for Cold-Formed Steel Framing—Nonstructural Members**, continues to address requirements for construction with nonstructural members made from cold-formed steel. This standard provides an integrated treatment of Allowable Strength Design (ASD), and Load and Resistance Factor Design (LRFD). This is accomplished by including the appropriate resistance factors (φ) for use with LRFD, and the appropriate factors of safety (Ω) for use with ASD. The following major revisions were made in the 2015 edition:

- Performance requirements for screw penetration were added in Section A6.6.
- Referenced documents in Section A7 were updated.
- Errata in Section B1(b) was fixed; i.e., "using $\beta_0 = 1.6$" was added.
- Testing requirements were expanded in Section F1 to reference the new AISI S916 Test Standard, when required to determine the strength and stiffness of composite nonstructural interior partition wall assemblies.
- Testing requirements were added in Section F2 to reference the new AISI S915 Test Standard, when required to determine the strength and deformation behavior of bridging connectors.
- Testing requirements for screw penetration were added in Section F3, and the test method was added in Appendix 1.

Both Table 2506.2 and Table 2507.2 previously referenced AISI S200 for cold-formed steel structural framing. This reference is updated to AISI S240. Additionally, the screw penetration test, which was previously referenced from ASTM C955 Section 8 for the “cold-formed steel studs and track, structural” entry, is recommended for deletion in both tables. Upon review, the AISI Committee on Framing Standards, which is responsible for developing the provisions of AISI S240, determined that the test procedure was not really applicable to structural members.

Additionally, in both Table 2506.2 and Table 2507.2, the screw penetration test, which was previously referenced from ASTM C645 Section 10 for the “cold-formed steel studs and track, nonstructural” entry, has been incorporated into the 2015 edition of AISI S220 and is, therefore, recommended for deletion in both tables.

Cost Impact: Will increase the cost of construction

This code change proposal adopts the latest industry standards for cold-formed steel. At this time, it is difficult to anticipate how cost of construction will be fully impacted, other than to note that some of the additional costs will be offset by new efficiencies in the design and installation of cold-formed steel.

Analysis: A review of the standard(s) proposed for inclusion in the code, AISI S240, with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 1, 2016.
Public Comment 1:

Proponent: Mike Fischer, Kellen, representing The Gypsum Association (mfischer@kellencompany.com) requests Approve as Modified by this Public Comment.

Modify as follows:

2015 International Building Code

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>STANDARD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Accessories for gypsum board</td>
<td>ASTM C1047</td>
</tr>
<tr>
<td>Adhesives for fastening gypsum board</td>
<td>ASTM C557</td>
</tr>
<tr>
<td>Cold-formed steel studs and track, structural</td>
<td>AISI S240  and ASTM C 955, Section 8</td>
</tr>
<tr>
<td>Cold-formed steel studs and track, nonstructural</td>
<td>AISI S220  and ASTM C 645, Section 10</td>
</tr>
<tr>
<td>Elastomeric joint sealants</td>
<td>ASTM C 920</td>
</tr>
<tr>
<td>Fiber-reinforced gypsum panels</td>
<td>ASTM C 1278</td>
</tr>
<tr>
<td>Glass mat gypsum backing panel</td>
<td>ASTM C 1178</td>
</tr>
<tr>
<td>Glass mat gypsum panel 5</td>
<td>ASTM C 1658</td>
</tr>
<tr>
<td>Glass mat gypsum substrate</td>
<td>ASTM C 1177</td>
</tr>
<tr>
<td>Joint reinforcing tape and compound</td>
<td>ASTM C 474; C 475</td>
</tr>
<tr>
<td>Nails for gypsum boards</td>
<td>ASTM C 514, F 547, F 1667</td>
</tr>
<tr>
<td>Steel screws</td>
<td>ASTM C 954; C 1002</td>
</tr>
<tr>
<td>Standard specification for gypsum board</td>
<td>ASTM C 1396</td>
</tr>
<tr>
<td>Testing gypsum and gypsum products</td>
<td>ASTM C 22; C 472; C 473</td>
</tr>
</tbody>
</table>

Commenter's Reason: The original proposal replaces AISI S200 with newly completed AISI S240-15. The Gypsum Association members support the inclusion of the revised standards, but want to retain the current ASTM referenced standards, ASTM C645 and ASTM C955. The proponent's reason statement indicates AISI S220 incorporates the requirements for fastener penetration of the current version of ASTM C645, but the Gypsum Association is concerned that future versions of both standards may not be aligned, and that the IBC should retain both.

The reason statement justifies the removal of the current reference to ASTM C955, Section 8, because “it was the consensus of the AISI Committee on Framing Standards that the screw penetration test was not necessary for loadbearing CFS framing members. The basis of the determination is that the test never produced a failed result for the thickness of members used in structural framing applications.”

The Gypsum Association is concerned that removing the screw penetration requirements in ASTM C955 could have an adverse effect on the performance of gypsum fasteners, resulting in screw spin-outs and additional penetrations in the boards, which can compromise the integrity of the boards.
Committee Action: Disapproved

Proposed Change as Submitted

Proponent: Jay Crandell, ARES Consulting, representing Foam Sheathing Committee of the American Chemistry Council (jcrandell@aresconsulting.biz)

2015 International Building Code

Revise as follows:

2510.6 Water-resistive barriers. Water-resistive barriers shall be installed as required in Section 1404.2 and, where applied over wood-based sheathing, shall include a water-resistive vapor permeable barrier with a performance at least equivalent to two layers of water-resistive barrier complying with ASTM E 2556, Type I. The individual layers shall be installed independently such that each layer provides a separate continuous plane and any flashing (installed in accordance with Section 1405.4) intended to drain to the water-resistive barrier is directed between the layers.

Exception: Where the water-resistive barrier that is applied over wood-based sheathing has a water resistance equal to or greater than that of a water-resistive barrier complying with ASTM E 2556, Type II and is separated from the stucco by an intervening, substantially non-water-absorbing layer or drainage space.

Add new text as follows:

2510.6.1 Application over wood-based sheathing Water-resistive barrier applications over wood-based sheathing shall comply with one of the following:

1. Two layers of Grade D paper complying with ASTM E2556, Type I, installed independently such that each layer provides a separate continuous plane in accordance with Section 1404.2 and any flashing intended to drain to the water-resistive barrier is directed between the layers.

2. One layer of Grade D paper complying with ASTM E2556, Type II, installed in accordance with Section 1404.2 and separated from the stucco by an intervening, substantially non-water-absorbing layer or drainage space with any flashing intended to drain to the water-resistive barrier directed between the layers or into the drainage space.

3. One layer of an approved water-resistive barrier material with a water resistance not less than one layer of Grade D paper complying with ASTM E 2556, Type II, and installed in accordance with the manufacturer's installation instructions. The approved water-resistive barrier material shall be separated from the stucco by an intervening, substantially non-water-absorbing layer or drainage space with any flashing intended to drain to the water-resistive barrier directed between the layers or into the drainage space.

Reason: This proposal improves the clarity and enforceability of this section of code by clarifying general requirements (Section 2510.6) and transparently distinguishing the three options for materials and methods associated with water-resistive barrier applications over wood-based sheathing (new Section 2510.6.1). In addition, the building science intent of the code is improved by removing an exclusionary and problematic specification of a “vapor permeable” water resistive barrier (WRB). The exclusive specification of “vapor permeable” conflicts with the ability to use a vapor permeable or non-vapor permeable WRB when it is properly coordinated with the vapor retarder provisions of Section 1405.3. For example, in warm/humid climates it is actually preferable to have a lower vapor permeance (non-vapor permeable) WRB on the exterior behind the stucco to mitigate excessive inward vapor drives and moisture movement. In cold climates, it is also possible to apply provisions of Section 1405.3.2 (Class III vapor retarder) or Section 1405.3.1 (Class I or II vapor retarder) with an appropriate amount of exterior continuous insulation to allow the use of a lower vapor permeance (non-vapor permeable) WRB. Thus, the code appropriately permits the use of vapor permeable and non-vapor permeable WRB materials when properly coordinated with use of vapor retarders in Section 1405.3. Finally, the specific recognition of Grade D paper in coordination with reference to the ASTM E2556 standard is restored since the ASTM E2556 standard addresses not only Grade D paper, but also other approved materials that are not necessarily equivalent to traditional Grade D paper, thus potentially obscuring requirements for other approved materials in regard to equivalent installed water penetration resistance and maximum permeance (both of which are not addressed in ASTM E2556).

Cost Impact: Will not increase the cost of construction

The proposal primarily clarifies existing requirements and provides additional options for WRB selection.

Public Hearing Results

Committee Action: Disapproved
Committee Reason: The committee believes that rather than being primarily editorial this proposal would actually change requirements. In addition the committee acknowledged the testimony stating that the proposal will increase the cost of construction. There is also a concern that the added wording, "...non-water absorbing layer or drainage space with any flashing intended to drain.......", would be confusing and it is not clear that why it is needed since there already is an exception that is based on providing equivalency.

Assembly Motion: As Submitted
Online Vote Results: Failed
Support: 18.62% (46) Oppose: 81.38% (201)
Assembly Action: None

Individual Consideration Agenda

Proponent: Jay Crandell, P.E., ARES Consulting, representing Foam Sheathing Committee of the American Chemistry Council (jcrandell@aresconsulting.biz) requests Approve as Submitted.

Commenter's Reason: This proposal was not understood correctly by the structural committee. It was intended to be both editorial and technical. From an editorial perspective it improves the format and clarity of the existing requirements in the code. From a technical perspective, it removes an exclusionary specification in the code. This was explained in the original reason statement for the proposal which also serves as the rational and justification for this PC. The language that the committee considered confusing is already in the code and this proposal does not bring any new confusion to its use. For these reasons, we are requesting approval as submitted.

S301-16
Committee Action: Approved as Modified

Proposed Change as Submitted

Proponent: Jay Crandell, representing Foam Sheathing Committee of the American Chemistry Council (jcrandell@aresconsulting.biz)

2015 International Building Code

Revise as follows:

2510.6 Water-resistive barriers. Water-resistive barriers shall be installed as required in Section 1404.2 and, where applied over wood-based sheathing, shall include a water-resistive vapor-permeable barrier with a performance at least equivalent to two layers of water-resistive barrier complying with ASTM E 2556, Type I. The individual layers shall be installed independently such that each layer provides a separate continuous plane and any flashing (installed in accordance with Section 1405.4) intended to drain to the water-resistive barrier is directed between the layers.

Exceptions:
- Where the water-resistive barrier that is applied over wood-based sheathing has a water resistance equal to or greater than that of a water-resistive barrier complying with ASTM E 2556, Type II and is separated from the stucco by an intervening, substantially nonwater-absorbing layer or drainage space.
- Where the water-resistive barrier is applied over vapor permeable or wood-based sheathing in Climate Zones 1A, 2A, 3A, 4A, 5A, and 4C in accordance with Chapter 3 of the International Energy Conservation Code, the water-resistive barrier shall have a water vapor permeance of not less than 10 perms in accordance with ASTM E96 (Method A) to minimize inward moisture movement. Alternatively, a ventilated air space shall be provided between the stucco and water-resistive barrier.

Reason: In many climates, having a vapor permeable WRB that is too vapor permeable (i.e., > 10 perms) can result in significant solar-driven inward moisture movement into and through the exterior sheathing and farther into the wall assembly (e.g., to the interior vapor retarder or interior finishes), causing significantly increased risk of moisture damage and mold. This concern is particularly relevant to Section 2510.6 which deals with conventional stucco -- a moisture storage (“reservoir”) cladding. Consequently, a new exception is provided to address this problem and is based on consistent findings and recommendations from several studies including Derome (2010), Wilkinson, et al. (2007), BSC (2005), and Lepage and Lstiburek (2013). Key findings and recommendations from these studies also are summarized in ABTG (2015). It is also important to note that this proposal does NOT eliminate the use of WRB materials of greater than 10 perms in the stated conditions because an alternative allows for use of a ventilated air space to avoid the 10 perm limitation.


Cost Impact: Will not increase the cost of construction
The proposal provides limitations that may affect some product choices (or cladding detailing) under specified conditions of use, but options remain available for all WRB types and many are unaffected.

Public Hearing Results

Modification: Approved as Modified
2510.6 Water-resistive barriers. *Water-resistive barriers* shall be installed as required in Section 1404.2 and, where applied over wood-based sheathing, shall include a water-resistive vapor-permeable barrier with a performance at least equivalent to two layers of *water-resistive barrier* complying with ASTM E 2556, Type I. The individual layers shall be installed independently such that each layer provides a separate continuous plane and any flashing (installed in accordance with Section 1405.4) intended to drain to the *water-resistive barrier* is directed between the layers.

- **Exceptions:**
  - 1. Where the *water-resistive barrier* that is applied over wood-based sheathing has a water resistance equal to or greater than that of a *water-resistive barrier* complying with ASTM E 2556, Type II and is separated from the stucco by an intervening, substantially nonwater-absorbing layer or drainage space.
  - 2. Where the *water-resistive barrier* is applied over vapor-permeable or wood-based sheathing in Climate Zones 1A, 2A, or 3A, 4A, 5A, and 4C in accordance with Chapter 3 of the International Energy Conservation Code, the *water-resistive barrier* shall have a water vapor permeance of not more than 10 perms in accordance with ASTM E96 (Method A) to minimize inward moisture movement. Alternatively, a ventilated air space shall be provided between the stucco and *water-resistive barrier*.

**Committee Reason:** This proposal adds an option to address inward moisture drive issues in various climate zones. It also provides the universal solution of providing a vented air space. The modification eliminates an arbitrary limit on vapor permeance and limits the climate zones where it applies.

**Assembly Action:** None

*Individual Consideration Agenda*

**Proponent:** Theresa Weston, representing DuPont Building Innovations (theresa.a.weston@dupont.com) requests Disapprove.

**Commenter’s Reason:** I am concerned about the structure of the code language in this proposal. An additional requirement is included as an exception and, therefore, may create confusion. Is exception 2 required in climate zones 1A, 2A and 3A? Or can you use your choice of the charging language provisions, exception 1 or exception 2? As written I believe the provision will add confusion to the code and be a barrier to enforcement.
Proposed Change as Submitted

Proponent: Laverne Dalgleish, Building Professionals, representing Building Professionals
(idalgleish@buildingprofessionals.com)

2015 International Building Code

Revise as follows:

2510.6 Water-resistive barriers. Water-resistive barriers shall be installed as required in Section 1404.2 and, where applied over wood-based sheathing, shall include a water-resistive vapor-permeable barrier with a performance at least equivalent to two layers of water-resistive barrier complying with ASTM E 2556, Type I. The individual layers shall be installed independently such that each layer provides a separate continuous plane and any flashing (installed in accordance with Section 1405.4) intended to drain to the water-resistive barrier is directed between the layers.

Exception: Where the water-resistive barrier that is applied over wood-based sheathing has a water resistance equal to or greater than that of a water-resistive barrier complying with ASTM E 2556, Type II and is separated from the stucco by an intervening, substantially nonwater-absorbing layer or drainage space or material complying with ASTM E2925.

Reference standards type: This reference standard is new to the ICC Code Books

Add new standard(s) as follows:

ASTM E2925 - 14 Standard Specification for Manufactured Polymeric Drainage and Ventilation Materials used to Provide a Rainscreen Function

Reason: Material complying with ASTM E2925 provides a minimum drainage space and provides a test method for the material to confirm the drainage space.

Cost Impact: Will not increase the cost of construction

As this addition is only one of the three options available, the cost of construction will not increase.

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM E2925, with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 1, 2016.

Public Hearing Results

Committee Action: Disapproved

Committee Reason: There are concerns that the proposed referenced standard might allow "gaming" and that it is missing performance criteria so the proposal would have to define that,

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Laverne Dalgleish, Building Professionals, representing Building Professionals (idalgleish@buildingprofessionals.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

Reference standards type: This reference standard is new to the ICC Code Books

Add new standard(s) as follows:

Commenter's Reason: The concern raised at the Group B Hearings was that the standard referenced (ASTM E2925) listed specific requirements for both the substrate and the water resistive barrier in the construction of the test specimen. This standard is currently being modified to make the requirements generic both for the substrate and for the water resistive barrier. The ASTM E2925 standard originally stated;

This will now state;
"A.1.2.1 The test specimen shall be 1200 mm by 2400 mm constructed from 50 mm by 100 mm (nominal) framing for the perimeter framing with two vertically studs every 400 mm on center across the 1200 mm leg, have a typical substrate used in building construction installed on one side of the wood framing which is then covered with a water resistive barrier (WRB) that it is seamless and continuous."

With the modified requirements in the standard, any substrate and any water resistive barrier is acceptable to be used in constructing the mockup.

**Analysis:** The proposed modification to this code change proposal includes update of the year edition of standard ASTM E2925 from -14 to -16. CP28, Section 3.6.3.1 and newly referenced standard "shall be completed and readily available prior to the Public Comment Hearing based on the cycle of code development which includes the code change proposal."

Therefore, the proponent is required to provide information verifying that the standard ASTM E2925-16 is completed and readily available at the time of the public comment hearings.

S303-16
Committee Action: Disapproved

Proponent: Laverne Dalgleish, Building Professionals, representing Building Professionals (ldalgleish@buildingprofessionals.com)

2015 International Building Code

Revise as follows:

2510.6 Water-resistive barriers. Water-resistive barriers shall be installed as required in Section 1404.2 and, where applied over wood-based sheathing, shall include a water-resistive vapor-permeable barrier with a performance at least equivalent to two layers of water-resistive barrier complying with ASTM E 2556, Type I. The individual layers shall be installed independently such that each layer provides a separate continuous plane and any flashing (installed in accordance with Section 1405.4) intended to drain to the water-resistive barrier is directed between the layers.

Exception: Where the water-resistive barrier that is applied over wood-based sheathing has a water resistance equal to or greater than that of a water-resistive barrier complying with ASTM E 2556, Type II and is separated from the stucco by

a. ASTM E 2556, Type II and is separated from the stucco by an intervening, substantially non-water-absorbing layer or drainage space, or
b. ASTM E2556, Type I and is separated from the stucco by an intervening, drainage space provided by materials complying with ASTM E2925.

Reference standards type: This reference standard is new to the ICC Code Books

Add new standard(s) as follows:

ASTM E2925 - 14 Standard for Manufactured Polymeric Drainage and Ventilation Materials used to Provide a Rainscreen Function

Reason: Materials complying with ASTM E2925 provide a minimum drainage and ventilation space. By having a material specification, test methods are identified to be able to identify materials than meet the specification

Cost Impact: Will not increase the cost of construction

As there is three options, the cost of construction will not increase. Using materials that comply with ASTM E2925 allows you to use a Type I water resistive barrier which is not as costly as a Type II.

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM E2925, with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 1, 2016.

Public Hearing Results

Committee Reason: Similar to disapproval of S303-16. The proposed reference standard has no acceptance criteria. There was no justification provided for changing the type of water-reisistive barrier. The proposed Type I is 10 minute paper, so this is possibly reducing the water-resistant nature of the assembly.

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Laverne Dalgleish, Building Professionals, representing Building Professionals (ldalgleish@buildingprofessionals.com) requests Approve as Modified by this Public Comment.

Modify as Follows:

Reference standards type: This reference standard is new to the ICC Code Books

Add new standard(s) as follows:

Commenter's Reason: The concern raised at the Group B Hearings was that the standard referenced (ASTM E2925) listed
specific requirements for both the substrate and the water resistive barrier in the construction of the test specimen. This standard is currently being modified to make the requirements generic both for the substrate and for the water resistive barrier. The ASTM E2925 standard originally stated:

"A1.2.1 Construct one 1200 mm by 2400 mm test wall assembly comprised of 50 mm by 100 mm (nominal) perimeter framing and 50 mm by 100 mm (nominal) framing vertically at 400 mm on center. Install a wood panel of 11 mm oriented strand board (OSB) to the framing and fasten with 10d fasteners at 200 mm on center. Install a water resistive barrier (WRB) complying with Specification E2556/E2556M on the OSB in a seamless, continuous manner."

This will now state;

"A.1.2.1 The test specimen shall be 1200 mm by 2400 mm constructed from 50 mm by 100 mm (nominal) framing for the perimeter framing with two vertically studs every 400 mm on center across the 1200 mm leg, have a typical substrate used in building construction installed on one side of the wood framing which is then covered with a water resistive barrier (WRB) that it is seamless and continuous."

With the modified requirements in the standard, any substrate and any water resistive barrier is acceptable to be used in constructing the mockup. The proposal has been modified to simply add ASTM E2925 material as an alternative to the two other exceptions.

**Analysis:** The proposed modification to this code change proposal includes update of the year edition of standard ASTM E2925 from -14 to -16. CP28, Section 3.6.3.1 and newly referenced standard "shall be completed and readily available prior to the Public Comment Hearing based on the cycle of code development which includes the code change proposal."

Therefore, the proponent is required to provide information verifying that the standard ASTM E2925-16 is completed and readily available at the time of the public comment hearings.

**Public Comment 2:**

Proponent : Laverne Dalgleish, representing Building Professionals (ldalgleish@airbarrier.org) requests Approve as Modified by this Public Comment.

Modify as Follows:

**2015 International Building Code**

**2510.6 Water-resistive barriers.** Water-resistive barriers shall be installed as required in Section 1404.2 and, where applied over wood-based sheathing, shall include a water-resistive vapor-permeable barrier with a performance at least equivalent to two layers of water-resistive barrier complying with ASTM E 2556, Type I. The individual layers shall be installed independently such that each layer provides a separate continuous plane and any flashing (installed in accordance with Section 1405.4) intended to drain to the water-resistive barrier is directed between the layers.

**Exception:** Where the water-resistive barrier that is applied over wood-based sheathing has a water resistance equal to or greater than that of a water-resistive barrier complying with: ASTM E 2556, Type II and is separated from the stucco by an intervening, substantially nonwater-absorbing layer or drainage space, or a material complying with ASTM E2925.

- a. ASTM E 2556, Type II and is separated from the stucco by an intervening, substantially nonwater-absorbing layer or drainage space,
- b. ASTM E2925, Type I and is separated from the stucco by an intervening, drainage space provided by materials complying with ASTM E2925.

**Reference standards type:** This reference standard is new to the ICC Code Books

**Add new standard(s) as follows:**

**Commenter's Reason:** The concern raised at the Group B Hearings was that the standard referenced (ASTM E2925) listed specific requirements for both the substrate and the water resistive barrier in the construction of the test specimen. This standard is currently being modified to make the requirements generic both for the substrate and for the water resistive barrier. The ASTM E2925 standard originally stated:

"A1.2.1 Construct one 1200 mm by 2400 mm test wall assembly comprised of 50 mm by 100 mm (nominal) perimeter framing and 50 mm by 100 mm (nominal) framing vertically at 400 mm on center. Install a wood panel of 11 mm oriented strand board (OSB) to the framing and fasten with 10d fasteners at 200 mm on center. Install a water resistive barrier (WRB) complying with Specification E2556/E2556M on the OSB in a seamless, continuous manner."

This will now state;

"A.1.2.1 The test specimen shall be 1200 mm by 2400 mm constructed from 50 mm by 100 mm (nominal) framing for the
perimeter framing with two vertically studs every 400 mm on center across the 1200 mm leg, have a typical substrate used in building construction installed on one side of the wood framing which is then covered with a water resistive barrier (WRB) that it is seamless and continuous."

With the modified requirements in the standard, any substrate and any water resistive barrier is acceptable to be used in constructing the mockup.

The proposal has also been modified to simply add ASTM E2925 material as a third exception.

**Analysis:** The proposed modification to this code change proposal includes update of the year edition of standard ASTM E2925 from -14 to -16. CP28, Section 3.6.3.1 and newly referenced standard “shall be completed and readily available prior to the Public Comment Hearing based on the cycle of code development which includes the code change proposal.” Therefore, the proponent is required to provide information verifying that the standard ASTM E2925-16 is completed and readily available at the time of the public comment hearings.
Proposed Change as Submitted

Proponent: Gregory Wilson (gregory.wilson2@fema.dhs.gov); Rebecca Quinn, representing Federal Emergency Management Agency (rcquinn@earthlink.net)

2015 International Building Code

Add new text as follows:

G101.5 Designation of Floodplain Administrator. The [INSERT JURISDICTION'S SELECTED POSITION TITLE] is designated as the floodplain administrator. The floodplain administrator is permitted to delegate performance of certain duties to other employees of the jurisdiction.

Revise as follows:

G103.1 Permit applications. All applications for permits must comply with the following:
1. The building official floodplain administrator shall review all permit applications to determine whether proposed development is located in flood hazard areas established in Section G102.2.
2. Where a proposed development site is in a flood hazard area, all development to which this appendix is applicable as specified in Section G102.1 shall be designed and constructed with methods, practices and materials that minimize flood damage and that are in accordance with this code and ASCE 24.

G103.2 Other permits. It shall be the responsibility of the building official floodplain administrator to ensure that approval of a proposed development shall not be given until proof that necessary permits have been granted by federal or state agencies having jurisdiction over such development.

G103.3 Determination of design flood elevations. If design flood elevations are not specified, the building official floodplain administrator is authorized to require the applicant to:
1. Obtain, review and reasonably utilize data available from a federal, state or other source; or
2. Determine the design flood elevation in accordance with accepted hydrologic and hydraulic engineering techniques. Such analyses shall be performed and sealed by a registered design professional. Studies, analyses and computations shall be submitted in sufficient detail to allow review and approval by the building official floodplain administrator. The accuracy of data submitted for such determination shall be the responsibility of the applicant.

G103.4 Activities in riverine flood hazard areas. In riverine flood hazard areas where design flood elevations are specified but floodways have not been designated, the building official floodplain administrator shall not permit any new construction, substantial improvement or other development, including fill, unless the applicant submits an engineering analysis prepared by a registered design professional, demonstrating that the cumulative effect of the proposed development, when combined with all other existing and anticipated flood hazard area encroachment, will not increase the design flood elevation more than 1 foot (305 mm) at any point within the community.

G103.5 Floodway encroachment. Prior to issuing a permit for any floodway encroachment, including fill, new construction, substantial improvements and other development or land-disturbing activity, the building official floodplain administrator shall require submission of a certification, prepared by a registered design professional, along with supporting technical data, demonstrating that such development will not cause any increase of the base flood level.

G103.6 Watercourse alteration. Prior to issuing a permit for any alteration or relocation of any watercourse, the building official floodplain administrator shall require the applicant to provide notification of the proposal to the appropriate authorities of all affected adjacent government jurisdictions, as well as appropriate state agencies. A copy of the notification shall be maintained in the permit records and submitted to FEMA.

G103.6.1 Engineering analysis. The building official floodplain administrator shall require submission of an engineering analysis, prepared by a registered design professional, demonstrating that the flood-carrying capacity of the altered or relocated portion of the watercourse will not be decreased. Such watercourses shall be maintained in a manner that preserves the channel's flood-carrying capacity.

G103.7 Alterations in coastal areas. Prior to issuing a permit for any alteration of sand dunes and mangrove stands in coastal high-hazard areas and coastal A zones, the building official floodplain administrator shall require submission of an engineering analysis, prepared by a registered design professional, demonstrating that the proposed alteration will not increase the potential for flood damage.

G103.8 Records. The building official floodplain administrator shall maintain a permanent record of all permits issued in flood hazard areas, including copies of inspection reports and certifications required in Section 1612.

G103.9 Inspections. Development for which a permit under this appendix is required shall be subject to inspection. The
building official floodplain administrator or the floodplain administrator's designee shall make, or cause to be made, inspections of all development in flood hazard areas authorized by issuance of a permit under this appendix.

G104.1 Required. Any person, owner or owner's authorized agent who intends to conduct any development in a flood hazard area shall first make application to the building official floodplain administrator and shall obtain the required permit.

G104.2 Application for permit. The applicant shall file an application in writing on a form furnished by the building official floodplain administrator. Such application shall:

1. Identify and describe the development to be covered by the permit.
2. Describe the land on which the proposed development is to be conducted by legal description, street address or similar description that will readily identify and definitely locate the site.
3. Include a site plan showing the delineation of flood hazard areas, floodway boundaries, flood zones, design flood elevations, ground elevations, proposed fill and excavation and drainage patterns and facilities.
4. Include in subdivision proposals and other proposed developments with more than 50 lots or larger than 5 acres (20 234 m²), base flood elevation data in accordance with Section 1612.3.1 if such data are not identified for the flood hazard areas established in Section G102.2.
5. Indicate the use and occupancy for which the proposed development is intended.
6. Be accompanied by construction documents, grading and filling plans and other information deemed appropriate by the building official floodplain administrator.
7. State the valuation of the proposed work.
8. Be signed by the applicant or the applicant's authorized agent.

G104.3 Validity of permit. The issuance of a permit under this appendix shall not be construed to be a permit for, or approval of, any violation of this appendix or any other ordinance of the jurisdiction. The issuance of a permit based on submitted documents and information shall not prevent the building official floodplain administrator from requiring the correction of errors. The building official floodplain administrator is authorized to prevent occupancy or use of a structure or site that is in violation of this appendix or other ordinances of this jurisdiction.

G104.4 Expiration. A permit shall become invalid if the proposed development is not commenced within 180 days after its issuance, or if the work authorized is suspended or abandoned for a period of 180 days after the work commences. Extensions shall be requested in writing and justifiable cause demonstrated. The building official floodplain administrator is authorized to grant, in writing, one or more extensions of time, for periods not more than 180 days each.

G104.5 Suspension or revocation. The building official floodplain administrator is authorized to suspend or revoke a permit issued under this appendix wherever the permit is issued in error or on the basis of incorrect, inaccurate or incomplete information, or in violation of any ordinance or code of this jurisdiction.

G105.2 Records. The building official floodplain administrator shall maintain a permanent record of all variance actions, including justification for their issuance.

G105.7 Conditions for issuance. Variances shall only be issued by the board of appeals where all of the following criteria are met:

1. A technical showing of good and sufficient cause that the unique characteristics of the size, configuration or topography of the site renders the elevation standards inappropriate.
2. A determination that failure to grant the variance would result in exceptional hardship by rendering the lot undevelopable.
3. A determination that the granting of a variance will not result in increased flood heights, additional threats to public safety, extraordinary public expense, nor create nuisances, cause fraud on or victimization of the public or conflict with existing local laws or ordinances.
4. A determination that the variance is the minimum necessary, considering the flood hazard, to afford relief.
5. Notification to the applicant in writing over the signature of the building official floodplain administrator that the issuance of a variance to construct a structure below the base flood level will result in increased premium rates for flood insurance up to amounts as high as $25 for $100 of insurance coverage, and that such construction below the base flood level increases risks to life and property.

Reason: When local jurisdictions join the National Flood Insurance Program they are required to designate the local official responsible for enforcing floodplain management regulations. For a variety of reasons, many jurisdictions identify an official other than the building official, in part because many responsibilities are not directly related to enforcement of requirements for buildings. Appendix G is scoped to apply to “development,” which is defined in Appendix G, and it governs activities other than buildings and structures. When a local jurisdiction uses IBC Appendix G to regulate development other than buildings it should be able to designate the appropriate official, which may or may not be the building official.

Cost Impact: Will not increase the cost of construction
There is no cost impact because this proposal is related to designation of personnel by individual jurisdictions.

Committee Action: Disapproved
Assembly Motion: As Submitted
Online Vote Results: Failed
Support: 34.91% (74) Oppose: 65.09% (138)
Assembly Action: None

Public Hearing Results

Committee Reason: In the building code the building official is authorized and directed to enforce the provisions of the code, if the proposed wording were adopted into the building code, the floodplain administrator is not in that position.

Assembly Motion: As Submitted
Online Vote Results: Failed
Support: 34.91% (74) Oppose: 65.09% (138)
Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

Proponent: Gregory Wilson (gregory.wilson2@fema.dhs.gov); Rebecca Quinn, representing Federal Emergency Management Agency (rcquinn@earthlink.net) requests Approve as Modified by this Public Comment.

Modify as Follows:

2015 International Building Code

G101.5 Designation of Floodplain Administrator. The [INSERT JURISDICTION'S SELECTED POSITION TITLE] is designated as the floodplain administrator and is authorized and directed to enforce the provisions of this appendix. The floodplain administrator is permitted to delegate performance of certain duties to other employees of the jurisdiction.

Commenter's Reason: The motion to disapproved was based on an unfounded concern that designating an official other than the building official to administer Appendix G would somehow mean that other official would be responsible for enforcing the building code. To eliminate that as a possible interpretation, the proposal is modified using phrasing similar to Section 101.4 to clearly state the designated administrator is authorized to enforce this appendix.

Appendix G is scoped to apply to “development,” which is defined in Appendix G, and it governs activities other than buildings and structures. When a local jurisdiction uses IBC Appendix G to regulate development other than buildings it should be able to designate the appropriate official, which may or may not be the building official. When local jurisdictions join the National Flood Insurance Program they are required to designate the local official responsible for enforcing floodplain management regulations. For a variety of reasons, many jurisdictions identify an official other than the building official, in part because many responsibilities are not directly related to enforcement of requirements for buildings.

S306-16
Committee Action: Disapproved

Assembly Motion: As Submitted

Online Vote Results: Failed
Support: 23.15% (47) Oppose: 76.85% (156)

Assembly Action: None

---

**S308-16 : G103.10 (New).**

### Proposed Change as Submitted

**Proponent**: Gregory Wilson (gregory.wilson2@fema.dhs.gov); Rebecca Quinn, representing Federal Emergency Management Agency (rcquinn@earthlink.net)

**2015 International Building Code**

Add new text as follows:

**G103.10 Submission of new technical data.** Not later than six months after the date new technical data or studies of flood hazard areas and base flood elevations become available, including proposed changes to flood hazard area boundaries or base flood elevations established by the Federal Emergency Management Agency, the building official shall submit or require submission of the technical data or studies to the Federal Emergency Management Agency. The building official shall permit use of changed flood hazard area boundaries or base flood elevations for proposed buildings or developments provided applicants have applied to the Federal Emergency Management Agency and received approvals for conditional revisions of Flood Insurance Rate Maps.

**Reason:** Virtually every community with identified areas subject to flooding adopts the Federal Emergency Management Agency's Flood Insurance Study and Flood Insurance Rate Maps (FIRMs) as the official maps. If a community develops its own flood study or if an applicant provides data or studies that show a change to a FIRM is appropriate, the data must be submitted to FEMA so the official maps are maintained with the best available information. Local officials do not have the authority to change FEMA's maps and data. If a flood zone or Base Flood Elevation is changed by a study and that change is not shown on the FIRM, decisions regarding future permit requirements and NFIP flood insurance policies would not be based on the best available information. The requirement to submit new studies within 6 months is in federal regulation (44 CFR Section 65.3). Another proposal for Appendix G proposes changing "building official" to "floodplain administrator;" if that proposal is approved, the same change should be made in this section.

**Cost Impact:** Will not increase the cost of construction
No cost impact; communities that participate in the NFIP are already required to submit, or require applicants to submit, new data and studies to FEMA.

---

**Public Hearing Results**

**Committee Action:** Disapproved

**Committee Reason:** The committee questioned whether the six month time limit is needed and there is also a concern that this differs from state mandated retention policies. Also some found the second sentence confusing and suggest replacing ".the building official shall permit the use of...." with wording indicating ".can't use the changed boundaries and elevations unless the applicant has applied to FEMA........".

**Assembly Motion:** As Submitted

**Support:** 23.15% (47) **Oppose:** 76.85% (156)

**Assembly Action:** None

---

**Individual Consideration Agenda**

Public Comment 1:

**Proponent**: Gregory Wilson, FEMA (gregory.wilson2@fema.dhs.gov); Rebecca Quinn, representing Federal Emergency Management Agency (rcquinn@earthlink.net) requests Approve as Modified by this Public Comment.

**Modify as Follows:**

**2015 International Building Code**

G103.10 Submission Use of new changed technical data. Not later than six months after the date new technical data or studies of flood hazard areas and base flood elevations become available, including proposed changes to flood hazard area boundaries or base flood elevations established by the Federal Emergency Management Agency, the building official shall submit or require submission of the technical data or studies to the Federal Emergency Management Agency. The building
official and the applicant shall permit not use of changed flood hazard area boundaries or base flood elevations for proposed buildings or developments provided applicants have, unless the building official or applicant has applied to the Federal Emergency Management Agency and received approvals for conditional revisions of Flood Insurance Rate Maps.

Commenter's Reason: The motion to disapprove expressed concern about the 6 month period. While FEMA does expect communities that undertake studies to submit them in a timely manner, the discussion prompted a modification to make the proposal pertinent to new technical data submitted by applicants, but only if the applicant proposes changes flood hazard area boundaries and base flood elevations. FEMA has a formal process to amend flood data. Local officials do not have the authority to change FEMA's maps and data. If a flood zone or Base Flood Elevation is changed by a study and that change is not shown on the FIRM, decisions regarding future permit requirements and NFIP flood insurance policies would not be based on the best available information. Another proposal for Appendix G proposes changing "building official" to "floodplain administrator;" if that proposal is approved, the same change should be made in this section.
Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com)

2015 International Building Code

Revise as follows:

APPENDIX L EARTHQUAKE STRONG-MOTION RECORDING INSTRUMENTATION

SECTION L101 GENERAL

L101.1 General. Every structure located where the 1-second spectral response acceleration within 15 miles of a potential earthquake source of M 6.0 or greater, $S_1$, in accordance with Section 1613.3 is and within 25 miles of a potential earthquake source of M 7.0 or greater than 0.40 that either: (1) exceeds six stories in height with an aggregate floor area of 60,000 square feet (5574 m$^2$) or more; or (2) exceeds 10 stories in height regardless of floor area, shall be equipped with not less than three approved strong-motion recording accelerographs instruments. The accelerographs instruments shall be interconnected for common start and common timing.

Reason: Instruments should be located based upon the locations of the potentially largest earthquakes, not according to the output of a mathematically flawed hazard model, with all the systemic problems of psha.

Bibliography: See also BIBLIOGRAPHY in Proposal: Figure 1613.3.1 RISK-TARGETED MCER
Cost Breakdown of Nonstructural Building Elements
Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake.
http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032

Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Permalink: http://dx.doi.org/10.1061/(ASCE)CF.1943-5509.0000630
Read More: http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630
Homeowner's Guide to Earthquake Safety

Retrofitting Questions and Answers
Earthquake Safety, Inc., 2015 (web based)
http://www.earthquakesafety.com/earthquake-retrofitting-faq.html

Cost and Seismic Design
https://www.google.com/#q=cost+and+seismic+design+christopher+arnold+pdf

Earthquake Architecture website
http://www.iitk.ac.in/nicee/wcee/article/14_05-06-0185.PDF
Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee

Cost Impact: Will increase the cost of construction
Will probably both increase and decrease the cost of construction, since the mapped spectral response accelerations are often either too low, or too high
Committee Action: Disapproved

Committee Reason: This proposal would introduce terminology not in common use and it would also create conflicts between the IBC and the terminology of ASCE 7.

Assembly Action: None

Individual Consideration Agenda

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com) requests Approve as Submitted.

Commenter's Reason: "It is better to be right than to be consistent."
    - Winston Churchill

"If your definition is wrong, you'll look for the wrong thing."
    - Carol Cleland

“Anyone who conducts an argument by appealing to authority is not using his intelligence; he is just using his memory.“
    - Leonardo da Vinci

“Just because it comes from a consensus standard doesn't mean it isn't without problems."
    - Jay Crandell

The Committee Reason for Disapproval is based on the Logical Fallacies: APPEAL to AUTHORITY and STRAW MAN.
The Straw Man fallacy is committed when a person simply ignores a person’s actual position and substitutes a distorted, exaggerated or misrepresented version of that position. This sort of “reasoning” has the following pattern:

- Person A has position X.
- Person B presents position Y (which is a distorted version of X).
- Person B attacks position Y.
- Therefore X is false/incorrect/flawed.

within 15 miles of a potential earthquake source of M 6.0 or greater, and within 25 miles of a potential earthquake source of M 7.0 or greater are, in fact, terminology very much in common use. Likewise, the term "strong motion instrument" returned 2,480,000 “hits” – “accelerograph only returned 20,700! And this corresponds to the wording in the California “Strong Motion Instrumentation Program.”
Proposed Change as Submitted

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com)

2015 International Building Code
Revise as follows:

APPENDIX M TSUNAMI-GENERATED FLOOD TSUNAMI HAZARD and RISK

The provisions contained in this appendix are not mandatory unless specifically referenced in the adopting ordinance.

User note: Code change proposals to this chapter will be considered by the IBC – Structural Code Development Committee during the 2016 (Group B) Code Development Cycle. See explanation on page iv.

M101.1 General. The purpose of this appendix is to provide tsunami regulatory criteria safety guidelines for those communities that have a tsunami hazard and have elected to develop and adopt a map of their tsunami hazard inundation zone.

M101.2 Definitions. The following words and terms shall, for the purposes of this appendix, have the meanings shown herein. Refer to Chapter 2 of this code for general definitions.

M101.3 Establishment of tsunami hazard zone. When a community has adopted a Tsunami Hazard Zone Map or Maps, that map(s) shall be used to establish boundaries of a community's tsunami hazard zone.

M101.4 Construction within the tsunami hazard zone. Construction of structures designated Risk Occupancy and Use Categories III and IV, as specified under Section 1604.5, shall be prohibited within a tsunami hazard zone.

Exceptions:

0.1. A vertical evacuation tsunami refuge shall be permitted to be located in a tsunami hazard zone provided it is constructed in accordance with FEMA P646.

0.2. Community critical facilities shall be permitted to be located within the tsunami hazard zone when such a location is necessary to fulfill their function, providing suitable structural and emergency evacuation measures have been incorporated.

Exception:

Community critical facilities, such as fire, police, wastewater, etc., shall be permitted to be located within the tsunami hazard zone - whenever such a location is a necessary requirement to fulfill their function, and providing suitable structural, nonstructural and emergency evacuation measures have been incorporated.

TSUNAMI HAZARD ZONE. The area vulnerable to being flooded or inundated by a design event tsunami as identified on a community's Tsunami Hazard Zone Map.

TSUNAMI HAZARD ZONE MAP. A map adopted by the community that designates the extent of inundation by a design event tsunami. This map shall be based on the tsunami inundation map that is developed and provided to a community by either the applicable state agency or the National Atmospheric and Oceanic Administration (NOAA) under the National Tsunami Hazard Mitigation Program, but shall be permitted to utilize a different probability or hazard level.

Reference standards type: This is an update to reference standard(s) already in the ICC Code Books
Add new standard(s) as follows:

FEMA P646---12 Guidelines for Design of Structures for Vertical Evacuation from Tsunamis, in its entirety.

Reason: FEMA P646 is defective, and therefore shouldn't be promulgated in a regulatory framework.
REFERENCE STANDARD REASON: The incorporation of so-called probabilistic tsunami hazard analysis in inappropriate, as explained elsewhere regarding psha.

Bibliography: See also BIBLIOGRAPHY in Proposal: Figure 1613.3.1 RISK-TARGETED MCER
Cost Breakdown of Nonstructural Building Elements

Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake.
http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032

Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Permalink: http://dx.doi.org/10.1061/(ASCE)CF.1943-5509.0000630
Read More: http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630

Homeowner's Guide to Earthquake Safety

Retrofitting Questions and Answers
Earthquake Safety, Inc., 2015 (web based)
http://www.earthquakesafety.com/earthquake-retrofitting-faq.html

Cost and Seismic Design
https://www.google.com/#q=cost+and+seismic+design+christopher+arnold+pdf
Earthquake Architecture website
http://www.itk.ac.in/nicee/kwce/article/14_05-06-0185.PDF

Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee

**Cost Impact:** Will not increase the cost of construction
Guidelines do not impact the cost of construction

---

**Public Hearing Results**

**Committee Action:** Disapproved

**Committee Reason:** This appendix chapter on tsunami hazard is not mandatory. The reference to FEMA guidelines is appropriate and it should remain in appendix.

**Assembly Action:** None

**Individual Consideration Agenda**

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com) requests Approve as Submitted.

Commenter's Reason: "It is better to be right than to be consistent."
- Winston Churchill

"It is better to be alive than underwater."

"If your definition is wrong, you'll look for the wrong thing."
- Carol Cleland
“Anyone who conducts an argument by appealing to authority is not using his intelligence; he is just using his memory.”

- Leonardo da Vinci

“Just because it comes from a [FEMA Guideline] consensus standard doesn't mean it isn't without problems.”

- Jay Crandell

The Committee Reason for Disapproval is based on the Logical Fallacies: **APPEAL to AUTHORITY; BEGGING the QUESTION (CIRCULAR REASONING); and STRAW MAN.**

Guidelines are "things to be considered" and not **prescriptive paths** to an end result of seeming endorsement. They are more undefined than standards (without vetting by experience) and therefore unenforceable. Whereas “a standard is a published technical document that represents an industry consensus on how a material, product or assembly is to be designed, manufactured, tested or installed so that a specific level of performance is obtained;” and also a standard “is developed in response to an identified need.” This FEMA Guideline, on the other hand, is, in effect, **creating its own need** – without **acknowledging critically serious flaws regarding tsunami inundation wave height determinations** that could potentially be **catastrophically underestimated**, leading to unnecessary deaths of the said structure's occupants – as have, unfortunately, already occurred in Japan 2011.

These guidelines do, in fact, become **mandatory**, if "specifically referenced in the adopting ordinance." Thus they could become so adopted by jurisdictions who may not fully understand their uncertainties, assumptions, and also their serious limitations.

Proposed Change as Submitted

Proponent: Mike Mahoney, Federal Emergency Management Agency, representing SELF (mike.mahoney@fema.dhs.gov)

2015 International Building Code

Revise as follows:

APPENDIX M TSUNAMI-GENERATED FLOOD HAZARD

SECTION M101 REFUGE STRUCTURES FOR VERTICAL EVACUATION FROM TSUNAMI-GENERATED FLOOD HAZARD

M101.1 General. The purpose of this appendix is to provide tsunami regulatory vertical evacuation planning criteria for those coastal communities that have a tsunami hazard and have elected to develop and adopt as shown in a map of their tsunami hazard inundation zone Tsunami Design Zone Map.

M101.2 Definitions. The following words and terms shall, for the purposes of this appendix, have the meanings shown herein. Refer to Chapter 2 of this code for general definitions.

Delete without substitution:

TSUNAMI HAZARD ZONE.
The area vulnerable to being flooded or inundated by a design event tsunami as identified on a community's Tsunami Hazard Zone Map.

Revise as follows:

TSUNAMI HAZARD DESIGN ZONE MAP.
A map adopted by the community that designates the extent of inundation by a design event tsunami. This map shall be based on the tsunami inundation map that is developed and provided to a community Maximum Considered Tsunami, as defined by either the applicable state agency or the National Atmospheric and Oceanic Administration (NOAA) under the National Tsunami Hazard Mitigation Program, but shall be permitted to utilize a different probability or hazard level Chapter 6 of ASCE 7.

M101.3 Establishment of tsunami hazard design zone. Where applicable, if a community has adopted a the Tsunami Hazard Design Zone Map, that map shall be used to establish a community's tsunami hazard zone meet or exceed the inundation limit given by the ASCE 7 Tsunami Design Geodatabase.

M101.4 Construction Planning of tsunami vertical evacuation refuge structures within the tsunami hazard design zone. Construction of structures designated Risk Categories III and IV as specified under Section 1604.5 shall be prohibited Tsunami Vertical Evacuation Refuge Structures located within a tsunami hazard design zone shall be planned, sited, and developed in general accordance with the planning criteria of the FEMA P646 guidelines.

- Exceptions:
  1. A vertical evacuation tsunami refuge shall be permitted to be located in a tsunami hazard zone provided it is constructed in accordance with FEMA P646.
  2. Community critical facilities shall be permitted to be located within the tsunami hazard zone when such a location is necessary to fulfill their function, providing suitable structural and emergency evacuation measures have been incorporated.

Exception: These criteria shall not be considered mandatory for evaluation of existing buildings for evacuation planning purposes.

Reason: The amendments to Appendix M are necessary because the analysis and structural design aspects of FEMA P-646 (2012), Guidelines for Design of Structures for Vertical Evacuation from Tsunamis, have been superseded by ASCE 7-2016, Minimum Design Loads and Associated Criteria for Buildings and Other Structures. ASCE 7-16 now has a Chapter 6 on tsunami loads and effects, which also includes a set of tsunami design zone maps. As an accredited consensus-based standard, ASCE 7-16 incorporates more recent knowledge that takes precedence over the older FEMA guidelines. In particular, the FEMA guidelines for determining inundation depth, flow speed, and waterborne debris impact forces were found to lack reliability. The proposal updates Appendix M to make it refer to the tsunami evacuation and site planning criteria of P-646-12 and not to its tsunami hazard mapping and structural design guidelines, thereby removing conflicts that would otherwise occur between the two documents. The title of Appendix M is revised because the original title was overly broad; FEMA P646 only concerns tsunami refuge structures.

Cost Impact: Will not increase the cost of construction

Appendix M has not been adopted into the state or county codes of any of the five western states subject to significant tsunami hazard: Alaska, Washington, Oregon, California, and Hawaii.
Committee Action: Approved as Submitted

Committee Reason: This code change revises the tsunami appendix chapter to coordinate with the tsunami design provisions approved in S72-16.

Assembly Action: None

Individual Consideration Agenda

Proponent: Ed Berkel, representing ICC Code Correlation Committee (ccc@iccse.org) requests Disapprove.

Commenter's Reason: The Code Correlation Committee requests Disapproval of this code change proposal in order to bring a correlation issue to the attention of the full membership at the Public Comment Hearings and to allow the membership to coordinate action on this code change proposal with action taken on Code Change Proposal ADM94-16.

ADM94-16 is the administrative update to referenced standards in the I-Codes. One of these standards, ASCE7, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, was proposed for update to ASCE7-16. However, a successful assembly motion requests that the referenced ASCE7 remain at ASCE7-10 as it presently is referenced in the 2015 I-Codes. This code change proposal coordinates with and relies upon reference to ASCE7-16.

The Code Correlation Committee is a standing committee of the International Code Council whose objectives, procedures and organization are set forth in Council Policy CP#44-13. The objective of the Code Correlation Committee is to maintain technical and editorial consistency among the International Codes and to assist staff in the evaluation and processing of code change proposals and comments that are exclusively editorial.
Proposed Change as Submitted

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com)

2015 International Building Code

Delete and substitute as follows:

1603.1 General. Construction documents shall show the size, section and relative locations of structural members with floor levels, column centers and offsets dimensioned. The design loads and other information pertinent to the structural design required by Sections 1603.1.1 through 1603.1.8 shall be indicated on the construction documents.

- Exception: Construction documents for buildings constructed in accordance with the conventional light-frame construction provisions of Section 2308 shall indicate the following structural design information:
  1. Floor and roof live loads.
  2. Ground snow load, \( P_s \).
  3. Ultimate design wind speed, \( V_{ult} \) (3-second gust), miles per hour (mph) (km/hr) and nominal design wind speed, \( V_{asg} \) as determined in accordance with Section 1609.3.1 and wind exposure.
  4. Seismic design category and site class.
  5. Flood design data, if located in flood hazard areas established in Section 1612.3.
  6. Design load-bearing values of soils.

Construction documents shall show the size, section and relative locations of structural members with floor levels, column centers and offsets dimensioned. The design loads and other information pertinent to the structural design required by Sections 1603.1.1 through 1603.1.8 shall be indicated on the construction documents.

- Exception: Construction documents for buildings constructed in accordance with the conventional light-frame construction provisions of Section 2308 shall indicate the following structural design information:
  1. Floor and roof live loads.
  2. Ground snow load, \( P_s \).
  3. Ultimate design wind speed, \( V_{ult} \) (3-second gust), miles per hour (mph) (km/hr) and nominal design wind speed, \( V_{asg} \) as determined in accordance with Section 1609.3.1 and wind exposure.
  4. Site class.
  5. Flood design data, if located in flood hazard areas established in Section 1612.3.
  6. Design load-bearing values of soils.

Reason: seismic design category is deleted in its entirety throughout the code.

See Code Change: 1613.2 Definitions (D) for Reason/Cost Impact Statement

Cost Impact: Will increase the cost of construction

These are changes in terminology, for the purpose of clarifying both the intent of the code and the practice of earthquake engineering. Cost increase or decrease will be realized when the cited "lateral design strength parameters, or base shear coefficients," are actually used, as determined from Figures 16.13.1(1) through 1613.1(8). This more scientific approach reflects a much more straightforward and transparent of "seismic zonation," which is based upon the magnitude size of potential deterministic or scenario earthquakes.

This proposal may or may not affect the cost of construction, but only as a small portion of the less than 20% of total building cost that comprises the structural portion of a building. This is (1) because commercial buildings, as well as detached one- and two-family dwellings, must be already built to withstand the lateral forces due to wind; and (2) must include basements, "safe rooms", or other afforded protections to protect occupants against the deadly impacts of hurricanes and tornadoes.

The point is: Both Commercial buildings as well as detached one- and two-family need to consider the maximum Magnitude of realistic scenario earthquakes that they could, in fact, experience. And they should not be constructed vulnerable to earthquakes, because a flawed numerical hazard model "guesses" incorrectly as to the likelihood or possibility of earthquakes. This should remain a rational and a scientific decision based upon protecting both public safety and property. A second point is that "cost" due to structural elements is almost always less than 80% of the cost of a building!

"In general, better seismic performance is achieved through increased lateral design forces (i.e., base shear), and detailing requirements that improve structural connection strength or structural member behavior in the inelastic range of response. Requirements for seismic bracing and anchorage of nonstructural components reduce potential for nonstructural damage and loss of building (or system) functionality."

* viii, Executive Summary, NIST GCR 14-917-26

NEHRP Consultants Joint Venture A partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering.
In general, where costs might be increased, cost premiums above requirements for wind tend to fall within a range of +1-3%. For cases where seismic requirements would be now additional to what previous codes either applied/neglected/failed to enforce, estimates probably would fall within the range of 0.25 - 1%.

Committee Action: Disapproved
Assembly Action: None

Public Hearing Results
Committee Reason: Removing seismic design category would delete information that helps the engineer, owner and building official. It needs to be in the code.

Individual Consideration Agenda
Proponent : James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com) requests Approve as Submitted.
Commenter's Reason: "It is better to be right than to be consistent."
   - Winston Churchill

"If your definition is wrong, you'll look for the wrong thing."
   - Carol Cleland

“Anyone who conducts an argument by appealing to authority is not using his intelligence; he is just using his memory.”
   - Leonardo da Vinci

“Just because it comes from a consensus standard doesn't mean it isn't without problems.”
   - Jay Crandell

The Committee Reason for Disapproval is based on the Logical Fallacies: APPEAL to AUTHORITY; BEGGING the QUESTION (CIRCULAR REASONING); and STRAW MAN.

There is no need for seismic design categories, because the design values maps that are required to determine them have been recognized to be: (a) not only to be unstable between code cycles (values yo-yo); but (b) also fatally flawed - since the mathematics behind them is not correct. Moreover, the Exceptiononly applies to "buildings constructed in accordance with the conventional light-frame construction provisions of Section 2308". And "4. Site Class" does reflect lateral design strength coefficients (base shear) - that have superseded the previous design values maps, from which seismic design categories were determined.

See further background and discussion under Public Comment S118-16
Proposed Change as Submitted

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com)

2015 International Building Code
Delete and substitute as follows:

SECTION 202 DEFINITIONS

[BS] DESIGN EARTHQUAKE GROUND MOTION BASE SHEAR COEFFICIENT. The earthquake ground motion that buildings and structures are specifically proportioned to resist in Section 1613. The coefficient or factor that is reflecting the importance of increasing the lateral design strength of buildings (base shear) above that for moderate magnitude earthquakes (M 5-5.9); for strong (M 6-6.9), major (7-7.9), great (M 8 or more), and also giant megathrust or subduction zone earthquakes (M 9 or more) - such as the M 9.2 1964 Alaska Earthquake and Tsunami.

Delete without substitution:

202 [BS] RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCEp) GROUND MOTION RESPONSE ACCELERATIONS. The most severe earthquake effects considered by this code, determined for the orientation that results in the largest maximum response to horizontal ground motions and with adjustment for targeted risk.

202 [BS] SEISMIC DESIGN CATEGORY. A classification assigned to a structure based on its risk category and the severity of the design earthquake ground motion at the site.

Revise as follows:

1613.2 Definitions. The following terms are defined in Chapter 2:
   DESIGN EARTHQUAKE GROUND-MOTION BASE SHEAR COEFFICIENT.
   ORTHOGONAL.
   RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCEp) GROUND MOTION RESPONSE SPECTRUM ACCELERATION.
   SEISMIC DESIGN CATEGORY FORCE-RESISTING SYSTEM.
   SEISMIC FORCE-RESISTING SYSTEM SITE CLASS.
   SITE CLASS COEFFICIENTS.
   SITE COEFFICIENTS.

Reason: Deleting DESIGN EARTHQUAKE GROUND MOTION and replacing with EARTHQUAKE BASE SHEAR COEFFICIENT.

Defined as: The coefficient or factor that is reflecting the importance of increasing the lateral design strength of buildings (base shear) above that for moderate magnitude earthquakes (M 5-5.9); for strong (M 6-6.9), major (7-7.9), great (M 8 or more), and also giant megathrust or subduction zone earthquakes (M 9 or more) - such as the M 9.2 1964 Alaska Earthquake and Tsunami.

To more realistically make clear to both code officials and design professionals as well - that the code does not design for a particular ground motion: the code only says – “DO THIS TO ACHIEVE THIS STRENGTH.” Furthermore, since, “in general, better seismic performance is achieved through increased lateral design forces (i.e., base shear),” particularly for long-duration shaking of up to five minutes, and the larger magnitudes of aftershocks that typically follow strong (M6-6.9), major (M7-7.9), great (M8 or more), and giant Cascadia type megathrust earthquakes (M9 or more); this encourages the design professional to both examine and think about the specific scenario earthquakes that may attack the site. The so-called ground motion contours on present code mandated design procedures are artifacts of seismic hazard models (scalar numbers only that do not reflect the tensor or directional natures of real earthquake ground motions, nor their frequency contents or durations of shaking). If the numbers are “low,” engineers often incorrectly assumed that these will be the values of ground shaking they are designing for. If, alternatively, the numbers are “high,” engineers balk, “because we have never designed for that before.”

Deleting RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCEp) RESPONSE ACCELERATION and replacing with RESPONSE SPECTRUM ACCELERATION.

To rectify the mischaracterization that we can replace the tensor (magnitude and directional) nature of real earthquake ground motion (magnitude, frequency content, duration, aftershocks) with a scalar quantity (number value only); which, because it is fictitious, has little to do with assessing the true effects of scenario earthquakes that can impact the site. So-called “Risk-Targeted” earthquake ground motions were copied from ASCE protocols for the design of nuclear plants, and they are neither...
adequate nor applicable for building code applications to protect public safety from the potential earthquake threats that may occur. So-called Risk Models are strongly utilized by the insurance market, but as models have been widely criticized for being non-transparent, subjective, blurring distinctions between assumptions and facts (and too often with a weak understanding of the limitations of the model assumptions.” Furthermore, just knowing what the risk is . . . I mean what you calculated it to be: doesn't mean that you know “what to do about it!”

Furthermore, there have been no “logical” arguments for implementing RISK-TARGETED’s conceptual language into code design requirements; and its completely arbitrary and subjective choices for representing risk (10%/50yr; 2%/50 yr; or anything else) are propped up more by well-known “logical fallacies” and ignorance, rather than they are by actual scientific fact and “common sense.”

Deleting SEISMIC DESIGN CATEGORY in its entirety
This is deleted in its entirety because: (1) the implication is that you can reduce the earthquake resistance of some constructions in harm's way by considering arbitrary and particularly fictitious earthquake ground motions in lieu of real or scenario earthquakes, which is not logical; (2) in engineering design practice, you can no longer look-at-a-map and instinctively and realistically know what your design values (minimum base shear) should be; (3) SDCs are completely at the mercy of the 25-30% yo-yo-ing oscillations of so-called design ground motion maps, which have been continually imposed upon the building community by the U.S. Geological Survey, despite their detrimental short-comings (30% decrease in design base shear for Mineral, Virginia over a 10-yr period prior to the M 5.8 2011 earthquake!), which have been well documented during tUSGS's supposed “users' workshops!”, and (4) in our present-day desire for communities to both plan for and achieve resilience in their both well-recognized and certain earthquake futures, seismic design categories are blatantly anti-resilience impediments to such a goal.

(1) SDCs do not realistically reflect the Magnitudes of earthquakes that may impact, in particular, “Detached one- and two-family dwellings,” nor their associated real intensities of shaking (accelerations and velocities, including pga and pgv); and (2) the contour seismic hazard-model maps, upon which the assigned SDCs are determined, are (a) numerical creations without physical reality; (b) mathematically flawed and incorrect (because a dimensionless number, the probability-in-one-year, is arbitrarily assigned dimensional units of “per yr.” or annual frequency – leading to the improperly applied notion of a so-called earthquake “return period” as the basis for determining and assigning earthquake design loads; and (c) non-stable between iterative cycles of creations (sometimes varying 25-30% between issuance); and (d) $S_S$ and $S_1$ or Spectral Response Acceleration based seismic design maps are still very confusing, misunderstood; and they are most certainly incorrectly interpreted or misunderstood by all of the vast entities (state decision makers, code officials, design professionals, contractors, and probably even the preponderance of ICC Committee members - as well as Hearings attendees!

REFERENCES:

1988 Uniform Building Code

1990 SEAOC BLUE BOOK

1997 Uniform Building Code

Robert E. Bachman and David R. Bonneville (2000)
For example, see TAKE ME HOME SEISMIC LOADS

Image: Moderate Magnitude M 5.8 Mineral, Virginia EQ Aug. 23, 2011  Shaking Intensity MMI VII - VIII

Bibliography:
Earthquake Magnitude Scale and Class
http://www.geo.mtu.edu/UPSeis/magnitude.html (http://www.geo.mtu.edu/UPSeis/magnitude.html)

M 5.8 Aug 11, 2011 Mineral VA Earthquake
(http://earthquake.usgs.gov/earthquakes/eqinthenews/2011/se082311a/)

The Mw 5.8 Virginia Earthquake of August 23, 2011

Residential and Building Damage near epicenter: M 5.8 Mineral, Virginia EQ  MMI VII – VIII

Louisa County High School Building Damage
(http://www.eqclearinghouse.org/2011-08-23-virginia/2011/09/02/louisa-county-high-school-damage/)

Damage from M 6.0 Wells, Nevada EQ 2008
https://www.google.com/search?q=wells+nevada+earthquake+2008&biw=1280&bih=899&tbm=isch&tbo=u&source=univ&sa=X&ved=0ahUKEwjO7KmNo__JAhULzGMK
q=wells+nevada+earthquake+2008&biw=1280&bih=899&tbnid=3mgjtjyo9K4bLM&ved=0ahUKEwjO7KmNo__JAhULzGMK

Cost Breakdown of Nonstructural Building Elements

Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake.
doi: http://dx.doi.org/10.1193/1.4000032 (http://dx.doi.org/10.1193/1.4000032)
http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032

Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Permalink: http://dx.doi.org/10.1061/(ASCE)CF.1943-5509.0000630
Read More: http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630

Homeowner’s Guide to Earthquake Safety

Retrofitting Questions and Answers
Earthquake Safety, Inc., 2015 (web based)
http://www.earthquakesafety.com/earthquake-retrofitting-faq.html

Cost and Seismic Design
https://www.google.com/#q=cost+and+seismic+design+christopher+arnold+pdf

Earthquake Architecture website
http://www.iitk.ac.in/nicee/wcee/article/14_05-06-0185.PDF

Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee

Cost Impact: Will not increase the cost of construction
These are definitions, for the purpose of clarifying both the intent of the code and the practice of earthquake engineering. This proposal may or may not affect the cost of construction. This is (1) because detached one- and two-family dwellings must be already built to withstand the lateral forces due to wind; and (2) must include basements, “safe rooms”), or other afforded protections to protect occupants against the deadly impacts of hurricanes and tornadoes.
The point is; Detached one- and two-family need to consider the maximum Magnitude of realistic scenario earthquakes that they could, in fact, experience. And they should not be constructed vulnerable to earthquakes, because a flawed numerical hazard model "guesses" incorrectly as to the likelihood or possibility of earthquakes. This should remain a rational and a scientific decision based upon protecting both public safety and property. A second point is that "cost" due to structural elements is almost always less than 80% of the cost of a building!

"In general, better seismic performance is achieved through increased lateral design forces (i.e., base shear), and detailing requirements that improve structural connection strength or structural member behavior in the inelastic range of response. Requirements for seismic bracing and anchorage of nonstructural components reduce potential for nonstructural damage and loss of building (or system) functionality."*

* viii, Executive Summary, NIST GCR 14-917-26


NEHRP Consultants Joint Venture A partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering.

In general, where costs might be increased, cost premiums above requirements for wind tend to fall within a range of +1-3%. For cases where seismic requirements would be now additional to what previous codes either applied/neglected/failed to enforce, estimates probably would fall within the range of 0.25 - 1%.

Public Hearing Results

Committee Action: Disapproved

Committee Reason: This code change would leave a number of terms undefined in the IBC, which would lead to conflicts with the referenced standard, ASCE 7.

Assembly Action: None

Individual Consideration Agenda

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com) requests Approve as Submitted.

Commenter's Reason: "It is better to be right than to be consistent."
- Winston Churchill

"If your definition is wrong, you'll look for the wrong thing."
- Carol Cleland

“Anyone who conducts an argument by appealing to authority is not using his intelligence; he is just using his memory.”
- Leonardo da Vinci

“Just because it comes from a consensus standard doesn't mean it isn't without problems.”
- Jay Crandell

The Committee Reason for Disapproval is based on the Logical Fallacies: APPEAL to AUTHORITY; BEGGING the QUESTION (CIRCULAR REASONING); and STRAW MAN.
What is a Straw Man fallacy?

The Straw Man fallacy is committed when a person simply ignores a person’s actual position and substitutes a distorted, exaggerated or misrepresented version of that position. This sort of “reasoning” has the following pattern:

- Person A has position X.
- Person B presents position Y (which is a distorted version of X).
- Person B attacks position Y.
- Therefore X is false/incorrect/flawed.

The terms deleted have been deleted because they are no longer used in the IBC. So the Committee Reason is irresponsibly FALSE!

See further background and discussion under Public Comment S118-16

Bibliography: See BIBLIOGRAPHY under Public Comment S118-16
S318-16
IBC: 1613.3.1.

*Proposed Change as Submitted*

**Proponent:** James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com)

**2015 International Building Code**

Revise as follows:

1613.3.1 Mapped acceleration lateral design strength parameters. The lateral design strength parameters $S_5$ and $S_1$, or base shear coefficients, shall be determined from the 0.2 and 1 second spectral response accelerations shown on Figures 1613.3.1(1) through 1613.3.1(8) and 1613.1(8) Where $S_1$ is less than or equal to 0.15, the structure is permitted to be

\[
S_5 = \text{less than or equal to } 0.04 \quad \text{or} \quad S_1 = \text{less than or equal to } 0.15
\]

or

- Reason: A fundamental rule, when hiring new employees (at least as applies with regards to breweries), is that they "should solve more problems than they create." Unfortunately, the mapped parameters $S_5$ and $S_1$, which are "determined from the 0.2 and 1 second response accelerations shown [to two decimal places]" on seismic design maps prepared by the U.S. Geological Survey (USGS), have (for too long in the past) and continue to *create more problems than they solve*! This because "Probabilistic seismic hazard analysis (PSHA) [the linchpin of the USGS seismic hazard model] is [no longer just] beginning to be seen as *unreliable*. The problem with PSHA is that its data are *inadequate* and its logic is *defective*. Much more reliable, and more scientific, are deterministic procedures, especially when coupled with engineering judgment." [Castanos and Lomnitiz (2002)]

Before listing some of these problems, it is important to consider the primary FORCES (powerful . . . but never truly AWAKENED) behind the present situation and conundrum!

Despite a truly *scientific revolution* in our understanding of the earth and of its plate tectonics, and despite also a *technological revolution* in our abilities to monitor in real-time both its heartbeats as well as its seismic awakenings (M8 Algorithm for twenty years and now electromagnetic precursors to large earthquake phenomena), earthquakes have continued to release their destructive forces, "and our society has failed to cope with them." And since we have become *addicted* (or at least habituated) to this USGS seismic hazard model for producing mapped acceleration parameters, which is as *irrational* as it is *perilous*, "we are unable to protect our natural environment from [earthquake] destruction." Community resilience requires being prepared for the earthquake after the next one, not just recovering from the next one!

The increasing importance of so-called "experts,"(who at the same time are remunerated as *Shamans* who can ostensibly foretell the future by their special abilities to: (1) interpret the *complexities of response accelerations* to two decimal places of accuracy; (2) further communicate with a code-spirit-world to assign: (a) *Seismic Design categories* SDCs; (b) *Risk Categories* I, II, III, IV; and (c) *Response Modification or R-Factors* (and for at least as long as things remain the same in their code universe), has fundamentally changed the code development game. And when this reality is further coupled with the *complexities* (far beyond normal awareness and understanding of those who are on the *receiving* end) of the *seismic hazard models* now forming the basis for determining these "mapped acceleration parameters," the combined result is the greatly "increased the ignorance of the public and its elected representatives" in matters of public safety and future economic security and well-being. And in a really nondemocratic way, *power and control* is becoming more-and-more concentrated in *both* the USGS and also a *code development elite* that seems to pride itself on serving for life.

To a design professional, a building is the 20% or less in cost forming the *structural* portion of the building. To a building owner and also to a community, a building is the 80% or more in total building cost that *provides* both the function and form of daily (hopefully safe) living and economic activities, i.e. *nonstructural* elements and building *contents*! This mismatch in goals is burdening communities with non-resilience and throw-away buildings. As they are more-and-more embracing a word, *resilience*, that they do understand; at the same time, because of the systemically "imposed ignorance," endemic in the present process, these same communities (non-participants) are virtually *unable* to understand what defective products are being delivered to them via a new (but code minimum) building.

I believe that code officials and code development committees have "the additional responsibility to educate the public, to evaluate the long-term social consequences of [their] endeavors, and to provide guidance in the formation of relevant public policy." But this has become ever harder to do, due to an "increased ignorance" of all! The nondemocratic processes of the ICC, which makes a mockery of any mention of "Roberts Rules of Order," provide an additional barrier.

"When you adopt a code, you are really saying: *this is all right!* We realize you are taking such-and-such a chance of being killed, but . . ."

But it is not all right! (1) **It is outdated!** While the earth sciences have matured immeasurably since the beginning of *plate tectonics*, somewhere around 1968, with *gps precision tracking* (in mm scale) of earth movements due to tectonic forces, *seafloor mapping* and *underwater coring* of megathrust earthquake *turbidite* (undersea landslides triggered by earthquakes)
records to assess Magnitude and recurrence intervals, tsunami modeling and NOAA DART real-time ocean buoys for tsunami warning, onshore paleoseismic studies of coastal marshes and estuaries with regard to the prehistoric record of Cascadia subduction zone earthquakes, even synthetic modeling of earthquake waves and their associated ground motions, etc.; the USGS seismic hazard model remains married to the original 1968 Alin Cornell proposal for “engineering seismic risk analysis.” No one in the “smart” community ever figured out on their own, including within the iccsafe framework, that (a) a dimensionless number is just that . . . a dimensionless number and not an annual frequency, of earthquakes per year; and (b) uncertainty is still uncertainty! - even if quantified with delusional precision.

(2) It is nonstable! The design values can change dramatically between successive iterations of the “mapped acceleration parameters” USGS products, sometimes yo-yoing as much as 25-30%! Thus an engineer coming up through the ranks in a design office becomes hampered in developing judgment over hopefully both a long and productive career. Lateral Design strength or base shear (even though it is but a small percentage of the 20% or less in “structural” cost of a building), an established predictor of good earthquake performance (probably because it historically has provided a lot of the overstrength that is part of the Response Modification or R factors that reduce numbers used in a final design) is diminished as a tool of the responsible design engineer, who now may leave a legacy of many identical buildings designed within the same geographic area, but to vastly different requirements – all someday to experience the same earthquake! The very real problem of existing Killer Buildings, generally those constructed of non-ductile concrete (meaning that their deflection is largely governed by their strength), is harder to attack, if the new code design requirements suddenly are lowered, now placing these same buildings outside of previous “trigger requirements” for retrofit action under community ordinances! Which seismic hazard map do you choose to protect public safety?

See also: Take Me Home, Seismic Lodes.

(3) It is a puzzle! . . . and not a solution to a problem! The USGS provided mapped acceleration parameters \( S_S \) and \( S_1 \), which are based upon the unreliable methodology of psha, are starting points for solving a puzzle. But the earthquake is a problem. Simply put, if something has happened, it can happen! So to protect both public safety as well as the 80% of building costs that the public sees to be their building, it’s time to see the earthquake as a problem and to more explicitly and transparently require a solution to that problem . . . hopefully before the SEISMIC FORCE AWAKENS!

Issac Newton (F = ma fame) says it best: “Truth is ever to be found in simplicity, and not in the multiplicity and confusion of things.”

Where is the wisdom we have lost in knowledge?
Where is the knowledge we have lost in information?

- T.S. Eliot

References:

1988 Uniform Building Code

1990 SEAOC BLUE BOOK

1997 Uniform Building Code
Robert E. Bachman and David R. Bonneville (2000)

"... and the problem depends just on you."
- Erno Rubik

Bibliography:

FORUM: Improving Earthquake Hazard Assessments in Italy: An Alternative to "Texas Sharpshooting."
Eos Vol. 93, No. 51 18 December 2012

"Texas Sharpshooter" Fallacy
http://www.investopedia.com/terms/t/texas-sharpshooter-fallacy.asp

Earthquake Magnitude Scale and Class
http://www.geo.mtu.edu/UPSeis/magnitude.html

M 5.8 Aug 11, 2011 Mineral VA Earthquake

The Mw 5.8 Virginia Earthquake of August 23, 2011

Residential and Building Damage near epicenter: M 5.8 Mineral, Virginia EQ MMI VII – VIII

Louisa County High School Building Damage

Damage from M 6.0 Wells, Nevada EQ 2008
https://www.google.com/search?q=wells+nevada+earthquake+2008&biw=1280&bih=899&tbs=isch&tbo=u&source=univ&sa=X&ved=0ahUKEwjO7KmNo__JAhULzGMKJADEdAIQ_AUoAQ
https://disastersafety.org/ibhs-risks-earthquake/

Cost Breakdown of Nonstructural Building Elements

Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake.
doi:http://dx.doi.org/10.1193/1.4000032
http://www.earthquakespectra.org/doi/abs/10.1193/1.4000032

Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Permalink: http://dx.doi.org/10.1061/(ASCE)CF.1943-5509.0000630
Read More: http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630
Cost Impact: Will increase the cost of construction
These are changes in terminology, for the purpose of clarifying both the intent of the code and the practice of earthquake engineering. Cost increase or decrease will be realized when the cited “lateral design strength parameters, or base shear coefficients,” are actually used, as determined from Figures 16.13.1(1) through 1613.1(8). This more scientific approach reflects a much more straightforward and transparent of “seismic zonation,” which is based upon the magnitude size of potential deterministic or scenario earthquakes.
This proposal may or may not affect the cost of construction, but only as a small portion of the less than 20% of total building cost that comprises the structural portion of a building. This is (1) because commercial buildings, as well as detached one- and two-family dwellings, must be already built to withstand the lateral forces due to wind; and (2) must include basements, “safe rooms”, or other afforded protections to protect occupants against the deadly impacts of hurricanes and tornadoes.
The point is: Both Commercial buildings as well as detached one- and two-family need to consider the maximum Magnitude of realistic scenario earthquakes that they could, in fact, experience. And they should not be constructed vulnerable to earthquakes, because a flawed numerical hazard model “guesses” incorrectly as to the likelihood or possibility of earthquakes. This should remain a rational and a scientific decision based upon protecting both public safety and property. A second point is that “cost” due to structural elements is almost always less than 80% of the cost of a building!
"In general, better seismic performance is achieved through increased lateral design forces (i.e., base shear), and detailing requirements that improve structural connection strength or structural member behavior in the inelastic range of response. Requirements for seismic bracing and anchorage of nonstructural components reduce potential for nonstructural damage and loss of building (or system) functionality.”*

In general, where costs might be increased, cost premiums above requirements for wind tend to fall within a range of +1-3%. For cases where seismic requirements would be now additional to what previous codes either applied/neglected/failed to enforce, estimates probably would fall within the range of 0.25 - 1%.

{{1143}}
Individual Consideration Agenda

Proponent: James Bela, representing Oregon Earthquake Awareness (sasquake@gmail.com) requests Approve as Submitted.

Commenter’s Reason: "It is better to be right than to be consistent."
- Winston Churchill

"If your definition is wrong, you’ll look for the wrong thing."
- Carol Cleland

“Anyone who conducts an argument by appealing to authority is not using his intelligence; he is just using his memory.”
- Leonardo da Vinci

“Just because it comes from a consensus standard doesn’t mean it isn’t without problems.”
- Jay Crandell

“I break for circular reasoning, and this is definitely the Last Straw!

The Committee Reason for Disapproval is based on the Logical Fallacies: APPEAL to AUTHORITY; BEGGING the QUESTION (CIRCULAR REASONING); and STRAW MAN.

What is a Straw Man fallacy?

- The Straw Man fallacy is committed when a person simply ignores a person’s actual position and substitutes a distorted, exaggerated or misrepresented version of that position. This sort of “reasoning” has the following pattern:
  - Person A has position X.
  - Person B presents position Y (which is a distorted version of X).
  - Person B attacks position Y.
  - Therefore X is false/incorrect/flawed.

(1) There is no “undefined terminology” - words are simple and more descriptive than the terms they are replacing. When in doubt or undefined, words have their common dictionary meanings. For example, “base shear” means/is “an estimate of the maximum expected lateral force that will occur due to seismic ground motion at the base of a structure. Calculations of base shear (V) depend on: soil conditions at the site, proximity to potential sources of seismic activity (such as geological faults).”

(2) The Proponent Reason Statement more than adequately explains the Proposal!
See further background and discussion under Public Comment S118-16

Bibliography: See BIBLIOGRAPHY in S318-16 and also in S118-16

S318-16