IBC - Structural

2016 GROUP B COMMITTEE ACTION HEARINGS

APRIL 17, 2016 – APRIL 27, 2016
KENTUCKY INTERNATIONAL
CONVENTION CENTER
LOUISVILLE, KY

INTERNATIONAL CODE COUNCIL®

First Printing

Publication Date: March 2016

PRINTED IN THE U.S.A.

2016 GROUP B – PROPOSED CHANGES TO THE INTERNATIONAL BUILDING CODE – STRUCTURAL

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TENTATIVE ORDER OF DISCUSSION
2016 PROPOSED CHANGES TO THE
INTERNATIONAL BUILDING CODE - STRUCTURAL

The following is the tentative order in which the proposed changes to the code will be discussed at the public hearings. Proposed changes which impact the same subject have been grouped to permit consideration in consecutive changes.

Proposed change numbers that are indented are those which are being heard out of numerical order. Indentation does not necessarily indicate that one change is related to another. Proposed changes may be grouped for purposes of discussion at the hearing at the discretion of the chair. Note that some S code change proposals may not be included on this list, as they are being heard by another committee.

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IBC: 1401.1, 1501.1.

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; <u>and</u> architectural *trim*<u>.</u>~~; 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 <u>, and balconies where the structural framing is protected by an impervious moisture barrier</u>.

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 seperate proposal. Since a balcony outside of the building envelope that has w eather 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 w as not modified w hen Group A code change FS15-15 removed Balconies, similar projections and Bay and oriel w indow s 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.

S1-16 : 1501.1-RICHARDSON12728

S2-16

IBC: 1503.1, 1503.2.

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

2015 International Building Code

Revise as follows:

1503.1 General. Roof decks shall be covered with *approved* roof coverings secured to the building or structure in accordance with the provisions of this chapter. Roof coverings shall be designed in accordance with this code, and installed in accordance with this code and the ~~*approved*~~manufacturer's *approved* instructions~~ such that the roof covering shall serve to protect the building or structure~~.

1503.2 Flashing. Flashing shall be installed in such a manner so as to prevent~~ moisture~~ liquid water from entering the wall and roof through joints in copings, through moisture-permeable materials and at intersections with parapet walls and other penetrations through the roof plane.

Reason: The current code includes references to "approved manufacturer's instructions" w hich tends to indicate that it is addressing instructions from approved manufacturers. Since it is the instructions that are approved, not the manufacturer, the proposed text is grammatically correct. The proposal further removes the phrase regarding protection of "the building or structure" w hich is undefined and vague, and changes the term "moisture" (w hich could include w ater vapor) to "liquid w ater" to more correctly capture the role of flashing materials and assemblies.

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

S2-16 : 1503.1-FISCHER13524

S3-16

IBC: 1503.1, 1507.4.2, 1507.5.6.

Proponent : Andy Williams (afwilliams@Connect2amc.com)

2015 International Building Code

Revise as follows:

1503.1 General. Roof decks shall be covered with *approved* roof coverings secured to the building or structure in accordance with the provisions of this chapter. Roof coverings shall be designed and installed in accordance with this code and the~~*approved*~~ manufacturer's approved installation instructions such that the roof covering shall serve to protect the building or structure.

1507.4.2 Deck slope. Minimum slopes for metal roof panels shall comply with the following:

1. The minimum slope for lapped, nonsoldered seam metal roof panels without applied lap sealant shall be three units vertical in 12 units horizontal (25-percent slope).
2. The minimum slope for lapped, nonsoldered seam metal roof panels with applied lap sealant shall be one-half unit vertical in 12 units horizontal (4-percent slope). Lap sealants shall be applied in accordance with the manufacturer's approved ~~manufacturer's~~ installation instructions.
3. The minimum slope for standing-seam metal roof panel systems shall be one-quarter unit vertical in 12 units horizontal (2-percent slope).

1507.5.6 Attachment. Metal roof shingles shall be secured to the roof in accordance with the ~~*approved*~~manufacturer's approved installation instructions.

Reason: Use of the w ord "approved" as it is currently positioned in these sections leads the reader to believe that there is some process that should be applied to the manufufacturer. The intent is that the instructions are the "approved" subject.. The requirements for approved installation instructions should apply in all cases.

Cost Impact: Will not increase the cost of construction
This is simply a clarification of the intent of the code.

S3-16 : 1503.1-WILLIAMS11544

S4-16

IBC: 1503.3.

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

2015 International Building Code

Revise as follows:

1503.3 Coping. Parapet walls shall be properly coped with noncombustible, weatherproof materials or with a roof covering that can be used as part of a roofing system assembly that has an equal or greater fire classification than the system used on the actual roof. The fire classification shall be based on testing conducted in accordance with Section 1505.1. The width of the coping shall be no less than the thickness of the parapet wall.

Reason: This proposal w ould allow the use of fire classified roof membranes for coping of parapet w alls. This w ill allow a greater variety of options for w aterproofing the parapet w all, and provide additional options for maintaining a continuous air barrier. For example the roof membrane could be used to w rap the top of the parapet w all and extend dow n the exterior side of the w all. The membrane could then be tied into the w all air barrier system.

Cost Impact: Will not increase the cost of construction
No additional materials or detailing w ill be required based on this code change proposal, therefore it w ill not increase the cost of construction.

S4-16 : 1503.3-ENNIS11189

S5-16

Part I:
IBC: 1503.5, 1511.7 (New); IEBC: 706.7 (New).
Part II:
IRC: R908.7 (New).

THIS IS A 2 PART CODE CHANGE. PARTS I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC-BUILDING COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

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

Part I

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.

Part II

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 betw een 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.

S5-16 : 1503.5-FISCHER13549

S6-16

IBC: 1503.6 (New).

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

2015 International Building Code

Add new text as follows:

1503.6 Enclosed eave ventilation Vents shall be provided in accordance with Section 1203.2 where roof eaves or porch soffits are enclosed creating concealed space outside of the building envelope.

Reason: Section 1203.2 provides the requirements for ventilation of rafter spaces that are enclosed by a ceiling above the interior environment. Since this section is in Chapter 12 it is not clear that enclosed rafter spaces and attic space outside of the building envelope on a porch or eave must also be ventilated. Subsequent numbers are renumbered.

Cost Impact: Will not increase the cost of construction
This change does not add additional requirements. It clarifies the intent of the current code.

S6-16 : 1503.6 (NEW)-RICHARDSON12268

S7-16

IBC: 1503.7 (New).

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 w hen a balcony or elevated w alking surface serves as a w eather 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 w alking 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 w ith hand tools already comply w ith the intent of this requirement.

S7-16 : 1503.7 (NEW)-RICHARDSON12270

S8-16
Part I:
IBC: 1504.1.1.
Part II:
IRC: R905.2.4.1.

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC-BUILDING COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

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

Part I

2015 International Building Code

Revise as follows:

1504.1.1 Wind resistance of asphalt shingles. Asphalt shingles shall be tested in accordance with ASTM D 7158. Asphalt shingles shall meet the classification requirements of Table 1504.1.1 for the appropriate maximum basic wind speed. Asphalt shingle packaging shall bear a label to indicate compliance with ASTM D 7158 and the required classification in Table 1504.1.1.

> **Exception:** Asphalt shingles ~~that are~~ not included in the scope of ASTM D 7158 shall be tested and labeled <u>in accordance with ASTM D 3161. Asphalt shingle packaging shall bear a label</u> to indicate compliance with ASTM D 3161 and the required classification in Table 1504.1.1.

Part II

2015 International Residential Code

Revise as follows:

R905.2.4.1 Wind resistance of asphalt shingles. Asphalt shingles shall be tested in accordance with ASTM D 7158. Asphalt shingles shall meet the classification requirements of Table R905.2.4.1 for the appropriate ultimate design wind speed. Asphalt shingle packaging shall bear a label to indicate compliance with ASTM D 7158 and the required classification in Table R905.2.4.1.

> **Exception:** Asphalt shingles not included in the scope of ASTM D 7158 shall be tested and labeled <u>in accordance with ASTM D 3161. Asphalt shingle packaging shall bear a label</u> to indicate compliance with ASTM D 3161 and the required classification in Table R905.2.4.1.

Reason: This proposal is an editorial clarification that aligns the labeling requirement for the wind standards for asphalt shingles to provide consistent labeling requirements between the IRC and IBC.

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

S8-16 : 1504.1.1-FISCHER13527

S9-16

IBC: 1504.2.1.1.

Proponent : Rob Brooks, Rob Brooks and Associates, LLC representing Dow Building Solutions, representing Rob Brooks and Associates, LLC representing Dow Building Solutions (rob.brooks.mail@gmail.com)

2015 International Building Code

Revise as follows:

1504.2.1.1 Overturning resistance. Concrete and clay roof tiles shall be tested to determine their resistance to overturning due to wind in accordance with SBCCI SSTD 11 <u>or ASTM C 1568,</u> and Chapter 15.

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
<u>ASTM C1568-08(2013), *Standard Test Method for Wind Resistance of Concrete and Clay Roof Tiles (Mechanical Uplift Resistance Method)*</u>

Reason: In 2003, ASTM International Subcommittee C15.06 replicated SSTD 11-99 by subdividing the SBCCI standard into three different ASTM standards: 1) ASTM C1568-03, *Standard Test Method for Wind Resistance of Concrete and Clay Roof Tiles (Mechanical Uplift Resistance Method)*, 2) ASTM C1569-03, *Standard Test Method for Wind Resistance of Concrete and Clay Roof Tiles (Wind Tunnel Method)*, and 3) ASTM C1570-03, *Standard Test Method for Wind Resistance of Concrete and Clay Roof Tiles (Air Permeability Method)*.

The cross-correlation of ASTM C1568 and SSTD 11 is as follows:

C1568 Section 5 relates to SSTD 11 Section 201

C1568 Section 6 relates to SSTD 11 Sections 202, 204 and 205

C1568 Section 7 relates to SSTD 11 Section 203

C1568 Section 8 relates to SSTD 11 Section 206

C1568 Section 9 relates to SSTD 11 Section 207

C1568 Section 10 relates to SSTD 11 Section 300

C1568 Section 11 relates to SSTD 11 Section 400

C1568 Section 12 relates to SSTD 11 Section 500

C1568 Section 13 relates to SSTD 11 Section 600

C1568 Section 14 relates to SSTD 11 Section 700

There are no technical changes proposed with this code change request. ASTM C1568 is simply a duplication of the relevant sections of SSTD 11-99 with regard to the mechanical uplift resistance method. This modification now references a consensus standard that will have the capability to be updated in the future, as SSTD 11 has not been updated since 1999.

Bibliography: Additional information on the background and development of the ASTM standards is available at http://www.rci-online.org/interface/2014-11-smith-masters-gurley.pdf

Cost Impact: Will not increase the cost of construction
The ASTM standard replicates the current requirements of SBCCI SSTD-99, and therefore will increase the cost of construction.

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM C1568, 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.

S10-16

IBC: 1504.3.

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

2015 International Building Code

Revise as follows:

1504.3 Wind resistance of nonballasted roofs. Roof coverings installed on roofs in accordance with Section 1507 that are mechanically attached or adhered to the roof deck shall be designed to resist the design wind load pressures for components and cladding in accordance with Section ~~1609~~ 1609.5.2, the wind load on the roof covering is permitted to be determined using an allowable stress wind load of 0.5W instead of 0.6W in accordance with Section 1605.3.

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 300 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.

S10-16 : 1504.3-ENNIS11970

S11-16

IBC: 1504.3.

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

2015 International Building Code

Revise as follows:

1504.3 Wind resistance of nonballasted roofs. Roof coverings installed on roofs in accordance with Section 1507 that are mechanically attached or adhered to the roof deck shall be designed to resist the design wind load pressures for components and cladding in accordance with Section ~~1609~~ 1609.5.2. The wind load on the roof covering shall be determined using allowable stress design.

Reason: This proposal is being submitted to provide clarification regarding how to calculate wind loads on roof coverings. The introduction of ASCE 7-10 into the IBC, and its use of ultimate design wind speeds has caused confusion with design professionals when calculating wind loads on roof coverings. This proposal does not change any requirements, it simply clarifies that allowable stress wind loads shall be used for roof coverings.

Cost Impact: Will not increase the cost of construction
This proposal is a clarification regarding how to calculate wind loads on roof coverings. It will have no impact on the cost of construction.

S11-16 : 1504.3-ENNIS12915

S12-16

IBC: 1504.3.1.1 (New).

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

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.

S12-16 : 1504.3.1.1 (NEW)-ENNIS11191

S13-16

IBC: 1504.3.3 (New).

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 w ith the requirements of Chapter 15. Section 1504.1 describes requirements for the w ind resistance of roofs, how ever no guidance is provided for designing garden and landscaped roofs for w ind resistance. This proposal requests the addition of Section 1504.3.3, under Section 1504.3 Wind resistance of nonballasted roofs, that w ill provide requirements for garden and landscaped roofs. This new section w ould 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 w as 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 w ind speeds, building heights, exposure conditions, parapet heights and special conditions that may exist on the roof. The standard w as developed in cooperation w ith Green Roofs for Healthy Cities and contains important information developed by the National Roofing Contractors Association regarding potential blow -off of grow th media. Specifically the standard requires that any areas of exposed grow th media in excess of 4-inches have a system to prevent grow th media blow -off.

Cost Impact: Will increase the cost of construction
Because the ANSI/SPRI RP-14 standard requires that areas w ith exposed grow th media in excess of 4-inches have a system to prevent grow th 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, w ith regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) w ill be posted on the ICC w ebsite on or before April 1, 2016.

S13-16 : 1504.3.3 (NEW)-ENNIS10874

S14-16

IBC: 1504.1.1, 1504.3.3 (New).

Proponent : Andy Williams (afwilliams@Connect2amc.com)

2015 International Building Code

Revise as follows:

TABLE 1504.1.1
CLASSIFICATION OF ~~ASPHALT~~ STEEP SLOPE ROOF SHINGLES TESTED IN ACCORDANCE WITH ASTM D 7158 OR D 3161

(Portions of table not shown remain unchanged)

For SI: 1 foot = 304.8 mm; 1 mph = 0.447 m/s.

a. The standard calculations contained in ASTM D 7158 assume Exposure Category B or C and building height of 60 feet or less. Additional calculations are required for conditions outside of these assumptions.

Add new text as follows:

1504.3.3 Metal roof shingles. Metal roof shingles applied to a solid or closely fitted deck shall be tested in accordance with FM 4474, UL 580, UL 1897, or ASTM D 3161. Metal roof shingles tested in accordance with ASTM D 3161 shall meet the classification requirements of Table 1504.1.1 for the appropriate maximum basic wind speed and the metal shingle packaging shall bear a label to indicate compliance with ASTM D 3161 and the required classification in Table 1504.1.1.

Reference standards type: This is an update to reference standard(s) already in the ICC Code Books

Add new standard(s) as follows:

ASTM D3161/D3161M-~~13~~15: Standard Test Method for Wind-Resistance of Steep Slope Roofing Products (Fan-Induced Method)

Reason:

- This proposal separates "metal roof shingles" as a separate line item product in Section 1504, specifically under the non-ballasted roof systems provisions.
- This proposal also modifies the title of Table 1504.1.1 only as the table is appropriate to steep slope roofs other than asphalt shingles based on the modifications that first appeared in the 2013 Edition of ASTM Standard D3161.
- This proposal further modifies the ASTM standard to the 2015 edition.

This proposal would create a separate line item for metal roof shingles based on the fact that metal shingles are not the same in all respects as either asphalt shingles (Section 1504.1.1) or the other roof systems (Section 1504.3.1) provisions.

One of the major considerations for this product type is the wind uplift testing which is addressed by several industry standards including FM, UL, and ASTM. The majority of manufacturers use one or more of these standards and we propose that the choice should remain with the manufacturer to demonstrate compliance.

ASTM D3161 is a fan-induced wind test that was originally developed for asphalt shingles. The most recent version, ASTM D3161M-15, is no longer constrained to asphalt shingles, but expanded to evaluate wind resistance of discontinuous, air permeable, steep slope roofing products that results from the product's rigidity, with or without contribution from sealant or other adhesive to help hold down the leading edge of the tabs, or mechanical interlocking, with or without contribution from sealant or other adhesive to hold down the leading edge of the tab, or any combination thereof." This would clearly include metal shingles which are specifically identified in Scope Section 1.3.

Inclusion of this standard as a compliance path for metal shingles would alleviate many of the difficulties experienced by metal shingle manufacturers when required by current code language to conduct either UL 1897 or UL 580 in a non-air-permeable fashion that does not fairly represent this product class. Underwriter's Laboratories

has provided metal shingle wind classifications using earlier versions of ASTM D3161 for many years, and currently has D3161-related metal shingles in their Online Classification Directory and UL was also a proponent of the scope change to D3161. These points illustrate UL's acceptance of D3161 as a viable means to demonstrate wind resistance of metal shingles.

The scope of ASTM D3161M-15 is very clear in Section 1.3 where it states "This test method was formerly titled "Wind Resistance of Asphalt Shingles (Fan-Induced Method)" but was revised to acknowledge that the method is applicable to many other steep slope roofing products and has been used to evaluate the wind resistance of those products for many years by several testing and certification laboratories."

Cost Impact: Will not increase the cost of construction
This proposal would allow an alternate testing method which should add no cost to construction.

S14-16 : TABLE 1504.3.3 (NEW)-WILLIAMS11537

S15-16

IBC: 1504.5.

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~~
<u>In hurricane prone regions as defined in Section 202, low-slope</u> 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.

S15-16 : 1504.5-EDWARDS11848

S16-16

IBC: 1504.5.1 (New).

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 w ind 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 w ind events. Examples of these observations are show n below . SPRI developed the gutter test standard to address this issue. The w ind resistance tests included in this standard measure the resistance of the gutter system to w ind forces acting outw ardly (aw ay from the building) and to w ind forces acting upw ardly tending to lift the gutter off of the building. The standard also measures the resistance of the gutter system to static forces of w ater, snow and ice acting dow nw ard. Follow ing are examples of gutter failures during high w ind 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 2

Figure 3

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 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.

S16-16 : 1504.5.1 (NEW)-ENNIS11609

S17-16

IBC: 1504.7.

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~~ accordance with procedures adapted from UL2218.

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
UL 2218 - 2010 Impact Resistance of Prepared Roof Covering Materials, with revisions through May 1, 2012.

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.

S17-16 : 1504.7-ENNIS13287

S18-16

IBC: 1504.7, 1507.11.2, 1507.12.2, 1507.13.2; IRC: R905.11.2, R905.12.2, R905.13.2.

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.

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

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~~.

2015 International Residential Code

TABLE R905.11.2
MODIFIED BITUMEN ROOFING MATERIAL STANDARDS

MATERIAL	STANDARD
Acrylic coating	ASTM D 6083
Asphalt adhesive	ASTM D 3747
Asphalt cement	ASTM D 3019
Asphalt coating	ASTM D 1227; D 2824
Asphalt primer	ASTM D 41
Modified bitumen roof membrane	ASTM D 6162; D 6163; D 6164; D 6222; D 6223; D 6298~~;~~ ~~CGSB 37-GP-56M~~

R905.12.2 Material standards. Thermoset single-ply roof coverings shall comply with ASTM D 4637~~,~~ 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 w ithdraw n Canadian standards.

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

S18-16 : 1504.7-FISCHER13510

S19-16

IBC: 1504.4, 1504.8.

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. ~~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
~~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)[e] For Aggregate Surfaced Roof Coverings[a,b]

~~NOMINAL DESIGN WIND SPEED, V_{asd} (mph)[b, d]~~	~~MAXIMUM MEAN ROOF HEIGHT (ft)[a, c]~~		
	~~Exposure category~~		
	~~B~~	~~C~~	~~D~~
~~85~~	~~170~~	~~60~~	~~30~~
~~90~~	~~110~~	~~35~~	~~15~~
~~95~~	~~75~~	~~20~~	~~NP~~
~~100~~	~~55~~	~~15~~	~~NP~~
~~105~~	~~40~~	~~NP~~	~~NP~~
~~110~~	~~30~~	~~NP~~	~~NP~~
~~115~~	~~20~~	~~NP~~	~~NP~~

~~120~~	~~15~~	~~NP~~	~~NP~~
~~Greater than 120~~	~~NP~~	~~NP~~	~~NP~~

_	_	WIND EXPOSURE AND NOMINAL DESIGN WIND SPEED$_{asd}$ (MPH)[c,d]														
ASTM Gradation	Mean Roof Height[a] (ft)	Exposure B					Exposure C					Exposure D				
_		85	90	100	110	120	85	90	100	110	120	85	90	100	110	120
ASTM	15	0	0	15	20	25	22	25	31	38	45	27	31	38	45	53
D1863	20	0	12	17	23	25	23	27	33	40	47	29	32	40	47	55
No. 7	30	13	15	21	27	32	26	29	36	44	51	31	35	43	50	58
or No. 67	40	15	18	24	29	35	28	31	39	46	53	33	37	45	52	60
_	50	17	20	26	32	38	29	33	40	48	55	34	38	46	54	62
ASTM	60	18	21	28	34	40	30	34	42	49	57	35	39	47	56	64
D7655	80	21	24	30	37	43	32	36	44	52	60	37	41	49	58	66
No. 4	100	23	26	33	40	46	34	38	46	54	62	38	43	51	60	68
_	125	25	28	35	42	49	36	40	48	56	64	40	44	53	62	70
_	150	27	30	37	45	52	37	41	50	58	66	41	45	54	63	72
ASTM	15	0	0	11	15	20	16	19	25	31	37	22	25	31	38	45
D1863	20	0	0	13	17	22	18	21	27	34	40	23	26	33	40	47
No. 6	30	0	11	16	21	26	20	24	30	36	43	25	29	36	43	50
_	40	0	13	18	24	29	22	25	32	39	45	27	30	37	45	52
_	50	12	15	20	26	31	23	27	34	40	47	28	31	39	46	53
_	60	13	16	22	28	33	24	28	35	42	49	29	33	40	47	55
_	80	16	19	25	30	36	26	30	37	44	51	30	34	42	50	57
_	100	18	21	27	33	39	28	31	39	46	53	32	36	43	51	59
_	125	19	23	29	35	42	29	33	40	48	55	33	37	45	53	61
_	150	21	24	31	37	44	30	34	42	50	57	34	38	46	54	62

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: 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:
Crandell, J. H. and Smith, T.L.. (2010) Design Method Improvements to Prevent Roof Aggregate Blow-Off, Hugo Conference

International Building Code. Falls Church, VA

Kind, R.J. and Wardlaw R.L. (1976). Design of Rooftops Against Gravel Blow-Off. National Aeronautical Establishment, National Research Council, Canada. Kind, R.J. (1977). Further Wind Tunnel Tests on Building Models to Measure Wind Speeds at Which Gravel is Blown Off Rooftops. LTR-LA-189. National Aeronautical Establishment, National Research Council, Canada.

Smith, T.L. (June 1997). Aggregate Blow-Off from BUR and SPF Roofs: Recognizing the Potential Hazards and Avoiding Problems. Proceedings of The 8th U.S. Conference on Wind Engineering, AAWE. SPRI (2008). Wind Design Standard for Ballasted Single-Ply Roofing Systems.

Crandell, J. H. and Fischer, M. (2010) Winds of Change: Resolving Roof Aggregate Blow-Off, RCI Conference

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.

S19-16 : 1504.8-ENNIS11713

S20-16

IBC: 1504.8.

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

2015 International Building Code

Revise as follows:

1504.8 Aggregate. ~~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 w hen 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.

S20-16 : 1504.8-FISCHER13529

S21-16

IBC: 1504.8.

Proponent : Edward Kulik, representing Building Code Action Committee (bcac@iccsafe.org)

2015 International Building Code

Revise as follows:

1504.8 ~~Aggregate~~ Surfacing and ballast materials in hurricane-prone regions. ~~Aggregate used as surfacing for roof coverings and aggregate, gravel or stone used as ballast shall not be used on the roof of~~
For 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~~.~~ , the following materials shall not be used on the roof:

1. Aggregate used as surfacing for roof coverings
2. Aggregate, gravel or stone used as ballast

Reason: This proposal removes a run on sentence to allow for more consistent interpretation and enforcement. This proposal is submitted by the ICC Building Code Action Committee (BCAC). 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**

Cost Impact: Will not increase the cost of construction
No increase to the cost of construction as this is a clarification of current requirements.

S21-16 : 1504.8-KULIK10960

S22-16

IBC: 1504.9 (New).

Proponent : Edward Kulik, representing Building Code Action Committee (bcac@iccsafe.org); Marc Levitan, National Institute of Standards and Technology (NIST), representing National Institute of Standards and Technology (marc.levitan@nist.gov)

2015 International Building Code

Add new text as follows:

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.

Reason: Investigations of building performance following tornadoes have shown that loose aggregate, gravel and stone surfacing and ballast on roofs are significant contributors to building damage and injuries. In particular – Risk Category III and IV buildings such as schools and hospitals have often experienced significant glazing damage due to aggregate blow-off from their own roofs, and/or roofs of nearby buildings on their own campuses, in essence 'self-inflicted' damage. FEMA has also documented instances where people have been injured after being struck directly by roof aggregate in tornadoes, in Illinois (FEMA 2010) and Texas (FEMA 2007).
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. 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.

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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 tow er at SJRMC follow ing the Joplin tornado; interior damage at SJRMC. Note the extensive amount of roof aggregate inside the building.

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).

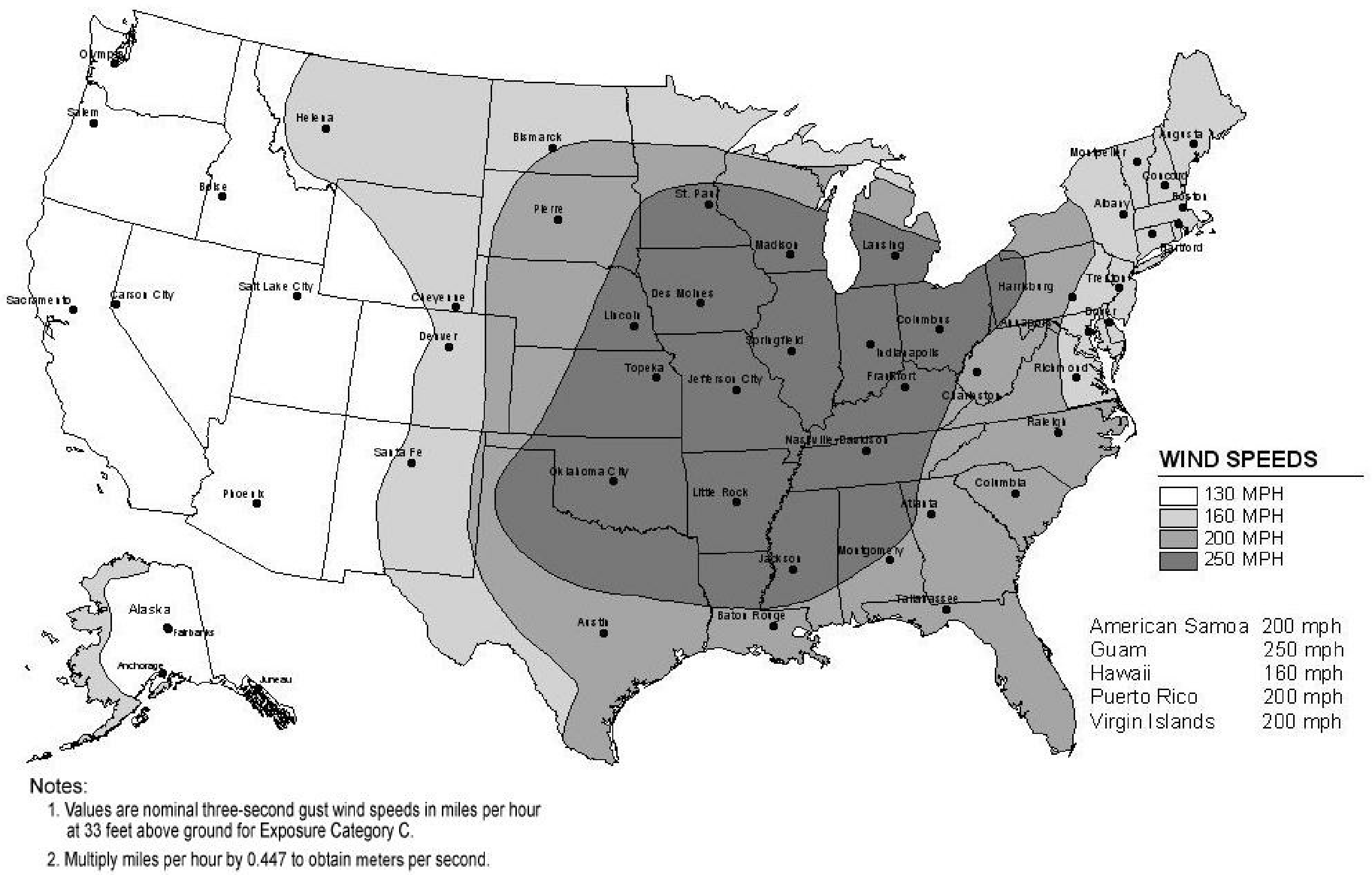

Figure 4 - Shelter Design Wind Speeds for Tornadoes - Source ICC 500-2014, International Code Council.

The ICC Building Code Action Committee (BCAC) is a co-proponent of this proposal. BCAC w as 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 w ere numerous Working Group meetings and conference calls for the current code development cycle, w hich included members of the committee as w ell as any interested party to discuss and debate the proposed changes. Related documentation and reports are posted on the BCAC w ebsite at: **BCAC**

Bibliography: References:

Crandell, J. H. and Smith, T.L. (2009), Design Method Improvements to Prevent Roof Aggregate Blow -off, P*roceedings of Hurricane Hugo 20th anniversary symposium on building safer communities – improving disaster resilience*, Applied Technology Council.

FEMA (2005). *Mitigation Assessment Team Report: Hurricane Charley in Florida.* FEMA 488. Federal Emergency Management Agency. April 2005. Available at https://w w w .fema.gov/media-library/assets/documents/905.

FEMA (2007). *Design Guide for Improving Hospital Safety in Earthquakes, Floods, and High Winds:Providing Protection to People and Buildings*. FEMA 577. Federal Emergency Management Agency. June 2007 (see p. 4-123, and 4-126 to 4-140). Available at https://w w w .fema.gov/media-library/assets/documents/10672.

FEMA (2010). *Design Guide for Improving School Safety in Earthquakes, Floods, and High Winds.* FEMA P-424. Federal Emergency Management Agency. December 2010 (see p. 6-23). Available at http://w w w .fema.gov/media-library/assets/documents/5264?id=1986.

FEMA (2012). *Mitigation Assessment Team Report: Spring 2011 Tornadoes: April 25-28 and May 22 Building Performance Observations, Recommendations, and Technical Guidance.* FEMA P-908. Federal Emergency Management Agency. May 2012 (see Tornado Recovery Advisory 6, Appendix F, p. 5). Available at http://www.fema.gov/media-library/assets/documents/25810.

NIST (2006). *Performance of Physical Structures in Hurricane Katrina and Hurricane Rita: A Reconnaissance Report*. NIST Technical Note 1476. National Institute of Standards and Technology June 2006. Available at http://www.nist.gov/customcf/get_pdf.cfm?pub_id=908281.

NIST (2014). *Final Report, National Institute of Standards and Technology (NIST) Technical Investigation of the May 22, 2011, Tornado in Joplin, Missouri*. NIST NCSTAR-3. March 2014. Available at http://dx.doi.org/10.6028/NIST.NCSTAR.3.

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.

S22-16 : 1504.9 (NEW)-KULIK10961

S23-16

IBC: , 1504.9 (New), 202 (New).

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.

S23-16 : 1504.9 (NEW)-ROBERTS4049

S24-16

IBC: 1504.9 (New).

Proponent : T. Eric Stafford, PE, representing AECOM; Andrew Herseth, representing Federal Emergency Management Agency (andrew.herseth@fema.dhs.gov)

2015 International Building Code

Add new text as follows:

1504.9 Wind resistance of gutters and leaders Gutters on the exterior of buildings shall be attached to resist the following loads:

1. Lateral loads using the component and cladding loads for walls in accordance with ASCE 7.
2. Vertical loads using the component and cladding loads for roof overhangs in accordance with ASCE 7.
   Attachment of gutters shall be determined using a design analysis or testing in accordance with Test Methods G-1 and G-2 of ANSI/SPRI GD-1.

Leaders on the exterior of buildings shall be attached to resist lateral loads using the component and cladding loads for walls in accordance with ASCE 7.

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
SPRI
ANSI/SPRI GD-1-2010 Structural Design Standard for Gutter Systems Use with Low-Slope Roofs

Reason: This code change is primarily providing more specific guidance for w hat is already required by the code. Section 1609.1 clearly states that buildings, structures, and parts thereof shall be designed to resist the minimum w ind loads prescribed in Section 1609. How ever, the code nor ASCE 7 provide specific guidance on the design loads for gutters and leaders. ANSI/SPRI GD-1 Structural Design Standard for Gutter Systems Used w ith Low -Slope Roofs does provide guidance for the design, attachment, and testing for gutters and leaders. This standard w as proposed for inclusion in the 2012 IBC as code change S15-12. It w as disapproved by the Structural Committee primarily because the w ind provisions in the standard are based on ASCE 7-05. ASCE 7 w as being updated to the 2010 edition during that code change cycle. Instead of simply referencing ASCE 7, GD-1 reproduces many of the provisions in ASCE 7 such Exposure Categories and the w ind Importance Factor. This reproduction of content makes it problematic to reference in the IBC since the Exposure Categories have changed and ASCE 7-10 doesn't require the use of a w ind Importance Factor. Its use may result in confusion w hen using ASCE 7-10.
While GD-1 is limited to low -slope roof systems, the design load methodology in GD-1 w ould technically apply to any roof system (low -slope or high-slope). From a w ind loading standpoint, the only reason GD-1 is limited to low -slope roofs is because the design criteria in GD-1 is based on the GCp values in ASCE 7 for roof slopes less than 7^0 and for mean roof heights ≤ 60 ft. The language proposed in this code change is consistent w ith the technical design loads for gutters in GD-1. The proposed language requires gutters to be attached to resist uplift loads using the roof overhang loads from ASCE 7 as gutters w ill feel the effects of w ind on the upper and low er surfaces. The GCp values in ASCE 7 for roof overhang state that the values include contributions from both upper and low er surfaces. Additionally, gutters are installed on vertical faces such as w alls or fascia boards w hich w ould necessitate they be designed for lateral loads using the component and cladding loads for w alls in ASCE 7.

GD-1 also contains a test method for determining the w ind resistance of gutter system. The proposal allow s gutter systems to be tested in accordance w ith Test Methods G-1 and G-2 as an alternative to designing the system. Only the test method is referenced. The rest of GD-1 w ould not be applicable.

Numerous studies of the aftermath of hurricanes have show n the need for better attachment of gutters. The Hurricane Charley (FEMA 488), Hurricane Ivan (FEMA 489), and Hurricane Katrina (FEMA 549) Mitigation Assessment Team (MAT) reports specifically recommend that design criteria for gutters and dow nspouts be added to the codes. See Sections 8.2.2, 8.5, and 8.7 of FEMA 488. See Tables 8-6, 8-7, and 8-9 of FEMA 489. See

Tables 11-6, 11-7, and 11-9 of FEMA 549. The Hurricane Charley MAT observed numerous failures to gutter systems on roofs and some were of sufficient mass to be very damaging missiles (See Figure 1). Additionally, the Hurricane Charley MAT observed damage to a low-sloped roof where wind lifted the gutter and metal edge flashing and peeled the modified bitumen membrane. The roof systems observed by the Hurricane Ivan MAT noted that membrane damage was typically caused by windborne debris punctures and tears, and by membrane lifting and peeling after lifting of either the gutter, edge flashing, or coping The Hurricane Katrina MAT observed several types of low-slope roof systems that included built-up roofs (BURs), modified bitumen, and single-ply where damage was typically caused by membrane lifting and peeling after lifting of the gutter, edge flashing, or coping, and by puncturing and tearing by windborne debris. Gutters and leaders that fail can become wind-borne debris and break unprotected glazing in addition to puncturing other areas of the building envelope such as the walls and roof. Additionally, when gutters lift up and initiate roof failure, water can enter the building.

The gutters lying on the lower roof in Figure 1 came from the right side of the upper roof and may have caused some of the glass breakage. However, the majority of the damage was broken by flying aggregate from the hospital's BURs.

Additionally, FEMA 424 Design Guide for Improving School Safety in Earthquakes, Floods, and High Winds (2010) recommends special attention be given to attaching gutters to prevent uplift and recommends uplift loads on gutters be calculated using the overhang coefficients in ASCE 7. The following is an excerpt from FEMA 424 regarding gutters and downspouts:

Gutters and downspouts

Gutters and Downspouts Storm-damage research has shown that gutters are seldom designed and constructed to resist wind loads. At the school shown in Figure 6-74, the gutter brackets were attached with a fastener near the top and bottom of the bracket. Hence, the fasteners prevented the brackets from rotating out from the wall. However, because the gutter was not attached to the brackets, the gutter blew away. When a gutter lifts, it typically causes the edge flashing that laps into the gutter to lift as well. Frequently, this results in a progressive lifting and peeling of the roof membrane. The membrane blow-off shown in Figure 6-75 was initiated by gutter uplift. The gutter was similar to that shown in Figure 6-74. The membrane blowoff caused significant interior water damage.

Figure 6-78: Blow-off of this downspout resulted in glazing breakage. Estimated wind speed: 105–115 mph. Hurricane Ivan (Florida, 2004)

Figure 6-74: Because this gutter was not attached to the bracket, wind lifted the gutter along with the metal edge flashing that lapped into the gutter. Bracket fasteners are indicated by the red arrows. Hurricane Francis (Florida, 2004)

Figure 6-75: The original modified bitumen membrane was blown away after the gutter lifted in the area shown by the red arrow (the black membrane is a temporary roof). Hurricane Francis (Florida, 2004)

Special design attention needs to be given to attaching gutters to prevent uplift, particularly for those in excess of 6 inches in w idth. Currently, there are no design guides or standards pertaining to gutter w ind resistance. It is recommended that the designer calculate the uplift load on gutters using the overhang coefficient from ASCE 7. There are tw o approaches to resist gutter uplift.

 Gravity-support brackets can be designed to resist uplift loads. In these cases, in addition to being attached at its top, the bracket should also be attached at its low end to the w all (as w as the case for the brackets show n in Figure 6-74). The gutter also needs to be designed so it is attached securely to the bracket in a w ay that w ill effectively transfer the gutter uplift load to the bracket (see Figure 6-76). Bracket spacing w ill depend on the gravity and uplift load, the bracket's strength, and the strength of connections betw een the gutter/bracket and the bracket/w all. With this option, the bracket's top w ill typically be attached to a w ood nailer, and that fastener w ill be designed to carry the gravity load. The bracket's low er connection w ill resist the rotational force induced by gutter uplift. Because brackets are usually spaced close together to carry the gravity load, developing adequate connection strength at the low er fastener is generally not difficult. Screw s rather than nails are recommended to attach brackets to the building because screw s are more resistant than nails to dynamically induced pull-out forces.

 The other option is to use gravity-support brackets only to resist gravity loads, and use separate sheet-metal straps at 45-degree angles to the w all to resist uplift loads (Figure 6-77). Strap spacing w ill depend on the gutter uplift load and strength of the connections betw een the gutter/strap and the strap/w all. Note that FMG Data Sheet 1-49 recommends placing straps 10 feet apart. How ever, at that spacing w ith w ide gutters, fastener loads induced by uplift are quite high. When straps are spaced at 10 feet, it can be difficult to achieve sufficiently strong uplift

connections.

Storm damage research has also shown that downspouts are seldom designed and constructed to resist wind loads (see Figure 6-78). Special design attention needs to be given to attaching downspouts to prevent blow-off. Currently there are no design guides or standards pertaining to downspout wind resistance. The keys to achieving successful performance include providing brackets that are not excessively spaced, bracket strength, and the strength of the connections between the brackets and wall.

Bibliography: FEMA 488, Mitigation Assessment Team Report: Hurricane Charley in Florida (2005)
FEMA 489, Hurricane Ivan in Alabama and Florida: Observations, Recommendations, and Technical Guidance (2005)

FEMA 549, Hurricane Katrina in the Gulf Coast: Mitigation Assessment Team Report, Building Performance Observations, Recommendation, and Technical Guidance (2006)

Cost Impact: Will increase the cost of construction
May result in an increase in cost of construction. Since the codes and design standards are not specific about design wind loads for gutters and leaders, it's unclear what criteria designers are actually using to attach gutters and leaders. However, any initial minimal up front construction costs will result in reduced owner residual risk through improved resilience to high wind loading, reduced wind driven rain associated damages and more than offset costs through mitigating already well documented failure modes and vulnerabilities.

Analysis: A review of the standard(s) proposed for inclusion in the code, ANSI/SPRI GD-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.

S24-16 : 1504.9 (NEW)-STAFFORD11712

S25-16

Part I:

IBC: 202, 1507.16, 1607.12.3.1, [BF] 1505.10, [BF] 1507.16.1.

Part II:

IFC: 202 (New), 317, 317.1, 317.2, 905.3.8.

Part III:

IECC: C202 (New), C402.3.

THIS IS A 3 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC-STRUCTURAL CODE COMMITTEE. PART II WILL BE HEARD BY THE FIRE CODE COMMITTEE. PART III WILL BE HEARD BY THE IECC-COMMERCIAL CODE COMMITTEE. SEE THE TENTATIVE HEARING ORDER FOR THESE COMMITTEES.

Proponent : Tim Earl, representing GBH International (tearl@gbhinternational.com)

Part I

2015 International Building Code

Revise as follows:

SECTION 202 DEFINITIONS

VEGETATIVE ROOF. An assembly of interacting components designed to waterproof ~~and normally insulate~~ a building's top surface that includes, by design, vegetation and related landscape elements.

[BF] 1505.10 ~~Roof gardens and landscaped~~ Vegetative roofs. ~~Roof gardens and landscaped~~ Vegetative roofs shall comply with Section 1507.16 and shall be installed in accordance with ANSI/SPRI VF-1.

1507.16 Vegetative roofs~~, roof gardens and landscaped roofs~~. *Vegetative roofs*~~, roof gardens and landscaped roofs~~ shall comply with the requirements of this chapter, Sections 1607.12.3 and 1607.12.3.1 and the *International Fire Code*.

[BF] 1507.16.1 Structural fire resistance. The structural frame and roof construction supporting the load imposed upon the roof by the *vegetative roof*~~, roof gardens or landscaped roofs~~ shall comply with the requirements of Table 601.

1607.12.3.1 Vegetative ~~and landscaped~~ roofs. *No change to text.*

Part II

2015 International Fire Code

Add new definition as follows:

SECTION 202 DEFINITIONS

VEGETATIVE ROOF. An assembly of interacting components designed to waterproof a building's top surface that includes, by design, vegetation and related landscape elements.

Revise as follows:

SECTION 317 ~~ROOFTOP GARDENS AND LANDSCAPED~~ VEGETATIVE ROOFS

317.1 General. ~~Rooftop gardens and landscaped~~ Vegetative roofs shall be installed and maintained in accordance with Sections 317.2 through 317.5 and Sections 1505 and 1507.16 of the *International Building Code.*

317.2 ~~Rooftop garden or landscaped~~ Vegetative roof size. ~~Rooftop garden or landscaped~~ Vegetative roof areas shall not exceed 15,625 square feet (1450 m^2) in size for any single area with a maximum dimension of 125 feet (39 m) in length or width. A minimum 6-foot-wide (1.8 m) clearance consisting of a Class A-rated roof system complying with ASTM E 108 or UL 790 shall be provided between adjacent ~~rooftop gardens or landscaped~~ vegetative roof areas.

905.3.8 ~~Rooftop gardens and landscaped~~ Vegetative roofs. Buildings or structures that have ~~rooftop gardens or landscaped~~ vegetative roofs and that are equipped with a standpipe system shall have the standpipe system extended to the roof level on which the ~~rooftop garden or landscaped~~ vegetative roof is located.

Part III

2015 International Energy Conservation Code

Add new definition as follows:

SECTION C202 VEGETATIVE ROOF.
An assembly of interacting components designed to waterproof a building's top surface that includes, by design, vegetation and related landscape elements.

Revise as follows:

C402.3 Roof solar reflectance and thermal emittance. Low-sloped roofs directly above cooled conditioned spaces in *Climate Zones* 1, 2 and 3 shall comply with one or more of the options in Table C402.3.

Exceptions: The following roofs and portions of roofs are exempt from the requirements of Table C402.3:

1. Portions of the roof that include or are covered by the following:
   - 1.1. Photovoltaic systems or components.
   - 1.2. Solar air or water-heating systems or components.
   - 1.3. ~~Roof gardens or landscaped~~ Vegetative roofs.
   - 1.4. Above-roof decks or walkways.
   - 1.5. Skylights.
   - 1.6. HVAC systems and components, and other opaque objects mounted above the roof.
2. Portions of the roof shaded during the peak sun angle on the summer solstice by permanent features of the building or by permanent features of adjacent buildings.
3. Portions of roofs that are ballasted with a minimum stone ballast of 17 pounds per square foot [74 kg/m^2] or 23 psf [117 kg/m^2] pavers.
4. Roofs where not less than 75 percent of the roof area complies with one or more of the exceptions to this section.

Reason: "Vegetative roof" is the accepted term used throughout codes and standards, not "landscaped roof" or "roof garden." "Vegetative roof" is defined in Chapter 2, and there is no definition for "landscaped roof" or "roof garden." This proposal replaces all instances of "lanscaped roof" with "vegetative roof." It also eliminates the undefined term "roof garden."
Finally, this proposal modifies the IBC definition of "vegetative roof" to match the definition approved by ASTM

Committee D08 on Roofing and Waterproofing. It also adds this definition to the IECC and the IFC.

Cost Impact: Will not increase the cost of construction
This is simple clarification so there will be no changes to construction requirements.

S25-16 : [BF]1505.10-EARL11838

S26-16

IBC: 1506.1.

Proponent : Jason Wilen AIA CDT RRO, National Roofing Contractors Association (NRCA), representing National Roofing Contractors Association (NRCA) (jwilen@nrca.net)

2015 International Building Code

Revise as follows:

1506.1 Scope. The requirements set forth in this section shall apply to the application of roof-covering materials specified herein. Roof coverings shall be applied in accordance with this chapter and the manufacturer's _approved_installation instructions. Installation of roof coverings shall comply with the applicable provisions of Section 1507.

Reason: This proposal is an editorial change to clarify that it is the "installation instructions" that are approved. This change w ill make Section 1506.1 consistent w ith Section 1507.

Cost Impact: Will not increase the cost of construction
The proposed change is a clarification and does not change the stringency of existing code requirements so the cost construction w ill be unchanged.

S26-16 : 1506.1-WILEN AIA CDT RRO12363

S27-16

1507.1.1 (New), 1507.1.2 (New), 1507.2.3, 1507.3.3, 1507.4.5, 1507.5.3, 1507.5.4, 1507.6.3, 1507.6.4, 1507.7.3, 1507.7.4, 1507.8, 1507.9.3, 1507.9.4, 1507.17.3, 1507.17.4

Proponent : T. Eric Stafford, PE, representing Institute for Business and Home Safety

2015 International Building Code

Add new text as follows:

1507.1.1 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 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 1507.1.1(1). Underlayment shall be applied in accordance with Table 1507.1.1(2). Underlayment shall be attached in accordance with Table 1507.1.1(3).

Exceptions:

1. As an alternative, self-adhering polymer modified bitumen underlayment complying with ASTM D 1970 installed in accordance with the manufacturer's installation 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 strip of self-adhering polymer modified bitumen membrane complying with ASTM D 1970 installed in accordance with the manufacturer's installation instructions for the deck material shall be applied over all joints in the roof decking. An approved underlayment for the applicable roof covering for design wind speeds less than 120 mph shall be applied over the 4-inch wide membrane strips.
3. As an alternative, two layers of underlayment complying with ASTM D 226 Type II or ASTM D 4869 Type IV shall be permitted to be installed as follows: Apply a 19-inch strip of underlayment parallel with the eave. Starting at the eave, apply 36-inch-wide strips of underlayment felt, overlapping successive sheets 19 inches. The underlayment shall be attached with corrosion-resistant fasteners in a grid pattern of 12 inches between side laps with a 6- inch spacing at side and end laps. End laps shall be 4 inches and shall be offset by 6 feet (1829 mm). Underlayment shall be attached using metal or plastic cap nails with a nominal cap diameter of not less than 1 inch. Metal caps shall have a thickness of not less than 32-gage sheet metal. Power-driven metal caps shall have a thickness of not less than 0.010 inch. Thickness of the outside edge of plastic caps shall be not less than 0.035 inch. The cap nail shank shall be not less than 0.083 inch for ring shank cap nails and 0.091 inch for smooth shank cap nails. Cap nail shank shall have a length sufficient to penetrate through the roof sheathing or not less than 3/4 inch into the roof sheathing.

1507.1.2 Ice barriers. In areas where there has been a history of ice forming along the eaves causing a backup of water, an ice barrier shall be installed for asphalt shingles, metal roof shingles, mineral-surfaced roll roofing, slate and slate-type shingles, wood shingles, and wood shakes. The ice barrier shall consists of not less than two layers of underlayment cemented together or a self-adhering polymer modified bitumen sheet shall be used in place of normal

underlayment and extend from the lowest edges of all roof surfaces to a point not less than 24 inches (610 mm) inside the exterior wall line of the building.

Exception: Detached accessory structures that contain no conditioned floor area.

TABLE 1507.1.1(1)
Underlayment Types

Roof Covering	Section	Maximum Ultimate Design Wind Speed, V_{ult} < 140 mph	Maximum Ultimate Design Wind Speed, Vult ≥ 140 mph
Asphalt Shingles	1507.2	ASTM D 226 Type I or II ASTM D 4869 Type I, II, III, or IV ASTM D 6757	ASTM D 226 Type II ASTM D 4869 Type IV ASTM D 6757
Clay and concrete tiles	1507.3	ASTM D 226 Type II ASTM D 2626 Type I ASTM D 6380 Class M mineral surfaced roll roofing	ASTM D 226 Type II ASTM D 2626 Type I ASTM D 6380 Class M mineral surfaced roll roofing
Metal panels	1507.4	Manufacturer's instructions	ASTM D 226 Type II ASTM D 4869 Type IV
Metal roof shingles	1507.5	ASTM D 226 Type I or II ASTM D 4869 Type I, II, III, or IV	ASTM D 226 Type II ASTM D 4869 Type IV
Mineral-surfaced roll roofing	1507.6	ASTM D 226 Type I or II ASTM D 4869 Type I, II, III, or IV	ASTM D 226 Type II ASTM D 4869 Type IV
Slate shingles	1507.7	ASTM D 226 Type II ASTM D 4869 Type III or IV	ASTM D 226 Type II ASTM D 4869 Type IV
Wood shingles	1507.8	ASTM D 226 Type I or II ASTM D 4869 Type I, II, III, or IV	ASTM D 226 Type II ASTM D 4869 Type IV
Wood shakes	1507.9	ASTM D 226 Type I or II ASTM D 4869 Type I, II, III, or IV	ASTM D 226 Type II ASTM D 4869 Type IV
Photovoltaic shingles	1507.17	ASTM D 226 Type I or II ASTM D 4869 Type I, II, III, or IV ASTM D 6757	ASTM D 226 Type II ASTM D 4869 Type IV ASTM D 6757

TABLE 1507.1.1(2)
Underlayment Application

Roof Covering	Section	Maximum Ultimate Design Wind Speed, Vult < 140 mph	Maximum Ultimate Design Wind Speed, Vult ≥ 140 mph
Asphalt shingles	1507.2	For roof slopes from two units vertical in 12 units horizontal (2:12), up to four units vertical in 12 units horizontal (4:12), underlayment shall be two layers applied in the following manner. Apply a 19-inch (483 mm) strip of underlayment felt parallel to and starting at the eaves, Starting at the eave, apply 36-inch-wide (914 mm) sheets of underlayment, overlapping successive sheets 19 inches (483 mm). End laps shall be 4 inches and shall be offset by 6 feet (1829 mm). Distortions in the underlayment shall not interfere with the ability of the shingles to seal. For roof slopes of four units vertical in 12 units horizontal (4:12) or greater, underlayment shall be one layer applied in the following manner. Underlayment shall be applied shingle fashion, parallel to and starting from the eave and lapped 2 inches (51 mm), Distortions in the underlayment shall not interfere with the ability of the shingles to seal. End laps shall be 4 inches and shall be offset by 6 feet (1829 mm).	Same as Maximum Ultimate Design Wind Speed, V_{ult} < 140 mph except all laps shall be a minimum of 4 inches
Clay and concrete tile	1507.3	For roof slopes from two and one-half units vertical in 12 units horizontal (2 1/2:12), up to four units vertical in 12 units horizontal (4:12), underlayment shall be a minimum of two layers underlayment applied as follows. Starting at the eave, apply a 19-inch (483 mm) strip of underlayment shall be applied parallel with the eave. Starting at the eave, apply a36-inch-wide (914 mm) strips of underlayment felt	Same as Maximum Ultimate Design Wind Speed, V_{ult} < 140 mph except all laps shall be a minimum of 4 inches

		shall be applied, overlapping successive sheets 19 inches (483 mm). End laps shall be 4 inches and shall be offset by 6 feet (1829 mm). For roof slopes of four units vertical in 12 units horizontal (4:12) or greater, underlayment shall be one layer applied in the following manner. Underlayment shall be applied shingle fashion, parallel to and starting from the eave and lapped 2 inches (51 mm), End laps shall be 4 inches and shall be offset by 6 feet (1829 mm).	
Metal roof panels	1507.4	Apply in accordance with the manufacturer's installation instructions	For roof slopes from two units vertical in 12 units horizontal (2:12), up to four units vertical in 12 units horizontal (4:12), underlayment shall be two layers applied in the following manner. Apply a 19-inch (483 mm) strip of underlayment felt parallel to and starting at the eaves. Starting at the eave, apply 36-inch-wide (914 mm) sheets of underlayment, overlapping successive sheets 19 inches (483 mm). End laps shall be 4 inches and shall be offset by 6 feet (1829 mm). For roof slopes of four units vertical in 12 units horizontal (4:12) or greater, underlayment shall be one layer applied in the following manner. Underlayment shall be applied shingle fashion, parallel to and starting from the eave and lapped 4 inches (51 mm), End laps shall be 4 inches and shall be offset by 6 feet (1829 mm).
Metal roof shingles	1507.5		
Mineral-surfaced roll roofing	1507.6		
Slate shingles	1507.7		
Wood shakes	1507.8		
Wood shingles	1507.9		
Photovoltaic shingles	1507.17	For roof slopes from three units vertical in 12 units horizontal (3:12), up to four units vertical in 12 units horizontal (4:12), underlayment shall be two layers applied in the following manner. Apply a 19-inch (483 mm) strip of underlayment felt parallel to and	Same as Maximum Ultimate Design Wind Speed, V_{ult} < 140 mph except all laps shall be a minimum of 4 inches

		starting at the eaves, Starting at the eave, apply 36-inch-wide (914 mm) sheets of underlayment, overlapping successive sheets 19 inches (483 mm). End laps shall be 4 inches and shall be offset by 6 feet (1829 mm). Distortions in the underlayment shall not interfere with the ability of the shingles to seal. For roof slopes of four units vertical in 12 units horizontal (4:12) or greater, underlayment shall be one layer applied in the following manner. Underlayment shall be applied shingle fashion, parallel to and starting from the eave and lapped 2 inches (51 mm), Distortions in the underlayment shall not interfere with the ability of the shingles to seal. End laps shall be 4 inches and shall be offset by 6 feet (1829 mm).	

TABLE 1507.1.1(3)
Underlayment Attachment

Roof Covering	**Section**	**Maximum Ultimate Desing Wind Speed, Vult < 140 mph**	**Maximum Ultimate Desing Wind Speed, Vult ≥ 140 mph**
Asphalt shingles	1507.2	Fastened sufficiently to hold in place	The underlayment shall be attached with corrosion-resistant fasteners in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at side and end laps. Underlayment shall be attached using metal or plastic cap nails or cap staples with a nominal cap diameter of not less than 1 inch. Metal caps shall have a thickness of not less than 32-gage sheet metal. Power-driven metal caps shall have a minimum thickness of 0.010 inch. Minimum thickness of the outside edge of plastic caps shall be 0.035 inch. The cap nail shank shall be not less than 0.083 inch for ring shank cap nails and 0.091 inch for smooth shank cap nails. Staples shall be not less than 21 gage. Cap nail shank and cap staple legs shall have a length sufficient to penetrate through the roof sheathing or not less than 3/4 inch into the roof sheathing.
Clay and concrete tile	1507.3		
Photovoltaic shingles	1507.17		

Metal roof panels	1507.4	Manufacturer's installation instructions	The underlayment shall be attached with corrosion-resistant fasteners in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at side and end laps. Underlayment shall be attached using metal or plastic cap nails or cap staples with a nominal cap diameter of not less than 1 inch. Metal caps shall have a thickness of not less than 32-gage sheet metal. Power-driven metal caps shall have a minimum thickness of 0.010 inch. Minimum thickness of the outside edge of plastic caps shall be 0.035 inch. The cap nail shank shall be not less than 0.083 inch for ring shank cap nails and 0.091 inch for smooth shank cap nails. Staples shall be not less than 21 gage. Cap nail shank and cap staple legs shall have a length sufficient to penetrate through the roof sheathing or not less than 3/4 inch into the roof sheathing.
Metal roof shingles	1507.5		
Mineral-surfaced roll roofing	1507.6		
Slate shingles	1507.7		
Wood shingles	1507.8		
Wood shakes	1507.9		

Revise as follows:

1507.2.3 Underlayment. ~~Unless otherwise noted, required underlayment~~ Underlayment shall ~~conform to ASTM D 226, Type I, ASTM D 4869, Type I, or ASTM D 6757.~~ comply with Section 1507.1.1

Delete without substitution:

~~**1507.2.4 Self-adhering polymer modified bitumen sheet.** Self-adhering polymer modified bitumen sheet shall comply with ASTM D 1970.~~

Revise as follows:

~~**1507.2.8.2**~~ **1507.2.8 Ice barrier.** ~~In areas where there has been a history of~~ When required, ice ~~forming along the eaves causing a backup of water, an ice barrier that consists of at least two layers of underlayment cemented together or of a self-adhering polymer modified bitumen sheet~~ barriers shall ~~be used in lieu of normal underlayment and extend from the lowest edges of all roof surfaces to a point at least 24 inches (610 mm) inside the *exterior wall* line of the building~~ comply with Section 1507.1.2.

~~**Exception:** Detached accessory structures that contain no conditioned floor area.~~

Delete without substitution:

~~**1507.2.8 Underlayment application.** For roof slopes from two units vertical in 12 units horizontal (17-percent slope) and up to four units vertical in 12 units horizontal (33-percent slope), underlayment shall be two layers applied in the following manner. Apply a minimum 19-inch-wide (483 mm) strip of underlayment felt parallel with and starting at the eaves, fastened sufficiently to~~

~~hold in place. Starting at the eave, apply 36-inch-wide (914 mm) sheets of underlayment overlapping successive sheets 19 inches (483 mm) and fasten sufficiently to hold in place. Distortions in the underlayment shall not interfere with the ability of the shingles to seal. For roof slopes of four units vertical in 12 units horizontal (33-percent slope) or greater, underlayment shall be one layer applied in the following manner. Underlayment shall be applied shingle fashion, parallel to and starting from the eave and lapped 2 inches (51 mm), fastened sufficiently to hold in place. Distortions in the underlayment shall not interfere with the ability of the shingles to seal.~~

~~**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 $^3/_4$ inch (19.1 mm) into the roof sheathing.~~

> ~~**Exception:** As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.~~

Revise as follows:

1507.3.3 Underlayment. ~~Unless otherwise noted, required underlayment~~ Underlayment shall ~~conform to: ASTM D 226, Type II; ASTM D 2626 or ASTM D 6380, Class M mineral-surfaced roll roofing~~ comply with Section 1507.1.1.

Delete without substitution:

~~**1507.3.3.1 Low-slope roofs.** For roof slopes from 2$^1/_2$ units vertical in 12 units horizontal (21-percent slope), up to four units vertical in 12 units horizontal (33-percent slope), underlayment shall be a minimum of two layers applied as follows:~~

1. ~~Starting at the eave, a 19-inch (483 mm) strip of underlayment shall be applied parallel with the eave and fastened sufficiently in place.~~
2. ~~Starting at the eave, 36-inch-wide (914 mm) strips of underlayment felt shall be applied overlapping successive sheets 19 inches (483 mm) and fastened sufficiently in place.~~

~~**1507.3.3.2 High-slope roofs.** For roof slopes of four units vertical in 12 units horizontal (33-percent slope) or greater, underlayment shall be a minimum of one layer of underlayment felt applied shingle fashion, parallel to, and starting from the eaves and lapped 2 inches (51 mm), fastened only as necessary to hold in place.~~

~~**1507.3.3.3 High wind attachment.** 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 $^{3}/_{4}$ inch (19.1 mm) into the roof sheathing.~~

> ~~**Exception:** As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.~~

Revise as follows:

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, ASTM D 4869 Type IV, or ASTM D 1970~~ Section 1507.1.1. ~~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 with ASTM D 1970 shall be permitted.~~

1507.5.3 Underlayment. Underlayment shall comply with ~~ASTM D 226, Type I or ASTM D 4869~~ Section 1507.1.1.

Delete without substitution:

~~**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 $^{3}/_{4}$ inch (19.1 mm) into the roof sheathing.~~

> ~~**Exception:** As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.~~

Revise as follows:

1507.5.4 Ice barrier. ~~In areas where there has been a history of~~ When required, ice ~~forming along the eaves causing a backup of water, an ice barrier that consists of at least two layers of underlayment cemented together or of a self adhering polymer modified bitumen sheet~~ barriers shall ~~be used in lieu of normal underlayment and extend from the lowest edges of all roof surfaces to a point at least 24 inches (610 mm) inside the exterior wall line of the building~~ comply with Section 1507.1.2.

> ~~**Exception:** Detached accessory structures that contain no conditioned floor area.~~

1507.6.3 Underlayment. Underlayment shall comply with ~~ASTM D 226, Type I or ASTM D 4869~~ Section 1507.1.1.

Delete without substitution:

~~**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. 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 with ASTM D 1970 shall be permitted.~~

Revise as follows:

1507.6.4 Ice barrier. ~~In areas where there has been a history of~~ When required, ice ~~forming along the eaves causing a backup of water, an ice barrier that consists of at least two layers of underlayment cemented together or of a self adhering polymer modified bitumen sheet~~ barriers shall ~~be used in lieu of normal underlayment and extend from the lowest edges of all roof surfaces to a point at least 24 inches (610 mm) inside the exterior wall line of the building~~ comply with Section 1507.1.2.

~~Exception:~~ ~~Detached accessory structures that contain no conditioned floor area.~~

1507.7.3 Underlayment. Underlayment shall comply with ~~ASTM D 226, Type II or ASTM D 4869, Type III or IV~~ Section 1507.1.1.

Delete without substitution:

~~**1507.7.3.1 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 $^3/_4$ inch (19.1 mm) into the roof sheathing.~~

~~**Exception:** As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.~~

Revise as follows:

1507.7.4 Ice barrier. ~~In areas where the average daily temperature in January is 25°F (-4°C) or less or where there is a possibility of~~
When required, ice ~~forming along the eaves causing a backup of water, an ice barrier that consists of at least two layers of underlayment cemented together or of a self-adhering polymer-modified bitumen sheet~~ barriers shall ~~extend from the lowest edges of all roof surfaces to a point at least 24 inches (610 mm) inside the exterior wall line of the building~~ comply with Section 1507.1.2.

~~Exception:~~ ~~Detached accessory structures that contain no conditioned floor area.~~

TABLE 1507.8
WOOD SHINGLE AND SHAKE INSTALLATION

ROOF ITEM	WOOD SHINGLES	WOOD SHAKES
1. Roof slope	Wood shingles shall be installed on slopes of not less than three units vertical in 12 units horizontal (3:12).	Wood shakes shall be installed on slopes of not less than four units vertical in 12 units hor- izontal (4:12).

2. Deck requirement		
Temperate climate	Shingles shall be applied to roofs with solid or spaced sheathing. Where spaced sheathing is used, sheathing boards shall be 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.	Shakes shall be applied to roofs with solid or spaced sheathing. Where spaced sheathing is used, sheathing boards shall be 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.
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.	Solid sheathing is required.	Solid sheathing is required.
3. Interlayment	No requirements.	Interlayment shall comply with ASTM D 226, Type 1.
4. Underlayment		
Temperate climate	Underlayment shall comply with <u>Section 1507.1.1</u> ~~ASTM D 226, Type 1~~.	Underlayment shall comply with <u>Section 1507.1.1</u> ~~ASTM D 226, Type 1~~.
~~In areas where there is a possibility of ice forming along the eaves causing a backup of water.~~	~~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~~	~~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 to a point at least 24~~

	~~of the building.~~	~~inches inside the exterior wall line of the building.~~
5. Application		
Attachment	Fasteners for wood shingles shall be hot-dipped galvanized or Type 304 (Type 316 for coastal areas) stainless steel with a minimum penetration of 0.75 inch into the sheathing. For sheathing less than 0.5 inch thick, the fasteners shall extend through the sheathing.	Fasteners for wood shakes shall be hot-dipped galvanized or Type 304 (Type 316 for coastal areas) with a minimum penetration of 0.75 inch into the sheathing. For sheathing less than 0.5 inch thick, the fasteners shall extend through the sheathing.
No. of fasteners	Two per shingle.	Two per shake.
Exposure	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.
Method	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.	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.
Flashing	In accordance with Section 1507.8.8.	In accordance with Section 1507.9.9.

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~~ <u>Section 1507.1.1</u>.

Delete without substitution:

~~**1507.8.3.1 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 $^3/_4$ inch (19.1 mm) into the roof sheathing.~~

> ~~**Exception:** As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.~~

Revise as follows:

1507.8.4 Ice barrier. ~~In areas where there has been a history of~~ When required, ice ~~forming along the eaves causing a backup of water, an ice barrier that consists of at least two layers of underlayment cemented together or of a self-adhering polymer-modified bitumen sheet~~ barriers shall ~~be used in lieu of normal underlayment and extend from the lowest edges of all roof surfaces to a point at least 24 inches (610 mm) inside the exterior wall line of the building~~ comply with Section 1507.1.2.

> ~~**Exception:** Detached accessory structures that contain no conditioned floor area.~~

1507.9.3 Underlayment. Underlayment shall comply with ~~ASTM D 226, Type I or ASTM D 4869~~ Section 1507.1.1.

Delete without substitution:

~~**1507.9.3.1 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 $^3/_4$ inch (19.1 mm) into the roof sheathing.~~

> ~~**Exception:** As an alternative, adhered underlayment complying with ASTM D 1970 shall~~

~~be permitted.~~

Revise as follows:

1507.9.4 Ice barrier. ~~In areas where there has been a history of~~ When required, ice ~~forming along the eaves causing a backup of water, an ice barrier that consists of at least two layers of underlayment cemented together or of a self-adhering polymer-modified bitumen sheet~~ barriers shall ~~be used in lieu of normal underlayment and extend from the lowest edges of all roof surfaces to a point at least 24 inches (610 mm) inside the exterior wall line of the building~~ comply with Section 1507.1.2.

> ~~**Exception:** Detached accessory structures that contain no conditioned floor area.~~

1507.17.3 Underlayment. ~~Unless otherwise noted, required underlayment~~ Underlayment shall ~~conform to ASTM D 226, ASTM D 4869 or ASTM D 6757~~ comply with Section 1507.1.1.

~~1507.17.4.2~~ 1507.17.4 Ice barrier. ~~In areas where there has been a history of~~ When required, ice ~~forming along the eaves causing a backup of water, an ice barrier that consists of at least two layers of underlayment cemented together or of a self-adhering polymer modified bitumen sheet~~ barriers shall ~~be used instead of normal underlayment and extend from the lowest edges of all roof surfaces to a point not less than 24 inches (610 mm) inside the *exterior wall* line of the building~~ comply with Section 1507.1.1.

> ~~**Exception:** Detached accessory structures that contain no conditioned floor area.~~

Delete without substitution:

~~**1507.17.4 Underlayment application.** Underlayment shall be applied shingle fashion, parallel to and starting from the eave, lapped 2 inches (51 mm) and fastened sufficiently to hold in place.~~

~~**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 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 with ASTM D 1970 shall be permitted.~~

Reason: This proposal is primarily a reorganization of the underlayment provisions contained within the IBC. In the current IBC, underlayment provisions are specified individually for each type of roof covering. Many of the roof covering provisions contain similar and overlapping requirements for underlayment type, application, and attachment. This proposal relocates the underlayment requirements for each roof covering to a single section at the beginning of Section 1507. This reorganization results in three new tables that address underlayment type,

application, and attachment required for each of the roof covings in the IBC that require underlayment. Consolidating the underlayment requirements into a single section will make the provisions more user-friendly and in particular highlights the key differences between the requirements for underlayment for the different types of roof coverings addressed by the IBC. The reorganization proposed here was approved during the last code development cycle for the 2015 IRC.

This proposal also clarifies the use of ASTM D 1970 as an underlayment. The proposal does not require the use of the self-adhering membrane, as it is already permitted by the code. In fact, the existing exception for using the self-adhering membrane was requested to be included by the IBC Structural Committee, and subsequently approved by the IRC Committee during the last code change cycle so that it was clear that a self-adhering membrane was permitted as an alternative to the underlayment provisions for high wind. This proposal simply clarifies the permitted installations of the self-adhering membrane that would provide an equivalent or better level of water intrusion prevention to the underlayment requirements for high wind. The criteria specified are consistent with the IBHS Fortified program requirements for creating a "sealed roof deck". Additionally, the provisions of this proposal are the most widely accepted methods recognized by insurance companies for providing discounts and credits in hurricane-prone regions.

This proposal revises the wind speed threshold that triggers the enhanced underlayment provisions from V_{asd} = 120 mph to V_{ult} = 140 mph which will make the IBC consistent with the IRC. This adjustment was approved during the last code cycle and is included in the 2015 IRC. The original code change that pegged this trigger at 120 mph was developed to correspond with the wind speed maps in the 2009 IBC and ASCE 7-05. During the 2009/2010 code cycle, the wind speed maps in the IBC and IRC have been updated for consistency with ASCE 7-10. A simple conversion of the enhanced underlayment provisions wind speed trigger does not accurately reflect the intent of the original proposal, particularly as it relates to the geographic areas affected. The trigger as originally proposed, was essentially chosen to capture the coastal areas of the hurricane-prone regions, where the potential for loss of roof covering is increased, accompanied by exposure to significant amounts of rainfall. The trigger was chosen based upon a geographic location on the wind speed map rather than a particular design limitation. However, the new maps in ASCE 7-10 have shifted the contours closer to the coast for the entire hurricane-prone region, which resulted in a reduction of the geographic area required to comply with the enhanced underlayment provisions. This proposal sets wind speed trigger for the enhanced underlayment provisions at $V_{ult} \geq 140$ mph, which corresponds better geographically with the 120 mph trigger that was intended to work the 2009 IBC wind speed maps and is consistent with the 2015 IRC.

Additionally, this proposal simply adds an additional method for preventing water penetration when the primary roof covering is lost due to high winds. Water penetration has been well document from post-hurricane damage assessments where hurricane winds were strong enough to blow off the primary roof covering, but not strong enough to blow off roof sheathing. In such instances, significant property damage and extended occupant displacement routinely occur due to water intrusion. Such damage is particularly common in inland areas, where hurricane-strength winds occur, but building codes and standards are not as stringent as in coastal jurisdictions.

While enhanced underlayment provisions are currently addressed in the code, the protection afforded by the self-adhering polymer-modified bitumen underlayment is in a bit of a different category. The 2015 IBC permits the of this product as an alternative to the enhanced underlayment (felt) provisions for roofing products address in this code change. When the self-adhering polymer-modified bitumen underlayment as described in proposed Exceptions 1 and 2 is used, the condition it creates is referred to as a "sealed roof deck" in that it prevents water from entering the building through gaps in the roof sheathing. It is also a component of the IBHS Fortified Program for creating a sealed roof deck. Recent tests conducted at the IBHS Research Facility have found the system proposed as new Exception 3, to perform similar to the self-adhering polymer-modified bitumen underlayment. As a result, this system of underlayment application and attachment is now recognized by the Fortified Program for creating a sealed roof deck. While this system is currently required in the code for roof slopes between 2:12 and 4:12, it provides an equal level of water penetration protection for roof slopes above 4:12. Incorporating this method in the code provides an option for reducing the risk of water penetration that is on par with the self-adhering polymer-modified bitumen underlayment.

Cost Impact: Will increase the cost of construction
Will result in an increase in cost in some areas. The wind speed trigger for the enhanced underlayment has been slightly lowered resulting in it applying to more areas. However, this change will achieve consistency with IRC with regards to underlayment.

S28-16

IBC: 1507.17.4.1, 1507.2.4, 1507.2.8.1, 1507.2.9.2, 1507.3.3.3, 1507.3.9, 1507.4.5, 1507.5.3.1, 1507.5.7, 1507.6.3.1, 1507.7.3.1, 1507.8.3.1, 1507.8.8, 1507.9.3.1, 1507.9.9; IRC: R905.1.1.

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~~ 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 $^3/_4$ 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.

1507.2.9.2 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~~ bearing a label indicating compliance with ASTM D 1970 shall be permitted in lieu of the lining material.

1507.3.3.3 High wind attachment. 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 $^3/_4$ 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.

1507.3.9 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 $^3/_4$ 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 $^3/_4$ 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 $^3/_4$ 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. 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.

1507.7.3.1 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 $^3/_4$ 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.

1507.8.3.1 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 $^3/_4$ 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 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.

1507.9.3.1 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 $^3/_4$ inch (19.1 mm) into the roof sheathing.

> **Exception:** As an alternative, adhered underlayment ~~complying~~ <u>bearing a label indicating compliance</u> 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~~ <u>bearing a label indicating compliance</u> 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 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.

2015 International Residential Code

Revise as follows:

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_{ult}, 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.

S28-16 : 1507.2.4-FISCHER13505

S29-16

Part I:
IBC: 1507.2.5, 1507.2.7.
Part II:
IRC: R905.2.6.

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC-BUILDING COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES

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

Part I

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~~ 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~~ in accordance with the ~~manufacturer~~ manufacturer's *approved* installation instructions.

Part II

2015 International Residential Code

Revise as follows:

R905.2.6 Attachment. Asphalt shingles shall have the minimum number of fasteners required by the ~~manufacturer~~ 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~~ 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.

S29-16 : 1507.2.5-FISCHER13532

S30-16

IBC: 1507.2.5.

Proponent : Jason Wilen AIA CDT RRO, National Roofing Contractors Association (NRCA), representing National Roofing Contractors Association (NRCA) (jwilen@nrca.net); T. Eric Stafford, PE, representing Institute for Business and Home Safety (testafford@charter.net)

2015 International Building Code

Revise as follows:

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

Reason: WILEN: ASTM D 225 has been withdrawn by ASTM and thus should no longer be referenced in IBC as a material standard for asphalt shingles with organic felt. Organic shingles are no longer available and a similar change was approved for removing ASTM D 255 from Chapter 9 of IRC for the 2015 edition (RB417-13). This change will make IBC Section 1507.2.5 consistent with IRC Section R905.2.4.
STAFFORD: ASTM D 225 was withdrawn by ASTM International in 2012 because shingles with organic felt are no longer produced by U.S manufacturers, nor are these products available on the market. Code change RB417–13 removed ASTM D 225 from the IRC for the same reason. The recognized standard specification for asphalt shingles is ASTM D 3462.

Cost Impact: Will not increase the cost of construction
The proposed change does not change the stringency of existing code requirements so the cost of construction will be unchanged.

S30-16 : 1507.2.5-WILEN AIA CDT RRO13147

S31-16

IBC: 1507.17.4.1, 1507.2.8.1, 1507.3.3.3, 1507.4.5, 1507.5.3.1, 1507.6.3.1, 1507.7.3.1, 1507.8.3.1, 1507.9.3.1.

Proponent : Andy Williams, representing International Staple, Nail, and Tool Association (afwilliams@Connect2amc.com)

2015 International Building Code

Revise as follows:

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 cap nails or cap staples. Caps shall be metal or plastic ~~cap nails~~ with a nominal head diameter of not less than 1 inch (~~25~~ 25.4 mm) ~~with~~ . Metal caps shall have a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. ~~The cap nail~~ Power-driven metal caps shall have a thickness of not less than 0.010 inch (0.25mm). Thickness of the outside edge of plastic caps shall be not less than 0.035 inch (0.89mm). Cap-nail shank shall be a minimum of 12 gage [0.105 inch (2.67 mm)] ~~with~~ . Staple gage shall be not less than 21 gage [0.032 inch (0.81mm)]. Cap-nail shank and cap staple legs shall have a length sufficient to penetrate through the roof sheathing or a minimum of $^3/_4$ inch (19.1 mm) into the roof sheathing.

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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. 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 <u>cap nails or cap staples. Caps shall be </u>metal or plastic ~~cap nails ~~with a <u>nominal</u> head diameter of not less than 1 inch (~~25~~ <u>25.4</u> mm)~~ with~~ <u>. Metal caps shall have</u> a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. ~~The cap nail~~ <u>Power-driven metal caps shall have a thickness of not less than 0.010 inch (0.25mm). Thickness of the outside edge of plastic caps shall be not less than 0.035 inch (0.89mm). Cap-nail</u> shank shall be a minimum of 12 gage [0.105 inch (2.67 mm)]~~ with~~ <u>. Staple gage shall be not less than 21 gage [0.032 inch (0.81mm)]. Cap-nail shank and cap staple legs shall have</u> a length<u> sufficient</u> to penetrate through the roof sheathing or a minimum of $^3/_4$ inch (19.1 mm) into the roof sheathing.

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1507.7.3.1 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 cap nails or cap staples. Caps shall be metal or plastic ~~cap nails~~ with a nominal head diameter of not less than 1 inch (~~25~~ 25.4 mm)~~with~~. Metal caps shall have a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. ~~The cap nail~~ Power-driven metal caps shall have a thickness of not less than 0.010 inch (0.25mm). Thickness of the outside edge of plastic caps shall be not less than 0.035 inch (0.89mm). Cap-nail shank shall be a minimum of 12 gage [0.105 inch (2.67 mm)]~~with~~. Staple gage shall be not less than 21 gage [0.032 inch (0.81mm)]. Cap-nail shank and cap staple legs shall have a length sufficient to penetrate through the roof sheathing or a minimum of $^3/_4$ inch (19.1 mm) into the roof sheathing.

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1507.9.3.1 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 cap nails or cap staples. Caps shall be metal or plastic ~~cap nails~~ with a nominal head diameter of not less than 1 inch (~~25~~ 25.4 mm) ~~with~~ . Metal caps shall have a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. ~~The cap nail~~ Power-driven metal caps shall have a thickness of not less than 0.010 inch (0.25mm). Thickness of the outside edge of plastic caps shall be not less than 0.035 inch (0.89mm). Cap-nail shank shall be a minimum of 12 gage [0.105 inch (2.67 mm)] ~~with~~ . Staple gage shall be not less than 21 gage [0.032 inch (0.81mm)]. Cap-nail shank and cap staple legs shall have a length sufficient to penetrate through the roof sheathing or a minimum of $^3/_4$ inch (19.1 mm) into the roof sheathing.

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Reason: This code change proposal is identical to the proposal approved as submitted by ISANTA in 2013 for the 2015 IRC.

The IBC requirement for cap nails for attachment of roof covering underlayment in high-w ind areas does not reflect commercially available cap staples successfully used in roofing material application. This proposal expands fastener alternatives to include cap staples based on testing w hich indicates the underlayment tears before the proposed cap staples fail. The cap staple bearing area on the underlayment is the same as for the cap nail - as determined by the cap diameter.

Testing w as conducted in general accordance w ith ASTM D4869 using Type IV underpayment ("30 pound"). That underpayment is at the high end of the thickness and toughness range of code-required underlayment w hich makes this a "w orst case test" for the cap staples.

The test procedure and results are as follow s

Report on Testing

July 2012

Testing w as performed by Stanley Black & Decker at the request of Internaitonal Staple, Nail and Tool Association (ISANTA)

Materials

- Roofing paper, 30# (ASTM D4869, Type IV)
- Sheathing material - 4-ply, 15/32-inch. Southern Pine Plyw ood, cut in 2 by 2 inch squares
- Fasteners - Ring shank cap nails w ith nail shank diameters before threading of 0.083 inch and 0.105 inch. Cap staples, 18 gage and 21 gage.
- Caps - 1 inch diameter plastic caps

Method

The test method w as designed to facilitate one of three potential failure modes: cap failure, fastener w ithdraw al, or cap pulling through the underlayment. A 14 x 14 inch sheet of underlayment w as cut from the roll. the cap-fastener w as driven through the center of the underlayment sheet into a 2x2 inch block of sheathing matrail. The assembled test specimen w as turned over so that the sheathing block w as visible and the fastener head w as dow n. The assembled speciment w as secured in the test fixture base w ith the fastener centered below the sheathing block clamping fixture. The sheathing block w as clamped by the fixture attached to the traversing head of the test machine. The test specimen w as loaded at a constant displacement of 1 inch/minute until failure. Load and displacemtn w ere measured continuously during the test. Failure mode w as observed and peak force w as recorded as the failure load. Photographs are attached.

Discussion

The test is intended to evaluate the functionality of the ISANTA proposal for adding additionalcommercially available cap fasteners for use on the same spacing as IBC's 0.105" cap nail w ith a plastic or metal 1" diameter cap (as specified).The underlayment is not w ind qualified. How ever, AC188 evaluation includes a requirement for tensile strength by using one of threeASTM standards, for example, ASTM D412. The AC does not include a punch-through or pull-through evaluation. The minimum tensile strength criterion of AC 188 is 20 lbf/inch-w idth. The 20 lbf/inch-w idth is a valuable benchmark in that it could also be used to assess the potential uplift resistance of teh underlayment because that is controlled by tensile strength.

Tensile strength also appears to be a predictor of pull through performance. The 1-inch caps generally pulled through the underlayment at approximately 32 lb. Some nonlinear behaviour occurs at the start of the loading process, then the load-deflection diagram becomes linear and as the load approaches the maximum a minor plastic region develops that reflects fiber separation and cap yielding. This w as generally characteristic for all cap-fasteners.

Conclusions

From the testing and review of test standardsand acceptance criteria, w e can conclude that the underlayment minimum tensile strength is teh controlling strength property of the system and is can be used as a reasonable approximation of the potential holding capacity of the cap-fasteners based on the cap diameter. Engineering analysis of the negative pressures on roof surfaces should provide reasonble estimates of expected forces that w ill be resisted by fasteners and can be used to establish fastening schedules that reflect the fastener holding capacity (pull-through or w ithdraw al) and tensile strength of the underlayment w hen loaded as a membrane betw een fasteners.

Results of Cap Fastener Testing w ith ASTM D4869, Type IV Underlayment

(# of Failures, By Failure Mode)

Cap Fastener[1]	Failure Load (pounds)	Fastener Withdrawl	Cap Failure	Underla
"Code Nail" 2012 IBC Cap Nail 0.105" nail diameter ring shank nail	31.8	1	7	8
0.083" nail diameter ring shank nail	32.4	0	4	2
21 Gage staple	36.2	0	0	5
18 Gage staple	32.1	0	2	9

References:

State of Florida

- Testing Application Standards (TAS) published in the State of Florida Buildign Code 2007 for High Velocity Hurricane Zone (HVHZ) product approval testing.
- TAS 111(B)-95, Test Procedure for Edge Metal Pull-off Performance
- TAS 117(C)-95, Test Procedure for Dynamic Pull-off Performance of Roofing Nail Heads or Fasteners w ith Bearing Plates
- TAS 117(A)-95, Test Procedure for Withdraw l Resistance Testing of MEchanical Fasteners Used in Roof System Assemblies
- TAS 117(B)-95, Test Procedure for Dynamic pull-through Performance of Roofing Membranes over Fastener Heads or Fasteners w ith Metal Bearing Plates.

ASTM Standards

- ASTM D1037, Standard Test Methods for Evaluating Properties of Wood-base fiber and Particle Panel Materials, Nail head Pull-through Test.
- ASTM D4869, Standard Specification for Asphalt-Saturated Organic Felt Underlayment Used in Steep Slope Roofing.
- ASTM D412, Test MEthod for Vulcanized Rubber and Thermoplastic Rubbers and Thermoplastic Elastomers-Tension.

Acceptance Criteria

- ICC-ES, AC188: Acceptance Criteria for Roof Underlayments. July 2007.

Cost Impact: Will not increase the cost of construction
Recognition of these cap nails and staples should provide greater choice to the end user of those products that are already commercially available and allow ed by the IRC.

S31-16 : 1507.2.8.1-WILLIAMS11446

S32-16

IBC: 1507.17.4.1, 1507.2.8.1, 1507.3.3.3, 1507.4.5, 1507.5.3.1, 1507.6.3.1, 1507.7.3.1, 1507.8.3.1, 1507.9.3.1.

Proponent : Andy Williams (afwilliams@Connect2amc.com)

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

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. 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~~ . Metal caps shall have a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. Power-driven metal caps shall have a minimum thickness of 0.010 onch (0.25mm). Miimum thickness of the outside edge of plastic caps shall be 0.035 inch (0.89mm). The ~~cap nail~~ cap-nail shank shall be a minimum of ~~12 gage [0.105~~ 0.083 inch (~~2.67~~ 2.11 mm)~~] with~~ for ring shank cap nails and 0.091 inch (2.31mm) for smooth shank cap nails. Cap nails shall have 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 with ASTM D 1970 shall be permitted.~~

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

1507.7.3.1 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~~ . Metal caps shall have a thickness of at least 32-gage [0.0134 inch

(0.34 mm)] sheet metal. Power-driven metal caps shall have a thickness of not less than 0.010 onch (0.25mm). Yhickness of the outside edge of plastic caps shall be not less than 0.035 inch (0.89mm). The ~~cap nail~~ cap-nail shank shall be a minimum of ~~12 gage [0.105~~ 0.083 inch (~~2.67~~ 2.11 mm)~~] with~~ for ring shank cap nails and 0.091 inch (2.31mm) for smooth shank cap nails. Cap nails shall have 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 with ASTM D 1970 shall be permitted.~~

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

1507.8.3.1 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~~ . Metal caps shall have a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. Power-driven metal caps shall have a thickness of not less than 0.010 onch (0.25mm). Thickness of the outside edge of plastic caps shall be not less than 0.035 inch (0.89mm). The ~~cap nail~~ cap-nail shank shall be a minimum of ~~12 gage [0.105~~ 0.083 inch (~~2.67~~ 2.11 mm)~~] with~~ for ring shank cap nails and 0.091 inch (2.31mm) for smooth shank cap nails. Cap nails shall have 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 with ASTM D 1970 shall be permitted.~~

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

1507.9.3.1 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~~ . Metal caps shall have a thickness of at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. Power-driven metal caps shall have a thickness of not less than 0.010 onch (0.25mm). Thickness of the outside edge of plastic caps shall be 0.035 inch (0.89mm). The ~~cap nail~~ cap-nail shank shall be a minimum of ~~12 gage [0.105~~ 0.083 inch (~~2.67~~ 2.11 mm)~~]~~ ~~with~~ for ring shank cap nails and 0.091 inch (2.31mm) for smooth shank cap nails. Cap nails shall have 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 with ASTM D 1970 shall be permitted.~~

> **Exception:** As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.

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~~ . Metal caps shall have a thickness of ~~not less than~~ at least 32-gage [0.0134 inch (0.34 mm)] sheet metal. Power-driven metal caps shall have a thickness of not less than 0.010 onch (0.25mm). Thickness of the outside edge of plastic caps shall be 0.035 inch (0.89mm). The ~~cap nail~~ cap-nail shank shall be a minimum of ~~12 gage [0.105~~ 0.083 inch (~~2.67~~ 2.11 mm)~~] with~~ for ring shank cap nails and 0.091 inch (2.31mm) for smooth shank cap nails. Cap nails shall have 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 with ASTM D 1970 shall be permitted.

Reason:

This code change proposal is identical to the proposal approved as submitted by ISANTA in 2013 for the 2015 IRC.

The cap nails listed for attachment of roof covering underlayment in high-wind areas does not reflect commercially available cap staples successfully used in roofing material application. IBC presently lists a minimum nail shank of 0.105". This proposal lowers the minimum shank diameter based on tests indicating that the underlayment tears prior to failure of the proposed cap nail. (Minimum diameter of 0.083" for ring shank nails and minimum diameter of 0.091" for smooth shank cap nails.)

Testing was conducted in general accordance with ASTM D4869 using Type IV underpayment ("30 pound"). That underpayment is at the high end of the thickness and toughness range of code-required underlayment which makes this a "worst case test" for the fastener. This proposal addresses both commercially available hand-driven and

pow er-driven cap-nails.

The test procedure and results are as follow s

Report on Testing

July 2012

Testing w as performed by Stanley Black & Decker at the request of Internaitonal Staple, Nail and Tool Association (ISANTA)

Materials

- Roofing paper, 30# (ASTM D4869, Type IV)
- Sheathing material - 4-ply, 15/32-inch. Southern Pine Plyw ood, cut in 2 by 2 inch squares
- Fasteners - Ring shank cap nails w ith nail shank diameters before threading of 0.083 inch and 0.105 inch. Cap staples, 18 gage and 21 gage.
- Caps - 1 inch diameter plastic caps

Method

The test method w as designed to facilitate one of three potential failure modes: cap failure, fastener w ithdraw al, or cap pulling through the underlayment. A 14 x 14 inch sheet of underlayment w as cut from the roll. the cap-fastener w as driven through the center of the underlayment sheet into a 2x2 inch block of sheathing matrail. The assembled test specimen w as turned over so that the sheathing block w as visible and the fastener head w as dow n. The assembled speciment w as secured in the test fixture base w ith the fastener centered below the sheathing block clamping fixture. The sheathing block w as clamped by the fixture attached to the traversing head of the test machine. The test specimen w as loaded at a constant displacement of 1 inch/minute until failure. Load and displacement w ere measured continuously during the test. Failure mode w as observed and peak force w as recorded as the failure load. Photographs are attached.

Discussion

The test is intended to evaluate the functionality of the ISANTA proposal for adding additionalcommercially available cap fasteners for use on the same spacing as IBC's 0.105" cap nail w ith a plastic or metal 1" diameter cap (as specified).The underlayment is not w ind qualified. How ever, AC188 evaluation includes a requirement for tensile strength by using one of threeASTM standards, for example, ASTM D412. The AC does not include a punch-through or pull-through evaluation. The minimum tensile strength criterion of AC 188 is 20 lbf/inch-w idth. The 20 lbf/inch-w idth is a valuable benchmark in that it could also be used to assess the potential uplift resistance of teh underlayment because that is controlled by tensile strength.

Tensile strength also appears to be a predictor of pull through performance. The 1-inch caps generally pulled through the underlayment at approximately 32 lb. Some nonlinear behaviour occurs at the start of the loading process, then the load-deflection diagram becomes linear and as the load approaches the maximum a minor plastic region develops that reflects fiber separation and cap yielding. This w as generally characteristic for all cap-fasteners.

Conclusions

From the testing and review of test standardsand acceptance criteria, w e can conclude that the underlayment minimum tensile strength is teh controlling strength property of the system and is can be used as a reasonable approximation of the potential holding capacity of the cap-fasteners based on the cap diameter. Engineering analysis of the negative pressures on roof surfaces should provide reasonble estimates of expected forces that w ill be resisted by fasteners and can be used to establish fastening schedules that reflect the fastener holding capacity (pull-through or w ithdraw al) and tensile strength of the underlayment w hen loaded as a membrane betw een fasteners.

Results of Cap Fastener Testing w ith ASTM D4869, Type IV Underlayment

(# of Failures, By Failure Mode)

Cap Fastener[1]	Failure Load (pounds)	Fastener Withdrawl	Cap Failure	Underla
"Code Nail" 2012 IBC Cap Nail 0.105" nail diameter	31.8	1	7	8

ring shank nail

0.083" nail diameter ring shank nail	32.4	0	4	2
21 Gage staple	36.2	0	0	5
18 Gage staple	32.1	0	2	9

References:

Reference Standards

State of Florida

- Testing Application Standards (TAS) published in the State of Florida Buildign Code 2007 for High Velocity Hurricane Zone (HVHZ) product approval testing.
- TAS 111(B)-95, Test Procedure for Edge Metal Pull-off Performance
- TAS 117(C)-95, Test Procedure for Dynamic Pull-off Performance of Roofing Nail Heads or Fasteners with Bearing Plates
- TAS 117(A)-95, Test Procedure for Withdrawl Resistance Testing of MEchanical Fasteners Used in Roof System Assemblies
- TAS 117(B)-95, Test Procedure for Dynamic pull-through Performance of Roofing Membranes over Fastener Heads or Fasteners with Metal Bearing Plates.

ASTM Standards

- ASTM D1037, Standard Test Methods for Evaluating Properties of Wood-base fiber and Particle Panel Materials, Nail head Pull-through Test.
- ASTM D4869, Standard Specification for Asphalt-Saturated Organic Felt Underlayment Used in Steep Slope Roofing.
- ASTM D412, Test MEthod for Vulcanized Rubber and Thermoplastic Rubbers and Thermoplastic Elastomers-Tension.

Acceptance Criteria

- ICC-ES, AC188: Acceptance Criteria for Roof Underlayments. July 2007.

Cost Impact: Will not increase the cost of construction
Recognition of these ring shank and smooth cap nails should provide greater choice to the end user of those products that are already commercially available and allowed by the IRC.

S32-16 : 1507.2.8.1-WILLIAMS11529

S33-16

Part I:

IBC: 1507.17.4.2, 1507.2.8.2, 1507.5.4, 1507.6.4, 1507.7.4, 1507.8.4, 1507.9.4, 1511.3.

Part II:

IRC: R905.1.2, R905.16.4.1, R905.2.7, R905.4.3.1, R905.5.3.1, R905.6.3.1, R905.7.1.1, R905.7.3.1, R905.8.1.1, R905.8.3.1, R908.3.

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC-BUILDING COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

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

Part I

2015 International Building Code

Revise as follows:

1507.2.8.2 ~~Ice barrier~~ <u>Water protection from ice damming</u>. In areas where ~~there~~ <u>the average daily temperature in January is 30°F (-1°C) or less, or where the building official</u> has ~~been~~ <u>determined there is</u> a ~~history~~ <u>possibility</u> of ice forming along the eaves causing a backup of water, ~~an ice~~ <u>a</u> barrier that consists of at least two layers of underlayment cemented together or of a self-adhering polymer modified bitumen sheet shall be used in lieu of normal underlayment and extend from the lowest edges of all roof surfaces to a point at least 24 inches (610 mm) inside the *exterior wall* line of the building.

> **Exception:**Detached accessory structures that contain no conditioned floor area.

1507.5.4 ~~Ice barrier~~ <u>Water protection from ice damming</u>. In areas where ~~there~~ <u>the average daily temperature in January is 30°F (-1°C) or less, or where the building official</u> has ~~been~~ <u>determined there is</u> a ~~history~~ <u>possibility</u> of ice forming along the eaves causing a backup of water, ~~an ice~~ <u>a</u> barrier that consists of at least two layers of underlayment cemented together or of a self-adhering polymer-modified bitumen sheet shall be used in lieu of normal underlayment and extend from the lowest edges of all roof surfaces to a point at least 24 inches (610 mm) inside the exterior wall line of the building.

> **Exception:**Detached accessory structures that contain no conditioned floor area.

1507.6.4 ~~Ice barrier~~ <u>Water protection from ice damming</u>. In areas where ~~there~~ <u>the average daily temperature in January is 30°F (-1°C) or less, or where the building official</u> has ~~been~~ <u>determined there is</u> a ~~history~~ <u>possibility</u> of ice forming along the eaves causing a backup of water, ~~an ice~~ <u>a</u> barrier that consists of at least two layers of underlayment cemented together or of a self-adhering polymer-modified bitumen sheet shall be used in lieu of normal underlayment and extend from the lowest edges of all roof surfaces to a point at least 24 inches (610 mm) inside the exterior wall line of the building.

> **Exception:**Detached accessory structures that contain no conditioned floor area.

1507.7.4 ~~Ice barrier~~ <u>Water protection from ice damming</u>. In areas where the average daily temperature in January is ~~25~~ <u>30</u>°F (~~-4~~ <u>1</u>°C) or less<u>,</u> or where<u> the building official has determined</u>

there is a possibility of ice forming along the eaves causing a backup of water, ~~an ice~~ a 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 to a point at least 24 inches (610 mm) inside the exterior wall line of the building.

Exception: Detached accessory structures that contain no conditioned floor area.

1507.8.4 ~~Ice barrier~~ Water protection from ice damming. In areas where ~~there~~ the average daily temperature in January is 30°F (-1°C) or less, or where the building official has ~~been~~ determined there is ~~a history~~ possibility of ice forming along the eaves causing a backup of water, ~~an ice~~ a barrier that consists of at least two layers of underlayment cemented together or of a self-adhering polymer-modified bitumen sheet shall be used in lieu of normal underlayment and extend from the lowest edges of all roof surfaces to a point at least 24 inches (610 mm) inside the exterior wall line of the building.

Exception: Detached accessory structures that contain no conditioned floor area.

1507.9.4 ~~Ice barrier~~ Water protection from ice damming. In areas where ~~there~~ the average daily temperature in January is 30°F (-1°C) or less, or where the building official has ~~been~~ determined there is ~~a history~~ possibility of ice forming along the eaves causing a backup of water, ~~an ice~~ a barrier that consists of at least two layers of underlayment cemented together or of a self-adhering polymer-modified bitumen sheet shall be used in lieu of normal underlayment and extend from the lowest edges of all roof surfaces to a point at least 24 inches (610 mm) inside the exterior wall line of the building.

Exception: Detached accessory structures that contain no conditioned floor area.

1507.17.4.2 ~~Ice barrier~~ Water protection from ice damming. In areas where ~~there~~ the average daily temperature in January is 30°F (-1°C) or less, or where the building official has ~~been~~ determined there is a ~~history~~ possibility of ice forming along the eaves causing a backup of water, ~~an ice~~ a barrier that consists of at least two layers of underlayment cemented together or of a self-adhering polymer modified bitumen sheet shall be used instead of normal underlayment and extend from the lowest edges of all roof surfaces to a point not less than 24 inches (610 mm) inside the *exterior wall* line of the building.

Exception: Detached accessory structures that contain no conditioned floor area.

1511.3 Roof replacement. *Roof replacement* shall include the removal of all existing layers of roof coverings down to the roof deck.

Exception: Where the existing roof assembly includes ~~an ice~~ a barrier membrane that provides water protection from ice damming that is adhered to the roof deck, the existing ~~ice~~ barrier membrane shall be permitted to remain in place and covered with an additional layer of ~~ice~~ barrier membrane in accordance with Section 1507.

Part II

2015 International Residential Code

Revise as follows:

R905.1.2 ~~Ice barriers~~ Water protection from ice damming. In areas where the average daily temperature in January is 30°F (-1°C) or less, or where the building official has determined there

has been a ~~history~~ possibility of ice forming along the eaves causing a backup of water as designated in Table R301.2(1), ~~an ice~~ a barrier shall be installed for asphalt shingles, metal roof shingles, mineral-surfaced roll roofing, slate and slate-type shingles, wood shingles ~~and~~ , wood shakes and photovoltaic shingles. The ~~ice~~ barrier shall consist of not fewer than two layers of underlayment cemented together, or a self-adhering polymer-modified bitumen sheet shall be used in place of normal underlayment and extend from the lowest edges of all roof surfaces to a point not less than 24 inches (610 mm) inside the exterior wall line of the building. On roofs with slope equal to or greater than 8 units vertical in 12 units horizontal, the ~~ice~~ barrier shall also be applied not less than 36 inches (914 mm) measured along the roof slope from the eave edge of the building.

Exception: Detached accessory structures not containing conditioned floor area.

Delete and substitute as follows:

R905.2.7 ~~Ice barrier~~ Water protection from ice damming. ~~Where required, ice barriers shall comply with Section R905.1.2.~~
Where required, water protection from ice damming shall comply with Section R905.1.2.

R905.4.3.1 ~~Ice barrier~~ Water protection from ice damming. ~~Where required, ice barriers shall comply with Section R905.1.2.~~
Where required, water protection from ice damming shall comply with Section R905.1.2.

R905.5.3.1 ~~Ice barrier~~ Water protection from ice damming. ~~Where required, ice barriers shall comply with Section R905.1.2.~~
Where required, water protection from ice damming shall comply with Section R905.1.2.

R905.6.3.1 ~~Ice barrier~~ Water protection from ice damming. ~~Where required, ice barriers shall comply with Section R905.1.2.~~
Where required, water protection from ice damming shall comply with Section R905.1.2.

Revise as follows:

R905.7.1.1 Solid sheathing required. In areas where the average daily temperature in January is ~~25~~ 30°F (~~-4~~ 1°C) or less, solid sheathing is required on that portion of the roof requiring ~~the application of an~~ water protection from ice ~~barrier~~ damming.

Delete and substitute as follows:

R905.7.3.1 ~~Ice barrier~~ Water protection from ice damming. ~~Where required, ice barriers shall comply with Section R905.1.2.~~
Where required, water protection from ice damming shall comply with Section R905.1.2.

Revise as follows:

R905.8.1.1 Solid sheathing required. In areas where the average daily temperature in January is ~~25~~ 30°F (~~-4~~ 1°C) or less, solid sheathing is required on that portion of the roof requiring ~~an~~ water protection from ice ~~barrier~~ damming.

Delete and substitute as follows:

R905.8.3.1 ~~Ice barrier~~ Water protection from ice damming. ~~Where required, ice barriers shall comply with Section R905.1.2.~~
Where required, water protection from ice damming shall comply with Section R905.1.2.

R905.16.4.1 ~~Ice barrier~~ Water protection from ice damming. ~~In areas where there has been a history of ice forming along the eaves causing a backup of water, as designated in Table R301.2(1), an ice barrier that consists of not less than two layers of underlayment cemented together or of a self-adhering polymer modified bitumen sheet shall be used in lieu of normal underlayment and extend from the lowest edges of all roof surfaces to a point not less than 24 inches (610 mm) inside the exterior wall line of the building.~~

> ~~**Exception:** Detached accessory structures that contain no conditioned floor area.~~

Where required, water protection from ice damming shall comply with Section R905.1.2.

Revise as follows:

R908.3 Roof replacement. Roof replacement shall include the removal of existing layers of roof coverings down to the roof deck.

> **Exception:** Where the existing roof assembly includes ~~an ice~~ a barrier membrane that provides water protection from ice damming that is adhered to the roof deck, the existing ~~ice~~ barrier membrane shall be permitted to remain in place and covered with an additional layer of ~~ice~~ barrier membrane in accordance with Section R905.

Reason: Revision of the section titles merely reflects that the use of the materials and methods contained in this section do not provide a barrier to ice, but merely protection from the w ater back up that is created by ice forming. Additionally, there are many areas in the US w here an ice barrier is not being required, despite w idespread instances of ice damming in those geographical areas. This proposal w ould provide additional guidance to the building official for determining the requirement for this protection. Finally, the temperature included in the proposed modification is consistent w ith the NRCA guidelines for installation of w ater protection from ice damming.

Cost Impact: Will not increase the cost of construction
The proposal is intended to be a clarification of the current intent of the code.

S33-16 : 1507.2.8.2-FISCHER13519

S34-16

Part I:
IBC: 1507.2.9.
Part II:
IRC: R905.2.8.

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC-BUILDING COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES

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

Part I

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~~ <u>*approved* installation</u> instructions.

Part II

2015 International Residential Code

Revise as follows:

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

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.

S34-16 : 1507.2.9-FISCHER13534

S36-16

IBC: 1507.4.4.

Proponent : Andy Williams (afwilliams@Connect2amc.com)

2015 International Building Code

Revise as follows:

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

1. Galvanized fasteners shall be used for steel roofs.
2. Copper, brass, bronze, copper alloy or 300 series stainless-steel fasteners shall be used for copper roofs.
3. Stainless-steel fasteners are acceptable for all types of metal roofs.
4. Aluminum fasteners are acceptable for aluminum roofs attached to aluminum supports.

Reason: Use of the w ord "approved" as it is currently positioned in these sections leads the reader to beleive that there is some process that should be applied to the manufacturer. That is not the case and the intent is that the fasteners should be the "approved" subject. Relocating the w ord "approved" in front of "fasteners" makes more sense and provides direction on w hat level of fasteners should be used.

Cost Impact: Will not increase the cost of construction
This proposal clarified the intent of the code and should not increase the cost.

S36-16 : 1507.4.4-WILLIAMS11547

S37-16

IBC: 1507.8, 1507.8.3, 1507.8.3.1, 1507.8.6, 1507.9, 1507.9.3, 1507.9.3.1, 1507.9.5, 1507.9.7, 1507.9.8.

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

ROOF ITEM	WOOD SHINGLES	WOOD SHAKES
1. Roof slope	Wood shingles shall be installed on slopes of not less than three units vertical in 12 units horizontal (3:12).	Wood shakes shall be installed on slopes of not less than four units vertical in 12 units horizontal (4:12).
2. Deck requirement		
Temperate climate	Shingles shall be applied to roofs with solid or spaced sheathing. Where spaced sheathing is used, sheathing boards shall be 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.	Shakes shall be applied to roofs with solid or spaced sheathing. Where spaced sheathing is used, sheathing boards shall be 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.
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.	Solid sheathing is required.	Solid sheathing is required.
3. Interlayment	No requirements.	Interlayment shall comply with ASTM D 226, Type ~~I~~ II or ASTM D 4869 Type IV.
4. Underlayment		
Temperate climate	Underlayment shall comply with ASTM D 226, Type ~~1~~II or ASTM D 4869 Type IV.	Underlayment shall comply with ASTM D 226, Type ~~1~~ II orASTM D 4869 Type IV.
In areas where there is a possibility of ice forming along the eaves causing a backup of water.	An ice barrier that consists of at least two lay- ers 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.	An ice barrier that consists of at least two lay- ers 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.
5. Application		
Attachment	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. ft ^{2})with a minimum penetration of 0.75 inch into the sheathing. For sheathing less than ~~0.5~~ 0.75 inch thick, the fasteners shall extend through the sheathing.	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. ft ^{2}) with a minimum penetration of 0.75 inch into the sheathing. For sheathing less than ~~0.5~~ 0.75 inch thick, the fasteners shall extend through the sheathing.
No. of fasteners	Two per shingle.	Two per shake.

Exposure	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.
Method	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.	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.~~
Flashing	In accordance with Section 1507.8.8.	In accordance with Section 1507.9.9.

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~~ II 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_{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 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 $^3/_4$ 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. ft^2). 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 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 ~~1/2~~ 3/4 inch (~~12.7 mm~~ 19.1mm) in thickness, the fasteners shall extend through the sheathing. Each shingle shall be attached with a minimum of two fasteners.

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~~ II or ASTM D 4869 Type IV.

1507.9.3.1 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 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 $^3/_4$ 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~~ II 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 ft^2). 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 ~~1/2~~ 3/4 inch (~~12.7~~ 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 ~~taper sawn~~ tapersawn shakes of naturally durable wood~~ and shall be $^1/_4$ to $^3/_8$ inch (6.4 to 9.5 mm) for preservative taper sawn 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 w ith the shingle or shake manufacturer's instructions. This is to clarify that the shingle or shake manufacturer's instructions and the w ritten code should prevail if there are issues w ith the required product's installation. There are other clarifications that make the language in the IBC the same as in the IRC.
They w ill have no cost impact.

In 1507.8.3.1: The specific underlayment is better defined than in the current code. The heavier w eight 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 w here moisture from dew and rain accumulates on the surface of the ASTM D 1970 type products and then is driven into the shingle w hen the roof cycles from cold to hot. Wood w hen w et on one side and dry on the opposite side w ill result in the shingle or shake cupping. Moisture barrier products, such as those complying w ith ASTM D1970 and similiar products retain moisture on their surface, w hich can be driven into the w ood creating cupping. This phenomenom 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, w hich drives the moisture out of the system. The roof deck is alw ays cooler than w hen 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 phenomenom.

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 in 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 shinlges or shakes are installed.

1507.9.8 There is no need to have different dimensional requirements for tapersaw n shakes.

Bibliography: Maze Nails w eb site.
http://w w w .mazenails.com/catalog/

Home Depot w eb site.

ttp://w w w .homedepot.com/

Cost Impact: Will increase the cost of construction
Section 1507.8
There w ill 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 w ould 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 likley 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 w ill 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.

S37-16 : 1507.8-ROODVOETS4174

S38-16

IBC: 1507.8, 1507.9.

Proponent : Jason Wilen AIA CDT RRO, National Roofing Contractors Association (NRCA), representing National Roofing Contractors Association (NRCA) (jwilen@nrca.net)

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~~.

Delete without substitution:

~~TABLE 1507.8~~
~~WOOD SHINGLE AND SHAKE INSTALLATION~~

ROOF ITEM	WOOD SHINGLES	WOOD SHAKES
~~1. Roof slope~~	~~Wood shingles shall be installed on slopes of not less than three units vertical in 12 units horizontal (3:12).~~	~~Wood shakes shall be installed on slopes of not less than four units vertical in 12 units horizontal (4:12).~~
~~2. Deck requirement~~		
~~Temperate climate~~	~~Shingles shall be applied to roofs with solid or spaced sheathing. Where spaced sheathing is used, sheathing boards shall be 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.~~	~~Shakes shall be applied to roofs with solid or spaced sheathing. Where spaced sheathing is used, sheathing boards shall be 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.~~
~~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.~~	~~Solid sheathing is required.~~	~~Solid sheathing is required.~~
~~3. Interlayment~~	~~No requirements.~~	~~Interlayment shall comply with ASTM D 226, Type 1.~~
~~4. Underlayment~~	–	–
~~Temperate climate~~	~~Underlayment shall comply with ASTM D 226, Type 1.~~	~~Underlayment shall comply with ASTM D 226, Type 1.~~
~~In areas where there is a possibility of ice forming along the eaves causing a backup of water.~~	~~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.~~	~~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 to a point at least 24 inches inside the exterior wall line of the building.~~
~~5. Application~~	–	–
~~Attachment~~	~~Fasteners for wood shingles shall be hot-dipped galvanized or Type 304 (Type 316 for coastal areas) stainless steel with a minimum penetration of 0.75 inch into the sheathing. For sheathing less than 0.5 inch thick, the fasteners shall extend through the sheathing.~~	~~Fasteners for wood shakes shall be hot-dipped galvanized or Type 304 (Type 316 for coastal areas) with a minimum penetration of 0.75 inch into the sheathing. For sheathing less than 0.5 inch thick, the fasteners shall extend through the sheathing.~~

~~No. of fasteners~~	~~Two per shingle.~~	~~Two per shake.~~
~~Exposure~~	~~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.~~
~~Method~~	~~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.~~	~~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.~~
~~Flashing~~	~~In accordance with Section 1507.8.8.~~	~~In accordance with Section~~ 1507.9.9.

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

Revise as follows:

1507.9 Wood shakes. The installation of wood shakes shall comply with the provisions of this section~~ and Table 1507.8~~.

Reason: The purpose of this change is to rectify conflicts that have resulted in Section 1507.8 and 1507.9 regarding w ood shingle and w ood shake roof systems.
In the final stages of development of the IBC, Table 1507.8-Wood Shingle and Shake Installation w as added as a summary of the installation specific requirements of Section 1507.8-Wood Shingles and Section 1507.9-Wood Shakes. With the IBC's 2000 Edition, the requirements in Table 1507.8 matched those of Section 1507.8 and Section 1507.9.

With the publication of IBC's 2003, 2006, 2009 and 2012 editions, changes have been made to the requirements in Section 1507.8 and Section 1507.9; how ever these same changes have not been consistently made to Table 1507.8. For example, in IBC 2012 new requirements for underlayment in high w ind regions w ere added in Section 1507.8.3.1 and Section 1507.9.3.1; these requirements w ere not added to Table 1507.8. There are a number of other similar examples. As a result, the requirements of Table 1507.8 are inconsistent and at times in conflict w ith those of Section 1507.8 and Section 1507.9.

Also, no requirements in Table 1507.8 w ill be lost due to the deletion of the table or the table's footnote. The Wood Shingle requirements in Table 1507.8 are located in IBC Sections 1507.8.1 and 8.1.1; 1507.8.2; 1507.8.3; 1507.8.4; 1507.8.6; 1507.8.7; and 1507.8.8. The Wood Shake requirements of Table 1507.8 are located in 1507.9.1 and 9.1.1; 1507.9.2; 1507.9.3; 1507.9.4, 1507.9.5; 1507.9.7; 1507.9.8; and 1507.9.9. Also the follow ing Subsections in Section 1507 have information that have been revised compared w ith text in Table 1507.8: 8.1; 8.3; 8.4; 8.6; 8.7; 9.1; 9.3; 9.4; 9.7; and 9.8. The table footnote contains only SI/Imperial conversions and its deletion has no bearing on code sections because conversions are already listed in each section w ere appropriate.

Deletion of Table 1507.8 and the pointers of the table in Section 1507.8 and Section 1507.9 eliminates the inconsistency and conflicts.

Cost Impact: Will not increase the cost of construction
The proposed change is a clarification and does not change the stringency of existing code requirements so the cost of construction w ill be unchanged.

S38-16 : 1507.8-WILEN AIA CDT RRO12446

S39-16

IBC: 1507.8.9 (New), 1507.9.9 (New).

Proponent : David Roodvoets, Cedar Shake and Shingle Buresu, representing Cedar Shake and Shingle Bureau (davelee@ix.netcom.com)

2015 International Building Code

Add new text as follows:

1507.8.9 Label Required Each bundle of shingles shall be identified by a label of an approved grading or inspection bureau or agency.

1507.9.9 Label Required Each bundle of shakes shall be identified by a label of an approved grading or inspection bureau or agency.

Reason: Labels need to be on the bundles of shakes and shingles so that building inspectors, contractors, architects, and ow ners can determine if the code compliant products are being installed.

Cost Impact: Will not increase the cost of construction
Most bundles of shakes and shingles are appropriately labeled now , how ever some non-compliant products do not have a label, leaving questions as to suitability w hen being installed. Labels are required by the Cedar Shake and Shingle Bureau now , so manufactures are already adding this minor cost to the distribution of the products.

S39-16 : 1507.8.9 (NEW)-ROODVOETS11968

S40-16

IBC: 1507.9.6, 1807.1.4, 2303.1.9.

Proponent : Colin McCown, representing American Wood Protection Association

2015 International Building Code

Revise as follows:

TABLE 1507.9.6
WOOD SHAKE MATERIAL REQUIREMENTS

MATERIAL	MINIMUM GRADES	APPLICABLE GRADING RULES
Wood shakes of naturally durable wood	1	CSSB
Taper sawn shakes of naturally durable wood	1 or 2	CSSB
Preservative-treated shakes and shingles of naturally durable wood	1	CSSB
Fire-retardant-treated shakes and shingles of naturally durable wood	1	CSSB
Preservative-treated taper sawn shakes of Southern pine treated in accordance with AWPA U1 (Commodity Specification A, Special Requirement 4.6~~Use Category 3B and Section 5.6~~)	1 or 2	TFS

CSSB = Cedar Shake and Shingle Bureau.

TFS = Forest Products Laboratory of the Texas Forest Services.

1807.1.4 Permanent wood foundation systems. Permanent wood foundation systems shall be designed and installed in accordance with AWC PWF. Lumber and plywood shall be preservative treated in accordance with AWPA U1 (Commodity Specification A,~~ Use Category 4B and Section 5.2~~ Special Requirement 4.2) and shall be identified in accordance with Section 2303.1.9.1.

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 permanent wood foundation systems shall conform to Chapter 18.

Reason: The existing text was outdated, requiring clarification and updates to current AWPA section numbering.

Cost Impact: Will not increase the cost of construction
These changes merely clarify and update the existing text without any impact on the required specifications for materials used.

S40-16 : TABLE 1507.9.6-MCCOWN13312

S41-16

Part I:
IBC: 1507.10.
Part II:
IRC: R905.9.

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC-BUILDING COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

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

Part I

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.

Part II

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.

S41-16 : 1507.10-FISCHER13535

S42-16

Part I:
IBC: 1507.11.
Part II:
IRC: R905.11.

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC-BUILDING COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

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

Part I

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.

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Part II

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 instrucitons.

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

S42-16 : 1507.11-
FISCHER13537

S43-16

Part I:
IBC: 1507.11.1, 1507.11.2, 1507.11.2.1 (New).
Part II:
IRC: R905.11.1, R905.11.2, R905.11.2.1 (New).

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC-BUILDING COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES

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

Part I

2015 International Building Code

Revise as follows:

1507.11.1 Slope. Modified bitumen ~~membrane roofs~~ roofing shall have a design slope of not less than one-fourth unit vertical in 12 units horizontal (2-percent slope) for drainage.

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

Add new text as follows:

1507.11.2.1 Base sheet. A base sheet that complies with the requirements of 1507.11.2, ASTM D 4601, or ASTM D 1970 shall be permitted to be used with a modified bitumen cap sheet.

Part II

2015 International Residential Code

Revise as follows:

R905.11.1 Slope. Modified bitumen ~~membrane roofs~~ roofing shall have a design slope of not less than one-fourth unit vertical in 12 units horizontal (2-percent slope) for drainage.

R905.11.2 Material standards. Modified bitumen ~~roof coverings~~ roofing shall comply with the standards in Table R905.11.2.

Add new text as follows:

R905.11.2.1 Base sheet. A base sheet that complies with the requirements of 1507.11.2, ASTM D 4601, or ASTM D 1970 shall be permitted to be used with a modified bitumen cap sheet.

Reason: The proposal includes a change to terminology for modified bitumen roofing materials for consistency with referenced standards and industry terminology. In addition, it includes a new section permitting the use of base sheet materials that comply with ASTM D 1970, ASTM D4601, or other standards as applicable.

Cost Impact: Will not increase the cost of construction
The proposal increases product options.

S43-16 : 1507.11.1-FISCHER13522

S44-16

IBC: 1507.14.2.

Proponent : Mike Fischer, Kellen, representing The Center for the Polyurethanes Industry of the American Chemistry Council (mfischer@kellencompany.com)

2015 International Building Code

Revise as follows:

1507.14.2 Material standards. Spray-applied polyurethane foam insulation shall comply with <u>ASTM C 1029 </u>Type III or IV~~ as defined in~~ <u>, or</u> ASTM ~~C 1029~~ <u>D7425</u>.

Reference standards type: This is an update to reference standard(s) already in the ICC Code Books

Add new standard(s) as follows:

ASTM D 7425/D 7425M—11 Standard Specification for Spray Polyurethane Foam Used for Roofing Application

Reason: The proposal adds in an alternate referenced standard applicable to spray-applied polyurethane foam insulation in roofing. ASTM D7425 is referenced in the 2015 IRC (Section R1905.14.2); it is added here for consistency.

Cost Impact: Will not increase the cost of construction
The proposal increases options for material suppliers; thus w ill increase product availability.

Analysis: The standard proposed for inclusion in this code, ASTM D7425, is referenced in the International Residential Code.

S44-16 : 1507.14.2-FISCHER3905

S45-16

IBC: 1507.10.2, 1507.14.3.

Proponent : Jason Wilen AIA CDT RRO, National Roofing Contractors Association (NRCA), representing National Roofing Contractors Association (NRCA) (jwilen@nrca.net)

2015 International Building Code

Revise as follows:

TABLE 1507.10.2
BUILT-UP ROOFING MATERIAL STANDARDS

MATERIAL STANDARD	STANDARD
~~Acrylic coatings used in roofing~~	~~ASTM D 6083~~
Aggregate surfacing	ASTM D 1863
Asphalt adhesive used in roofing	ASTM D 3747
Asphalt cements used in roofing	ASTM D 3019; D 2822; D 4586
Asphalt-coated glass fiber base sheet	ASTM D 4601
Asphalt coatings used in roofing	ASTM D 1227; D 2823; D 2824; D 4479
Asphalt glass felt	ASTM D 2178
Asphalt primer used in roofing	ASTM D 41
Asphalt-saturated and asphalt-coated organic felt base sheet	ASTM D 2626
Asphalt-saturated organic felt (perforated)	ASTM D 226
Asphalt used in roofing	ASTM D 312
Coal-tar cements used in roofing	ASTM D 4022; D 5643
Coal-tar saturated organic felt	ASTM D 227
Coal-tar pitch used in roofing	ASTM D 450; Type I or II
Coal-tar primer used in roofing, dampproofing and waterproofing	ASTM D 43

Glass mat, coal tar	ASTM D 4990
Glass mat, venting type	ASTM D 4897
Mineral-surfaced inorganic cap sheet	ASTM D 3909
Thermoplastic fabrics used in roofing	ASTM D 5665, D 5726

TABLE 1507.14.3
PROTECTIVE COATING MATERIAL STANDARDS

MATERIAL	STANDARD
~~Acrylic coating~~	~~ASTM D 6083~~
Silicone coating	ASTM D 6694
Moisture-cured polyurethane coating	ASTM D 6947

Reason: ASTM D 6083 has been withdrawn by ASTM and thus should no longer be referenced in the I-Codes.

Cost Impact: Will not increase the cost of construction
The proposed change is a clarification and does not change the stringency of existing code requirements so the cost of construction will be unchanged.

S45-16 : TABLE 1507.14.3-WILEN AIA CDT RRO9863

S46-16

IBC: 1507.15.3 (New).

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.

S46-16 : 1507.15.3 (NEW)-KIRBY13272

S47-16

IBC: 1507.17.2 (New).

Proponent : Lorraine Ross, representing The Dow Chemical Company (Intech@tampabay.rr.com)

2015 International Building Code

Add new text as follows:

1507.17.2 Deck Slope. *Photovoltaic shingles* shall be installed on roof slopes of not less than two units vertical in 12 units horizontal (2:12).

Reason: This proposal revises the minimum roof deck slope for the installation of photovoltaic shingles. The minimum of 2:12 slope is in conformance with an accepted slope for these products in the International Residential Code. The section was also edited for clarity.

Cost Impact: Will not increase the cost of construction
This proposal is a clarification of roof deck requirements for PV shingles. It does not increase the cost of construction.

S47-16 : 1507.17.2 (NEW)-ROSS13381

S48-16

IBC: 1507.18 (New), 1507.18.1 (New), 1507.18.2 (New), 1507.18.3 (New), 1507.18.4 (New), 1507.18.4.1 (New), 1507.18.4.2 (New), 1507.18.5 (New), 1507.18.6 (New), 1507.18.7 (New), 202 (New).

Proponent : Lorraine Ross, representing The Dow Chemical Company (Intech@tampabay.rr.com)

2015 International Building Code

Add new definition as follows:

SECTION 202 DEFINITIONS

BUILDING-INTEGRATED PHOTOVOLTAIC ROOF PANEL (BIPV ROOF PANEL) A *photovoltaic panel* that functions as a component of the building envelope.

Add new text as follows:

1507.18 Building-integrated (BIPV) roof panels applied directly to the roof deck.
The installation of *BIPV roof panels* shall comply with the provisions of this section.

1507.18.1 Deck requirements
BIPV roof panels shall be applied to a solid or closely-fitted deck, except where the roof covering is specifically designed to be applied over spaced sheatahing.

1507.18.2 Deck slope. *BIPV roof panels* shall be used only on roof slopes of two units vertical in 12 units (2:12) or greater.

1507.18.3 Underlayment. Unless otherwise noted, required underlayment shall conform to ASTM D226, ASTM D4869 or ASTM D6757.

1507.18.4 Underlayment application.
Underlayment shall be applied shingle fashion, parallel to and starting from the eave, lapped 2 inches (51 mm) and fastened sufficiently to hold in place.

1507.18.4.1 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 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 D226, Type III, ASTM D4869, Type IV or ASTM D6757. The 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) wiht a thickness of not less than 32-gage [0.0134 inch (0.34)] 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 with ASTM D1970 shall be permitted.

1507.18.4.2 Ice barrier. In areas where there has been a history of ice forming along the eaves causing a back-up of water, an ice barrier that consists of at least two layers of underlayment cemented together or of a self-adhering polymer modified bitumen sheet shall be used instead of normal underlayment and extend from the lowest edges of all roof surfaces to a point not less than 24 inches (610 mm) inside the exterior wall line of the building.

Exception: Detached accessory structures that contain no conditioned floor area.

1507.18.5 Material standards. *BIPV roof panels* shall be listed and labeled in accordance with UL 1703.

1507.18.6 Attachment. *BIPV roof panels* shall be attached in accordance with the manufacturer's installation instructions.

1507.18.7 Wind resistance. *BIPV roof panels* shall be tested in accordance with UL 1897. *BIPV roof panels* packaging shall bear a *label* to indicate complaince with UL 1897.

Reason: This proposal adds new sections to the IBC to address Building-integrated (BIPV) roof panels. These products form part of the building envelope and are subject to the same requirements as any other roof covering. As opposed to BIPV Shingles that are already regulated by the code, these BIPV panels are larger and the wind resistance is determined by UL 1897Uplift Tests for Roof Covering System. The overall proposal contains two parts:

- A new definition for BIPV Roof Panel is added to Chapter 2. The current IBC already contains a definition for photovoltaic panel and building-integrated photovoltaic product. This new definition is for a new type of building integrated photovoltaic product: **Building-integrated Photovoltaic (BIPV) Roof Panel**.
- A new section is added to Chapter 15 to detail the proper application of BIPV Roof Panels.The referenced wind resistance standard in new Section1507.18.7, UL 1897 Uplift Tests for Roof Covering System, is already recognized in Chapter 35 of the IBC.

Cost Impact: Will not increase the cost of construction
Will not increase the cost of construction. This proposal does not increase the cost of construction. It adds another type of roof covering, enhancing builder choices.

S48-16 : 1507.18 (NEW)-ROSS13333

S49-16

IBC: [BG] 1510.7.1.

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

2015 International Building Code

Delete without substitution:

~~**[BG] 1510.7.1 Wind resistance.** Rooftop-mounted *photovoltaic panels* and *modules* shall be designed for component and cladding wind loads in accordance with Chapter 16 using an effective wind area based on the dimensions of a single unit frame.~~

Reason: The expansion of Section 3111 in the International Building Code by Proposal G211-15 in the Group A cycle covers all that is within Section 1510.7 and its subsections, so Section 1501.7 is no longer needed. Further, ASCE 7-16 will include wind load calculation methods specific to rooftop solar photovoltaic systems, including PV systems installed parallel to a pitched roof in Section 29.4.4. This proposal to delete Section 1510.7 will remove conflicts with Section 3111 and ASCE 7-16.
2015 IBC Section 1510.7.1 (Section 1509.7.1 in 2012 IBC) requires the use of "an effective wind area based on the dimensions of a single unit frame." This requirement is inconsistent with ASCE 7.

In ASCE 7-10, Effective Wind Area is defined in Section 26.2 as follows:

> EFFECTIVE WIND AREA, A: The area used to determine (GCp). For component and cladding elements, the effective wind area in Figs. 30.4-1 through 30.4-7, 30.5-1, 30.6-1, and 30.8-1 through 30.8-3 is the span length multiplied by an effective width that need not be less than one-third the span length. For cladding fasteners, the effective wind area shall not be greater than the area that is tributary to an individual fastener.

In Wind loads on low profile solar photovoltaic systems on flat roofs (SEAOC-PV2-2012), the Structural Engineers Association of California includes a thorough discussion of effective wind area for photovoltaic panel systems within a commentary section of the paper. The closing statement of this commentary section in PV2 by SEAOC addresses the difference between IBC (2012 & 2015) and ASCE definitions:

> The requirements and commentary above differ from the provision of IBC 2012 Section 1509.7.1, which states, "Rooftop mounted photovoltaic systems shall be designed ... using an effective wind area based on the dimensions of a single unit frame." It is the consensus opinion of the SEAOC Solar Photovoltaic Systems Committee that this provision is not appropriate for many types of systems and parts of solar arrays. The provision can be un-conservative for a fastener with tributary area less than a "single unit frame" and is overly conservative for elements of a solar array, such as main supports or members that have a tributary area of several solar modules. The provision may also be overly conservative if applied to a framing member of a building supporting multiple attachments from a solar array.

In the 2013 California Building Code, the California Division of State Architect – Structural Safety (DSA-SS) revised 2012 IBC Section 1509.7.1 (which became 1510.7.1 in the 2015 IBC). This amendment was also adopted by the Building Standards Commission and Housing & Community Development. The amendment provided an exception to refer to IBC Chapter 16 and the definition of effective wind area in ASCE 7 Section 26.2, as follows:

> **1509.7 Photovoltaic systems.** Rooftop mounted photovoltaic systems shall be designed in accordance with this section.
>
> > **1509.7.1 Wind resistance.** Rooftop mounted photovoltaic systems shall be designed for wind loads for component and cladding in accordance with Chapter 16 using an effective wind area based on the dimensions of a single unit frame.
> >
> > ***Exception: [BSC, HCD-1, HCD-2, DSA-SS/CC]*** *The effective wind area shall be in accordance with Chapter 16 and ASCE 7 Section 26.2.*

To further support this topic, a proposal was submitted by SEAOC for ASCE 7-16 to specifically reference EWA for photovoltaic panel systems. ASCE 7-16 is still under development. Chapter 26 has been balloted successfully by the Wind Loads Subcommittee. In DRAFT ASCE 7-16, Effective Wind Area is defined in Section 26.2 as follows:

> [DRAFT] EFFECTIVE WIND AREA, A: The area used to determine the external pressure coefficient (GCp) and (GCrn). For C&C elements, the effective wind area in Figs. 30.3-1 through 30.3-7, 30.5-1, 30.5-1, and 30.7-1

through 30.7-3 is the span length multiplied by an effective width that need not be less than one-third the span length. For rooftop solar arrays, the effective wind area in Fig. 29.4-7 is equal to the tributary area for the structural element being considered, except that the width of the effective wind area need not be less than one-third its length. For cladding fasteners, the effective wind area shall not be greater than the area that is tributary to an individual fastener.

With the solar-specific definition of effective wind area in ASCE 7, there is no need for Section 1510.7.1.

Cost Impact: Will not increase the cost of construction
This proposal will not increase the cost of construction, as it is intended to coordinate the code with prior action approved under Group A code development.

Analysis: *Wind loads on low profile solar photovoltaic systems on flat roofs*, Report SEAOC-PV2-2012, Structural Engineers Association of California, August 2012.

S49-16 : [BG] 1510.7.1-CAIN11045

S50-16

IBC: 1511, 1511.1, 1511.2, 1511.3, 1511.3.1, 1511.3.1.1, 1511.4, 1511.5, 1511.6; IEBC: 706, [BS] 706.1, [BS] 706.2, [BS] 706.3, [BS] 706.4, [BS] 706.5, [BS] 706.6.

Proponent : Maureen Traxler (maureen.traxler@seattle.gov)

2015 International Building Code

Delete without substitution:

~~SECTION 1511 REROOFING~~

~~**1511.1 General.** Materials and methods of application used for recovering or replacing an existing roof covering shall comply with the requirements of Chapter 15~~
.

> ~~**Exceptions:**~~
> 1. ~~*Roof replacement* or *roof recover* of existing low-slope roof coverings shall not be required to meet the minimum design slope requirement of one-quarter unit vertical in 12 units horizontal (2-percent slope) in Section 1507 for roofs that provide positive roof drainage.~~
> 2. ~~Recovering or replacing an existing roof covering shall not be required to meet the requirement for secondary (emergency overflow) drains or scuppers in Section 1503.4 for roofs that provide for positive roof drainage. For the purposes of this exception, existing secondary drainage or scupper systems required in accordance with this code shall not be removed unless they are replaced by secondary drains or scuppers designed and installed in accordance with Section 1503.4.~~

~~**1511.2 Structural and construction loads.** Structural roof components shall be capable of supporting the roof-covering system and the material and equipment loads that will be encountered during installation of the system.~~

~~**1511.3 Roof replacement.** *Roof replacement* shall include the removal of all existing layers of roof coverings down to the roof deck.~~

> ~~**Exception:** Where the existing roof assembly includes an ice barrier membrane that is adhered to the roof deck, the existing ice barrier membrane shall be permitted to remain in place and covered with an additional layer of ice barrier membrane in accordance with Section 1507.~~

~~**1511.3.1 Roof recover.** The installation of a new roof covering over an existing roof covering shall be permitted where any of the following conditions occur:~~

1. ~~Where the new roof covering is installed in accordance with the roof covering manufacturer's approved instructions.~~
2. ~~Complete and separate roofing systems, such as standing-seam metal roof panel systems, that are designed to transmit the roof loads directly to the building's structural system and that do not rely on existing roofs and roof coverings for support, shall not require the removal of existing roof coverings.~~
3. ~~Metal panel, metal shingle and concrete and clay tile roof coverings shall be permitted to be installed over existing wood shake roofs when applied in accordance~~

~~with Section 1511.4.~~

~~4. The application of a new protective coating over an existing spray polyurethane foam roofing system shall be permitted without tear off of existing roof coverings.~~

~~**1511.3.1.1 Exceptions.** A *roof recover* shall not be permitted where any of the following conditions occur:~~

~~1. Where the existing roof or roof covering is water soaked or has deteriorated to the point that the existing roof or roof covering is not adequate as a base for additional roofing.~~
~~2. Where the existing roof covering is slate, clay, cement or asbestos-cement tile.~~
~~3. Where the existing roof has two or more applications of any type of roof covering.~~

~~**1511.4 Roof recovering.** Where the application of a new roof covering over wood shingle or shake roofs creates a combustible concealed space, the entire existing surface shall be covered with gypsum board, mineral fiber, glass fiber or other *approved* materials securely fastened in place.~~

~~**1511.5 Reinstallation of materials.** Existing slate, clay or cement tile shall be permitted for reinstallation, except that damaged, cracked or broken slate or tile shall not be reinstalled. Existing vent flashing, metal edgings, drain outlets, collars and metal counterflashings shall not be reinstalled where rusted, damaged or deteriorated. Aggregate surfacing materials shall not be reinstalled.~~

~~**1511.6 Flashings.** Flashings shall be reconstructed in accordance with *approved* manufacturer's installation instructions. Metal flashing to which bituminous materials are to be adhered shall be primed prior to installation.~~

2015 International Existing Building Code

Revise as follows:

~~706~~ 303 ~~706~~ 303 REROOFING

[BS] ~~706.1~~ 303.1 General. Materials and methods of application used for recovering or replacing an existing roof covering shall comply with the requirements of Chapter 15 of the *International Building Code*.

> **Exception:**Reroofing shall not be required to meet the minimum design slope requirement of one-quarter unit vertical in 12 units horizontal (2-percent slope) in Section 1507 of the *International Building Code*for roofs that provide positive roof drainage.

[BS] ~~706.2~~ 303.2 Structural and construction loads. *No change to text.*

[BS] ~~706.3~~ 303.3 Recovering versus replacement. New roof coverings shall not be installed without first removing all existing layers of roof coverings down to the roof deck where any of the following conditions occur:

1. Where the existing roof or roof covering is water soaked or has deteriorated to the point that the existing roof or roof covering is not adequate as a base for additional roofing.
2. Where the existing roof covering is wood shake, slate, clay, cement or asbestos-cement tile.

3. Where the existing roof has two or more applications of any type of roof covering.

Exceptions:

1. Complete and separate roofing systems, such as standing-seam metal roof systems, that are designed to transmit the roof loads directly to the building's structural system and that do not rely on existing roofs and roof coverings for support, shall not require the removal of existing roof coverings.
2. Metal panel, metal shingle and concrete and clay tile roof coverings shall be permitted to be installed over existing wood shake roofs when applied in accordance with Section 706.4.
3. The application of a new protective coating over an existing spray polyurethane foam roofing system shall be permitted without tear-off of existing roof coverings.
4. Where the existing roof assembly includes an ice barrier membrane that is adhered to the roof deck, the existing ice barrier membrane shall be permitted to remain in place and covered with an additional layer of ice barrier membrane in accordance with Section 1507 of the *International Building Code*.

[BS] ~~706.4~~ 303.4 Roof recovering. *No change to text.*

[BS] ~~706.5~~ 303.5 Reinstallation of materials. *No change to text.*

[BS] ~~706.6~~ 303.6 Flashings. *No change to text.*

Reason: This proposal removes the section on reroofing from the IBC. Reroofing is an alteration to an existing building which is within the scope of the IEBC rather than the IBC. We propose adding it as Section 303 because the prescriptive method relies on the IBC for reroofing regulations, and the work area method merely reprints the IBC section. If repeating the reroofing provisions in each of the compliance methods is preferred to locating them in Chapter 3, they could remain in Section 706 for the work area method, and be added to a new Section 411 for the prescriptive method.

Note that we submitted another proposal to update IEBC Section 706 to include the current IBC reroofing provisions. The intent of this proposal is that, If both are approved, the changes in the other proposal be applied to the IEBC reroofing section.

Cost Impact: Will not increase the cost of construction

This proposal relocates existing code language and will not affect the cost of construction.

S50-16 : 1511-TRAXLER11931

S51-16

Part I:

IBC: 1511.3.1, 202 (New); IEBC: , [BS] 706.3.

Part II:

IRC: R202 (New), R908.3.1.

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC-STRUCTURAL CODE COMMITTEE. PART II WILL BE HEARD BY THE IRC-BUILDING CODE COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

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

Part I

2015 International Building Code

Revise as follows:

1511.3.1 Roof recover. The installation of a new roof covering over an existing roof covering shall be permitted where any of the following conditions occur:

1. Where the new roof covering is installed in accordance with the roof covering manufacturer's approved instructions.
2. Complete and separate roofing systems, such as standing-seam metal roof panel systems, that are designed to transmit the roof loads directly to the building's structural system and that do not rely on existing roofs and roof coverings for support, shall not require the removal of existing roof coverings.
3. Metal panel, metal shingle and concrete and clay tile roof coverings shall be permitted to be installed over existing wood shake roofs when applied in accordance with Section 1511.4.
4. The application of a new protective *roof* coating over an existing protective *roof coating*, metal roof panel, metal roof shingle, mineral surfaced roll roofing, built-up roof, modified bitumen roofing, thermoset and thermoplastic single-ply roofing and spray polyurethane foam roofing system shall be permitted without tear off of existing roof coverings.

Add new definition as follows:

SECTION 202 DEFINITIONS

ROOF COATING. A fluid-applied and fully adhered coating used for roof maintenance, *roof repair*, or as a component of a *roof covering system* or *roof assembly.*

2015 International Existing Building Code

Add new definition as follows:

SECTION 202 DEFINITIONS

ROOF COATING. A fluid-applied and fully adhered coating used for roof maintenance, *roof repair*, or as a component of a *roof covering system* or *roof assembly*

Revise as follows:

[BS] 706.3 Recovering versus replacement. New roof coverings shall not be installed without first removing all existing layers of roof coverings down to the roof deck where any of the following conditions occur:

1. Where the existing roof or roof covering is water soaked or has deteriorated to the point that the existing roof or roof covering is not adequate as a base for additional roofing.
2. Where the existing roof covering is wood shake, slate, clay, cement or asbestos-cement tile.
3. Where the existing roof has two or more applications of any type of roof covering.

Exceptions:

1. Complete and separate roofing systems, such as standing-seam metal roof systems, that are designed to transmit the roof loads directly to the building's structural system and that do not rely on existing roofs and roof coverings for support, shall not require the removal of existing roof coverings.
2. Metal panel, metal shingle and concrete and clay tile roof coverings shall be permitted to be installed over existing wood shake roofs when applied in accordance with Section 706.4.
3. The application of a new protective *roof* coating over an existing protective *roof coating*, metal roof panel, metal roof shingle, mineral surfaced roll roofing, built-up roof, modified bitumen roofing, thermoset and thermoplastic single-ply roofing and spray polyurethane foam roofing system shall be permitted without ~~tear-off~~ tear off of existing roof coverings.
4. Where the existing roof assembly includes an ice barrier membrane that is adhered to the roof deck, the existing ice barrier membrane shall be permitted to remain in place and covered with an additional layer of ice barrier membrane in accordance with Section 1507 of the *International Building Code*.
5.

Part II

2015 International Residential Code

Revise as follows:

R908.3.1 Roof re-cover. The installation of a new roof covering over an existing roof covering shall be permitted where any of the following conditions occur:

1. Where the new roof covering is installed in accordance with the roof covering manufacturer's approved instructions
2. Complete and separate roofing systems, such as standing-seam metal roof systems, that are designed to transmit the roof loads directly to the building's structural system and do not rely on existing roofs and roof coverings for support, shall not require the removal of existing roof coverings.
3. Metal panel, metal shingle and concrete and clay tile roof coverings shall be permitted to be installed over existing wood shake roofs where applied in accordance with Section R908.4.

4. The application of a new protective *roof* coating over an existing protective *roof coating*, metal roof panel, metal roof shingle, mineral surfaced roll roofing, built-up roof, modified bitumen roofing, thermoset and thermoplastic single-ply roofing and spray polyurethane foam roofing system shall be permitted without ~~tear-off~~ tear off of existing roof coverings.
5.

Add new definition as follows:

SECTION R202 DEFINITIONS

ROOF COATING. A fluid-applied and fully adhered coating used for roof maintenance, *roof repair*, or as a component of a *roof covering system* or *roof assembly*

Reason: The proposal clarifies the enforcement process by adding additional substrates (i.e., existing roof coverings) that are acceptable for the installation of a roof coating.

Cost Impact: Will not increase the cost of construction
This will likely reduce the cost of construction when questions are raised about the appropriate use of a roof coating on existing roofs.

S51-16 : 1511.3.1-KIRBY13260

S52-16

IBC: 1512.2 (New).

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

2015 International Building Code

Add new text as follows:

1512.2 Ballasted photovoltaic panel systems. Ballasted photovoltaic panel systems installed on low-slope roof assemblies shall be secured to eliminate movement/slippage at design wind speeds determined in accordance with Section 1609.

Reason: Manufacturers of single ply roof membranes have experienced damage to their roof membranes due to slippage/movement of roof mounted ballasted photovoltaic modules. For this reason SPRI is requesting an addition to the code to require that these systems be secured to prevent slippage and movement at design w ind speeds. Follow ing are some examples of this issue.

Figure 1 Movement of the w ood blocking and trays carrying the pow er lines


Figure 2 Outline of the protection layer and blocking indicate movement of the tray carrying pow er line.

Figure 3 Entire PV array sliding into a davit post

Figure 4 PV array sliding off protection layer

Figure 5 PV array sliding over 12-inches w ith corner area over the gutter

Cost Impact: Will increase the cost of construction
This proposal w ould require that ballasted PV systems be secured to prevent slippage/movement w hen exposed to design w ind speeds. This may increase the cost of construction.

S52-16 : 1512.2 (NEW)-ENNIS12600

S53-16

1602.1

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~~ 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 Associatied 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).

S53-16 : 1602.1-GOUPIL11451

S54-16

IBC: 1602.1, 1603.1, 1603.1.4, 1609.1.1, 1609.1.1.1, 1609.1.2.2, 1609.2, 1609.3, 1609.3.1, 1609.4, 1609.4.1, 1609.4.2, 1609.4.3.

Proponent : Ronald Hamburger, SIMPSON GUMPERTZ & HEGER (rohamburger@sgh.com)

2015 International Building Code

Revise as follows:

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

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 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.
V_{asd}	=	~~Nominal~~ Allowable stress design wind speed ~~(3-second gust)~~, miles per hour (mph) (km/hr) where applicable.
$V_{\cancel{ult}}$	=	~~Ultimate~~ Basic 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~~ in accordance with ASCE 7.
W	=	Load due to wind pressure.
W_i	=	Wind-on-ice in accordance with Chapter 10 of ASCE 7.

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_g.
> 3. ~~Ultimate~~ Basic design wind speed, $V_{\cancel{ult}}$, ~~(3-second gust),~~ miles per hour (mph) (km/hr) and ~~nominal~~ allowable stress design wind speed, V_{asd}, 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.

1603.1.4 Wind design data. The following information related to wind loads shall be shown, regardless of whether wind loads govern the design of the lateral force-resisting system of the structure:

1. ~~Ultimate~~ Basic design wind speed, $V_{\cancel{ult}}$~~, (3-second gust)~~, miles per hour (km/hr) and ~~nominal~~ allowable stress design wind speed, V_{asd}, as determined in accordance with Section 1609.3.1.
2. *Risk category*.
3. Wind exposure. Applicable wind direction if more than one wind exposure is utilized.
4. Applicable internal pressure coefficient.
5. Design wind pressures to be used for exterior component and cladding materials not specifically designed by the *registered design professional*responsible for the design of the structure, psf (kN/m^2).

1609.1.1 Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapters 26 to ~~30~~ 31 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, $V_{\cancel{ult}}$, 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.
4. Designs using NAAMM FP 1001.
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~~ Where alternative wind speeds and ~~Sections 31.4~~ or exposures are estabished by the *building official*, those wind speeds and ~~31.5 of ASCE 7~~ exposures shall be used.

The wind speeds in~~ Figures 1609.3(1), 1609.3(2) and 1609.3(3)~~ Chapter 26 of ASCE 7 are ~~ultimate~~ basic design wind speeds, $V_{\cancel{ult}}$, and shall be converted in accordance with Section 1609.3.1 to ~~nominal~~ allowable stresss design wind speeds, V_{asd}, when the provisions of the standards referenced in Exceptions 4 and 5 are used.

1609.1.1.1 Applicability. The provisions of ICC 600 are applicable only to buildings located within Exposure B or C as defined in ~~Section 1609.4~~ Chapter 26 of ASCE 7. The provisions of ICC 600, AWC WFCM and AISI S230 shall not apply to buildings sited on the upper half of an isolated hill, ridge or escarpment meeting the following conditions:

1. The hill, ridge or escarpment is 60 feet (18 288 mm) or higher if located in Exposure B or 30 feet (9144 mm) or higher if located in Exposure C;
2. The maximum average slope of the hill exceeds 10 percent; and
3. The hill, ridge or escarpment is unobstructed upwind by other such topographic

features for a distance from the high point of 50 times the height of the hill or ~~1 mile~~ 2 miles (~~1.61~~ 3.22 km), whichever is greater.

1609.1.2.2 Application of ASTM E 1996. The text of Section 6.2.2 of ASTM E 1996 shall be substituted as follows:
6.2.2 Unless otherwise specified, select the wind zone based on the ~~strength~~ basic design wind speed, V_{ult}, as follows:
6.2.2.1 *Wind Zone 1*—130 mph ≤ ~~ultimate~~ basic design wind speed, V_{ult}< 140 mph.
6.2.2.2 *Wind Zone 2*—140 mph ≤ ~~ultimate~~ basic design wind speed, V_{ult}< 150 mph at greater than one mile (1.6 km) from the coastline. The coastline shall be measured from the mean high water mark.
6.2.2.3 *Wind Zone 3*—150 mph (58 m/s) ≤ ~~ultimate~~ basic design wind speed, V_{ult} ≤ 160 mph (63 m/s), or 140 mph (54 m/s) ≤ ~~ultimate~~ basic design wind speed, V_{ult} ≤ 160 mph (63 m/s) and within one mile (1.6 km) of the coastline. The coastline shall be measured from the mean high water mark.
6.2.2.4 *Wind Zone 4*— ~~ultimate~~ basic design wind speed, V_{ult} >160 mph (63 m/s).

1609.2 Definitions. For the purposes of Section 1609 and as used elsewhere in this code, the following terms are defined in Chapter 2.

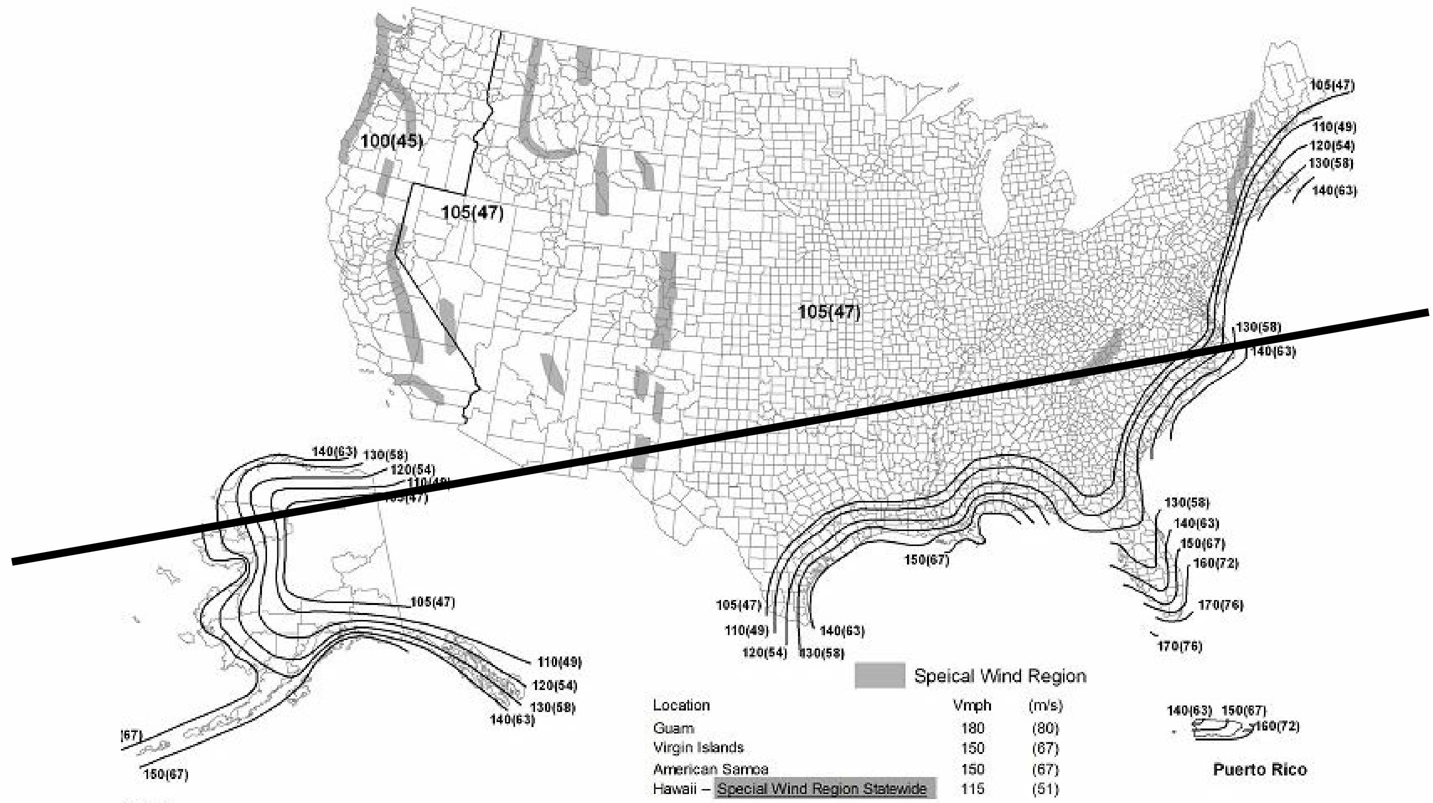
HURRICANE-PRONE REGIONS.
WIND-BORNE DEBRIS REGION.
WIND SPEED, ~~V_{ult}.~~
WIND SPEED, V_{asd}.

1609.3 ~~Ultimate~~ Basic design wind speed. The ~~ultimate~~ basic design wind speed, V_{ult}, in mph, for the determination of the wind loads shall be determined ~~by Figures 1609.3(1), 1609.3(2) and 1609.3(3). The ultimate design wind speed, V_{ult}, for use in the design of Risk Category II buildings and structures shall be obtained from Figure 1609.3(1). The ultimate design wind speed, V_{ult}, for use in the design of Risk Category III and IV buildings and structures shall be obtained from Figure 1609.3(2). The ultimate design wind speed, V_{ult}, for use in the design of Risk Category I buildings and structures shall be obtained from Figure 1609.3(3). The ultimate design wind speed, V_{ult}, for the special wind regions indicated near mountainous terrain and near gorges shall be~~ in accordance with ~~local jurisdiction requirements. The ultimate design wind speeds, V_{ult}, determined by the local jurisdiction shall be in accordance with Section 26.5.1~~ Chapter 26 of ASCE 7.

~~In nonhurricane-prone regions, when the ultimate design wind speed, V_{ult}, is estimated from regional climatic data, the ultimate design wind speed, V_{ult}, shall be determined in accordance with Section 26.5.3 of ASCE 7.~~

Delete without substitution:

~~1609.3~~ (3)
~~ULTIMATE DESIGN WIND SPEEDS, v_{ult}, FOR RISK CATEGORY I BUILDINGS AND OTHER STRUCTURES~~

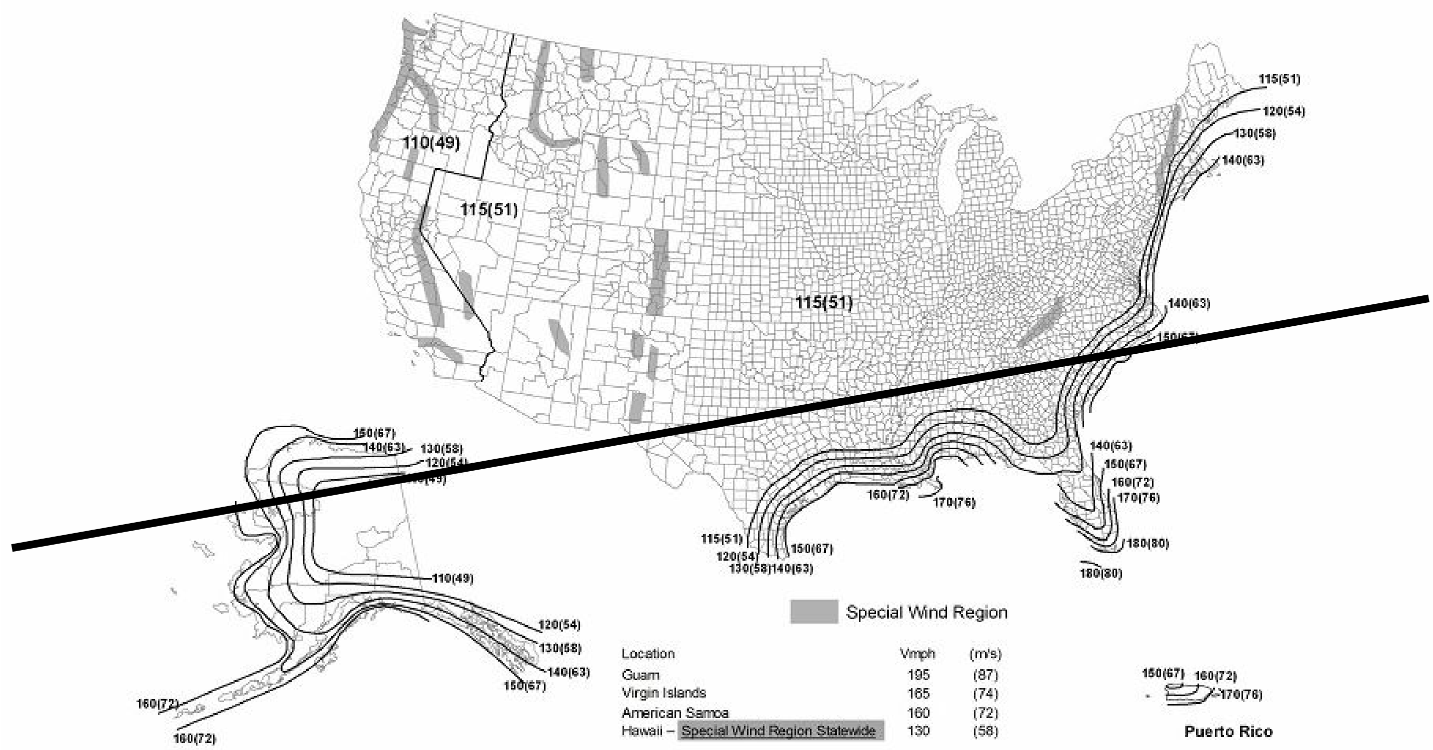


Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 15% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00333, MRI = 300 Years).

~~1609.3~~ (1)

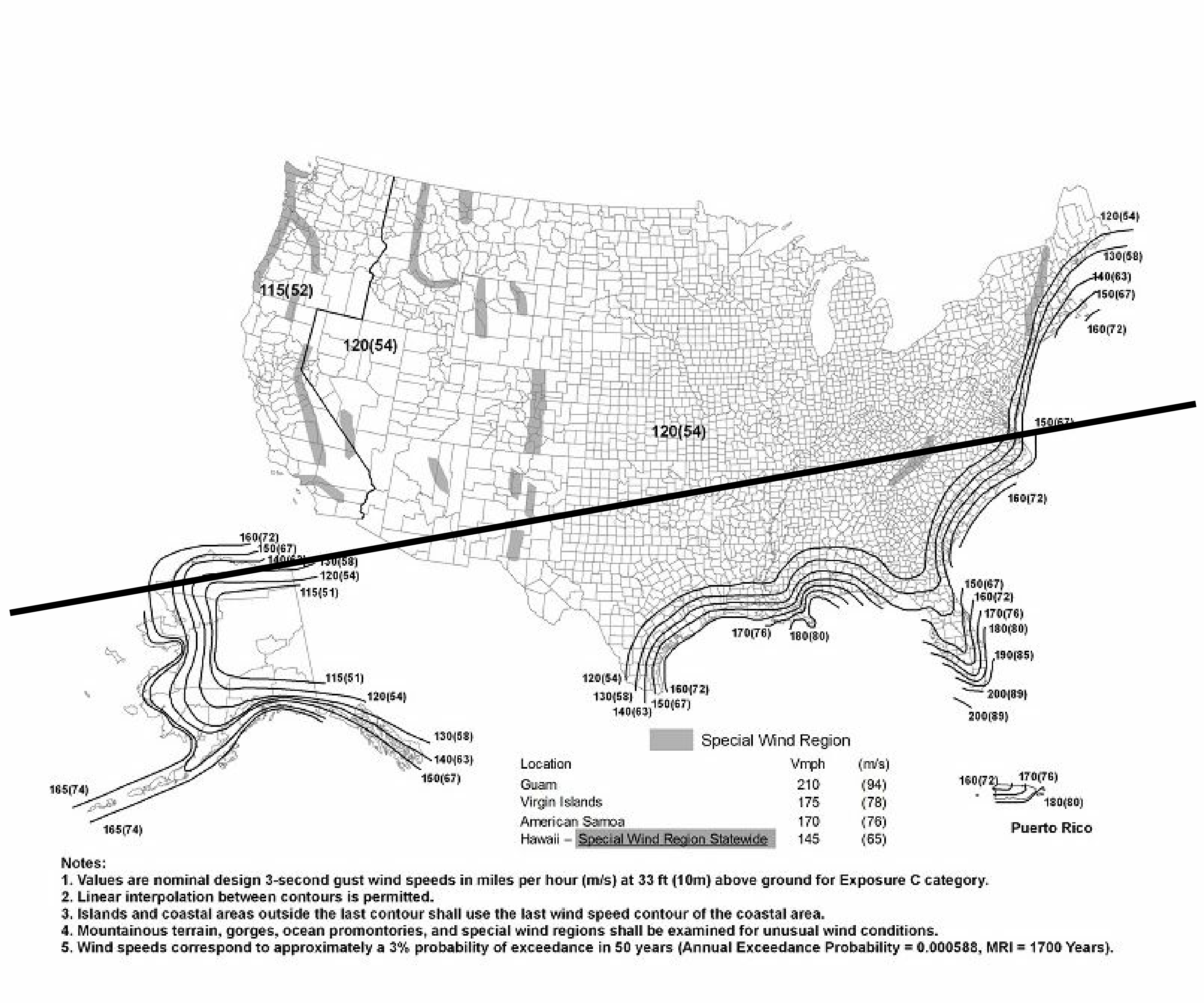
~~ULTIMATE DESIGN WIND SPEEDS, V_{ult}, FOR RISK CATEGORY II BUILDINGS AND OTHER STRUCTURES~~



Notes:
1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 Years).

~~1609.3~~ (2)

~~ULTIMATE DESIGN WIND SPEEDS, V_{ult}, FOR RISK CATEGORY III AND IV BUILDINGS AND OTHER STRUCTURES~~



Revise as follows:

1609.3.1 Wind speed conversion. When required, the ~~ultimate~~ basic design wind speeds of ~~Figures 1609.3(1), 1609.3(2) and 1609.3(3)~~ Chapter 26 of ASCE 7 shall be converted to ~~nominal~~ allowable stress design wind speeds, V_{asd}, using Table 1609.3.1~~ or Equation 16-33~~.

~~**(Equation 16-33)**~~

~~where:~~

~~V_{asd}~~	~~=~~	~~Nominal design wind speed applicable to methods specified in Exceptions 4 and 5 of Section 1609.1.1.~~
~~V_{ult}~~	~~=~~	~~Ultimate design wind speeds determined from Figures 1609.3(1), 1609.3(2) or 1609.3(3).~~

TABLE 1609.3.1
WIND SPEED CONVERSIONS[a, b, c]

V_{ult}	100	110	120	130	140	150	160	170	180	190	200

V_{asd}	78	85	93	101	108	116	124	132	139	147	155

For SI: 1 mile per hour = 0.44 m/s.

a. Linear interpolation is permitted.

b. V_{asd} = ~~nominal~~ allowable stress design wind speed applicable to methods specified in Exceptions 1 through 5 of Section 1609.1.1.

c. $V_{\cancel{ult}}$ = ~~ultimate~~ basic design wind speeds determined from ~~Figure 1609.3(1), 1609.3(2) or 1609.3(3)~~ Chapter 26 of ASCE 7.

Delete without substitution:

~~**1609.4 Exposure category.** For each wind direction considered, an exposure category that adequately reflects the characteristics of ground surface irregularities shall be determined for the site at which the building or structure is to be constructed. Account shall be taken of variations in ground surface roughness that arise from natural topography and vegetation as well as from constructed features.~~

~~**1609.4.1 Wind directions and sectors.** For each selected wind direction at which the wind loads are to be evaluated, the exposure of the building or structure shall be determined for the two upwind sectors extending 45 degrees (0.79 rad) either side of the selected wind direction. The exposures in these two sectors shall be determined in accordance with Sections 1609.4.2 and 1609.4.3 and the exposure resulting in the highest wind loads shall be used to represent winds from that direction.~~

~~**1609.4.2 Surface roughness categories.** A ground surface roughness within each 45-degree (0.79 rad) sector shall be determined for a distance upwind of the site as defined in Section 1609.4.3 from the categories defined below, for the purpose of assigning an exposure category as defined in Section 1609.4.3.~~

~~**Surface Roughness B.**~~

~~Urban and suburban areas, wooded areas or other terrain with numerous closely spaced obstructions having the size of single family dwellings or larger.~~

~~**Surface Roughness C.**~~

~~Open terrain with scattered obstructions having heights generally less than 30 feet (9144 mm). This category includes flat open country, and grasslands.~~

~~**Surface Roughness D.**~~

~~Flat, unobstructed areas and water surfaces. This category includes smooth mud flats, salt flats and unbroken ice.~~

~~**1609.4.3 Exposure categories.** An exposure category shall be determined in accordance with the following:~~

~~**Exposure B.**~~

~~For buildings with a mean roof height of less than or equal to 30 feet (9144 mm), Exposure B shall apply where the ground surface roughness, as defined by Surface Roughness B, prevails in the upwind direction for a distance of at least 1,500 feet (457 m). For buildings with a mean roof height greater than 30 feet (9144 mm), Exposure B shall apply where Surface Roughness B prevails in the upwind direction for a distance of at least 2,600 feet (792 m) or 20 times the height of the building, whichever is greater.~~

~~**Exposure C.**~~

~~Exposure C shall apply for all cases where Exposure B or D does not apply.~~

~~Exposure D.~~

~~Exposure D shall apply where the ground surface roughness, as defined by Surface Roughness D, prevails in the upwind direction for a distance of at least 5,000 feet (1524 m) or 20 times the height of the building, whichever is greater. Exposure D shall also apply where the ground surface roughness immediately upwind of the site is B or C, and the site is within a distance of 600 feet (183 m) or 20 times the building height, whichever is greater, from an Exposure D condition as defined in the previous sentence.~~

Reason: During the development of the ASCE 7-16 standard the ASCE 7 Wind Loads Subcommittee made substantial revisions to the w ind speed maps contained in the standard. These revisions include the development of separate maps for Risk Categories III and IV structures; reconstruction of the special w ind regions w ithin the maps, correcting know n deficiencies in the w ind speed contours; and modification of the w ind speeds based on updated climactic and w eather data. New hurricane counters in the northeastern states w ere developed based on updated hurricane models and the locations of the contours along the hurricane coastline w ere adjusted to reflect new research into the decay rate of hurricanes over land. New maps for the State of Haw aii w ere developed to eliminate it as a "special w ind region" and to provide guidance on the w ind patterns for the state that occur because of the unique topography there. Currently there are eight new maps for main w ind force and component and cladding design in the ASCE 7-16 standard along w ith four new serviceability maps.
Rather than place all of these updated maps in the 2018 IBC, to replace th three outdated maps contained in the 2015 IBC, and w ith the removal of the Alternative all-heights w ind pressure calculation method from the IBC, these proposed changes w ill refer to the ASCE 7 standard directly fo all w ind load design information, including the new updated maps.

Complementing the publication of the 2016 edition of *ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, the w ind maps and associated data w ill be made available separately from purchasing the full standard so that building officials and others w ill still have access to this specific data.

Cost Impact: Will not increase the cost of construction
The proposed changes w ill not impact the cost of construction and may decrease the cost of construction in the majority of the United States that is not subjected to hurricane w ind events. For the majority of the United States the basic design w ind speeds have been low ered in the new maps based on the latest data available, thus reducing the cost of construction. Along the hurricane coastline the w ind speeds remain unchanged from the current maps and thus the cost of construction w ill not increase.
This proposal is a re-organization of the pointers in the IBC to refer to the w ind provision in the referenced loading standard ASCE 7. *ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures* w ill 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).

S54-16 : 1602.1-HAMBURGER11648

S55-16

IBC: 1602.1, 1605.2, 1605.3, 1605.3.1, 1607.14.2 (New).

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:
<u>L_f = Fire wall horizontal live load.</u>

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)**
$1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$	**(Equation 16-2)**
$1.2(D + F) + 1.6(L_r \text{ or } S \text{ or } R) + 1.6H + (f_1 L \text{ or } 0.5W)$	**(Equation 16-3)**
$1.2(D + F) + 1.0W + f_1 L + 1.6H + 0.5(L_r \text{ or } S \text{ or } R)$	**(Equation 16-4)**
$1.2(D + F) + 1.0E + f_1 L + 1.6H + f_2 S$	**(Equation 16-5)**
$0.9D + 1.0W + 1.6H$	**(Equation 16-6)**
$0.9(D + F) + 1.0E + 1.6H$	**(Equation 16-7)**
<u>$0.9D + 1.6L_f + 1.6H$</u>	<u>**(Equation 16-8)**</u>

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

$D + H + F + 0.75\,(0.7\,E) + 0.75\,L + 0.75\,S$ **(Equation 16-14)**

$0.6\,D + 0.6\,W + H$ **(Equation 16-15)**

$0.6(D + F) + 0.7\,E + H$ **(Equation 16-16)**

<u>0.6D + Lf + H</u> <u>**(Equation 16-18)**</u>

Exceptions:

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

Add new text as follows:

<u>**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/m^2).</u>

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.

S55-16 : 1602.1-HUSTON13355

S56-16

IBC: , 1602.1, 1603.1, 1603.1.4, 1609.1.1, 1609.1.1.1, 1609.1.2.2, 1609.2, 1609.3, 1609.3(3) (New), 1609.3(5) (New), 1609.3(6) (New), 1609.3(7) (New), 1609.3(8) (New), 1609.3.1.

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

2015 International Building Code

Revise as follows:

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

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 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.
V_{asd}	=	~~Nominal~~ Allowable stress design wind speed ~~(3-second gust)~~, miles per hour (mph) (km/hr) where applicable.
$V_{\cancel{ult}}$	=	~~Ultimate~~ Basic design wind speeds ~~(3-second gust)~~, miles per hour (mph) (km/hr) determined from Figure 1609.3(1)~~, 1609.3(2), 1609.3(3~~ to 1609.3(8) or ASCE 7.
W	=	Load due to wind pressure.
W_i	=	Wind-on-ice in accordance with Chapter 10 of ASCE 7.

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_g.
> 3. ~~Ultimate~~ Basic design wind speed, $V_{\cancel{ult}}$, ~~(3-second gust),~~ miles per hour (mph) (km/hr) and ~~nominal~~ allowable stress design wind speed, V_{asd}, 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.

1603.1.4 Wind design data. The following information related to wind loads shall be shown, regardless of whether wind loads govern the design of the lateral force-resisting system of the structure:

1. ~~Ultimate~~ Basic design wind speed, $V_{\cancel{ult}}$, ~~(3-second gust),~~ _miles per hour (km/hr) and ~~nominal~~ allowable stress design wind speed, V_{asd}, as determined in accordance with Section 1609.3.1.
2. *Risk category*.
3. Wind exposure. Applicable wind direction if more than one wind exposure is utilized.
4. Applicable internal pressure coefficient.
5. Design wind pressures to be used for exterior component and cladding materials not specifically designed by the *registered design professional*responsible for the design of the structure, psf (kN/m^2).

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~~ basic design wind speed, $V_{\cancel{ult}}$, 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.
4. Designs using NAAMM FP 1001.
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.

The wind speeds in Figures 1609.3(1)~~, 1609.3(2) and 1609.3(3~~ thorough 1609.3(8) are ~~ultimate~~ basic design wind speeds, $V_{\cancel{ult}}$, and shall be converted in accordance with Section 1609.3.1 to ~~nominal~~ allowable stress design wind speeds, V_{asd}, when the provisions of the standards referenced in Exceptions 4 and 5 are used.

1609.1.1.1 Applicability. The provisions of ICC 600 are applicable only to buildings located within Exposure B or C as defined in Section 1609.4. The provisions of ICC 600, AWC WFCM and AISI S230 shall not apply to buildings sited on the upper half of an isolated hill, ridge or escarpment meeting the following conditions:

1. The hill, ridge or escarpment is 60 feet (18 288 mm) or higher if located in Exposure B or 30 feet (9144 mm) or higher if located in Exposure C;

2. The maximum average slope of the hill exceeds 10 percent; and
3. The hill, ridge or escarpment is unobstructed upwind by other such topographic features for a distance from the high point of 50 times the height of the hill or ~~1 mile~~ 2 miles (~~1.61~~ 3.22 km), whichever is greater.

1609.1.2.2 Application of ASTM E 1996. The text of Section 6.2.2 of ASTM E 1996 shall be substituted as follows:
6.2.2 Unless otherwise specified, select the wind zone based on the ~~strength~~ basic design wind speed, $V_{\cancel{ult}}$, as follows:
6.2.2.1 *Wind Zone 1*—130 mph ≤ ~~ultimate~~ basic design wind speed, $V_{\cancel{ult}}$ < 140 mph.
6.2.2.2 *Wind Zone 2*—140 mph ≤ ~~ultimate~~ basic design wind speed, $V_{\cancel{ult}}$ < 150 mph at greater than one mile (1.6 km) from the coastline. The coastline shall be measured from the mean high water mark.
6.2.2.3 *Wind Zone 3*—150 mph (58 m/s) ≤ ~~ultimate~~ basic design wind speed, $V_{\cancel{ult}}$ ≤ 160 mph (63 m/s), or 140 mph (54 m/s) ≤ ~~ultimate~~ basic design wind speed, $V_{\cancel{ult}}$ ≤ 160 mph (63 m/s) and within one mile (1.6 km) of the coastline. The coastline shall be measured from the mean high water mark.
6.2.2.4 *Wind Zone 4*— ~~ultimate~~ basic design wind speed, $V_{\cancel{ult}}$ >160 mph (63 m/s).

1609.2 Definitions. For the purposes of Section 1609 and as used elsewhere in this code, the following terms are defined in Chapter 2.

HURRICANE-PRONE REGIONS.
WIND-BORNE DEBRIS REGION.
BASIC WIND SPEED, $V_{\cancel{ult}}$.
ALLOWABLE STRESS WIND SPEED, V_{asd}.

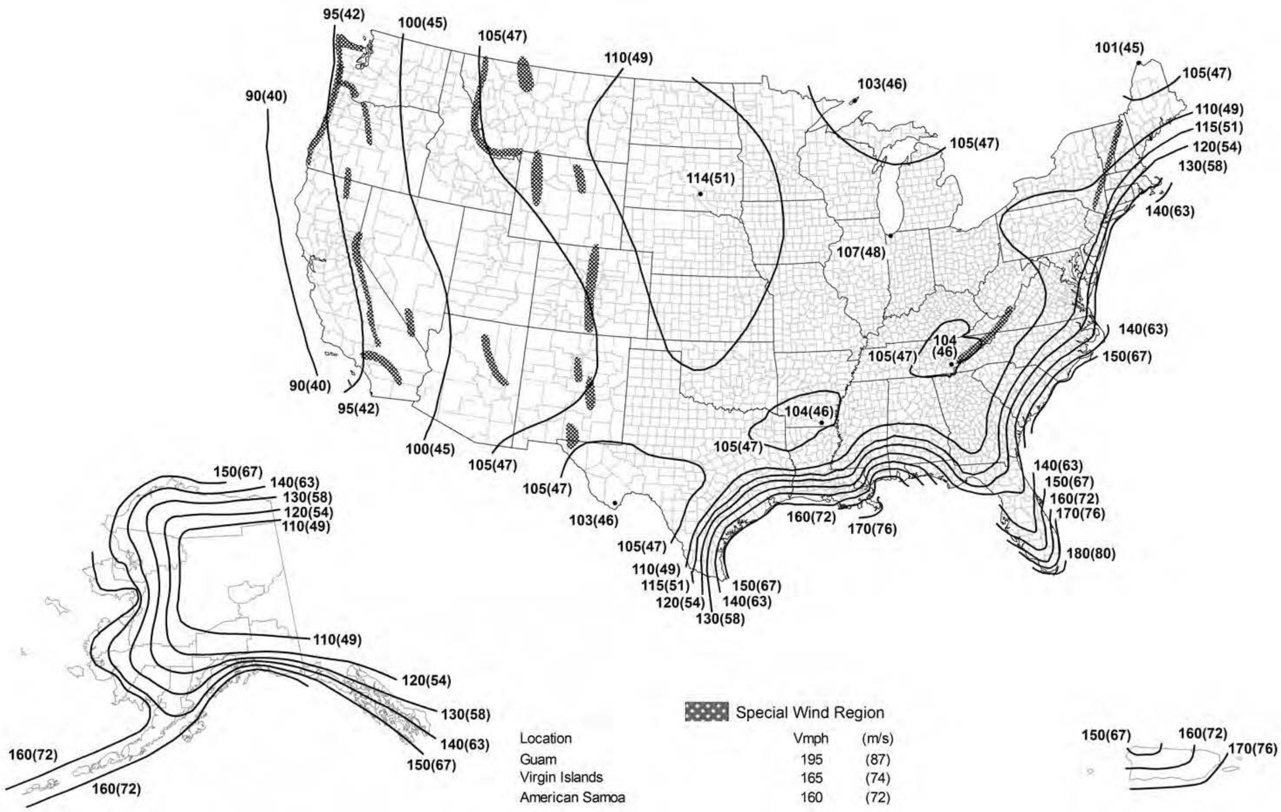
1609.3 ~~Ultimate~~ Basic design wind speed. The ~~ultimate~~ basic design wind speed, $V_{\cancel{ult}}$, in mph, for the determination of the wind loads shall be determined by Figures 1609.3(1)~~, 1609.3(2) and 1609.3(3~~ through (8). The ~~ultimate~~ basic design wind speed, $V_{\cancel{ult}}$, for use in the design of Risk Category II buildings and structures shall be obtained from Figure 1609.3(1) and (5). The ~~ultimate~~ basic design wind speed, $V_{\cancel{ult}}$, for use in the design of Risk Category III buildings and structures shall be obtained from Figure 1609.3(2) and (6). The basic design wind speed, *V*, for use in the design of Risk Category IV buildings and structures shall be obtained from Figure ~~1609.3(2~~ 1609.3(3) and (7). The ~~ultimate~~ basic design wind speed, $V_{\cancel{ult}}$, for use in the design of Risk Category I buildings and structures shall be obtained from Figure ~~1609.3(3~~ 1609.3(4) and (8). The ~~ultimate~~ basic design wind speed, $V_{\cancel{ult}}$, for the special wind regions indicated near mountainous terrain and near gorges shall be in accordance with local jurisdiction requirements. The ~~ultimate~~ basic design wind speeds, $V_{\cancel{ult}}$, determined by the local jurisdiction shall be in accordance with ~~Section 26.5.1~~ Chapter 26 of ASCE 7.

In nonhurricane-prone regions, when the ~~ultimate~~ basic design wind speed, $V_{\cancel{ult}}$, is estimated from regional climatic data, the ~~ultimate~~ basic design wind speed, $V_{\cancel{ult}}$, shall be determined in accordance with ~~Section 26.5.3~~ Chapter 26 of ASCE 7.

Delete and substitute as follows:

FIGURE ~~1609.3~~ 1609.3(1) (1)
~~ULTIMATE~~ BASIC DESIGN WIND SPEEDS, ~~V_{ult}~~ V , FOR RISK CATEGORY II BUILDINGS AND OTHER STRUCTURES

(Existing code figure not shown for clarity)



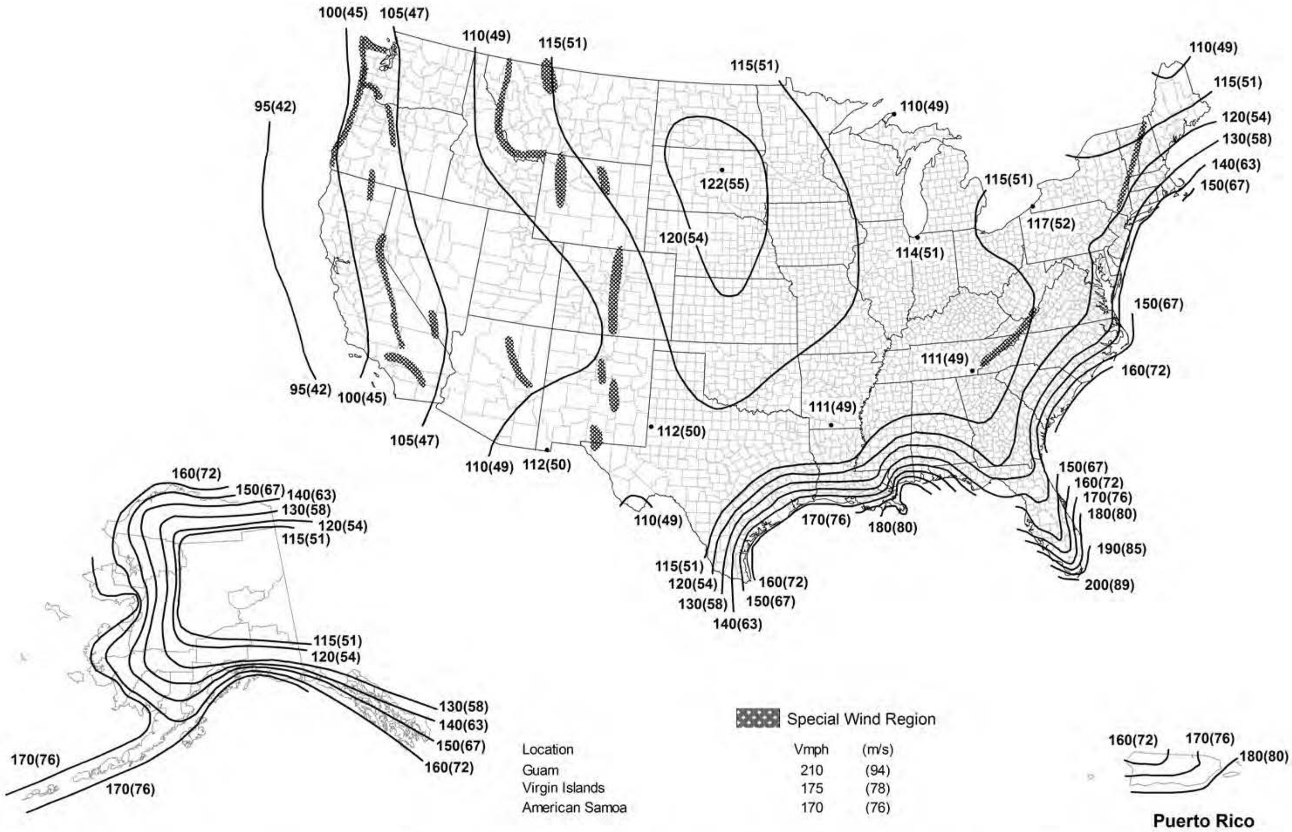
Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation is permitted between contours. Point values are provided to aid with interpolation.
3. Islands, coastal areas, and land boundaries outside the last contour shall use the last wind speed contour.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 Years).
6. Location-specific basic wind speeds shall be permitted to be determined using www.atcouncil.org/windspeed

Revise as follows:

FIGURE ~~1609.3~~ 1609.3(2) (2)
~~ULTIMATE~~ BASIC DESIGN WIND SPEEDS, $v_{\cancel{ult}}$, FOR RISK CATEGORY III ~~AND IV~~ BUILDINGS AND OTHER STRUCTURES

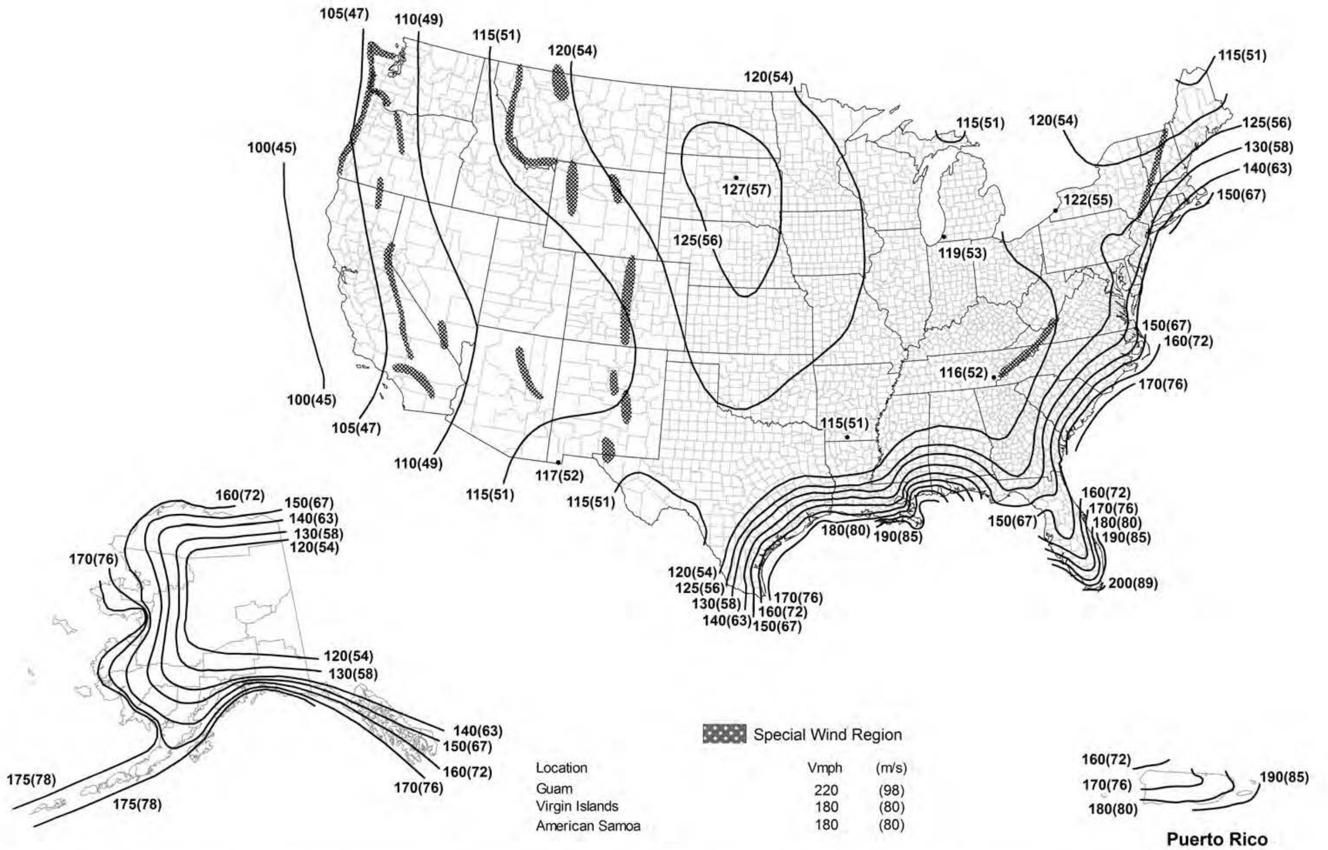
(Existing code figure not shown for clarity)



Notes:
1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation is permitted between contours. Point values are provided to aid with interpolation.
3. Islands, coastal areas, and land boundaries outside the last contour shall use the last wind speed contour.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (Annual Exceedance Probability = 0.000588, MRI = 1700 Years).
6. Location-specific basic wind speeds shall be permitted to be determined using www.atcouncil.org/windspeed

Add new text as follows:

FIGURE 1609.3(3)
BASIC DESIGN WIND SPEEDS, V, FOR RISK CATEGORY IV BUILDINGS AND OTHER STRUCTURES



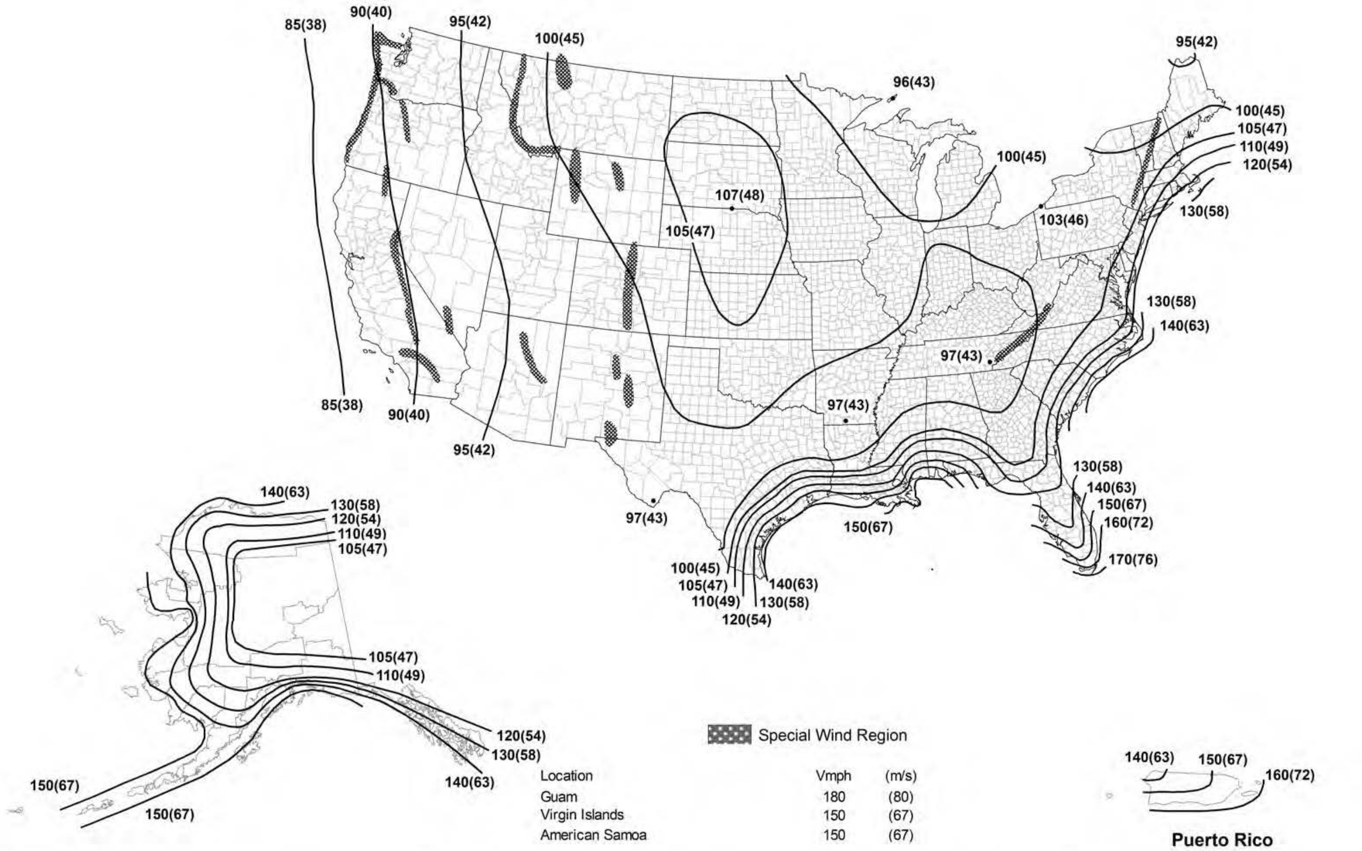
Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation is permitted between contours. Point values are provided to aid with interpolation.
3. Islands, coastal areas, and land boundaries outside the last contour shall use the last wind speed contour.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 1.6% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00033, MRI = 3000 Years).
6. Location-specific basic wind speeds shall be permitted to be determined using www.atcouncil.org/windspeed

Delete and substitute as follows:

FIGURE ~~1609.3~~ 1609.3(4) (3)
~~ULTIMATE~~ BASIC DESIGN WIND SPEEDS, ~~V_{ult}~~V , FOR RISK CATEGORY I BUILDINGS AND OTHER STRUCTURES

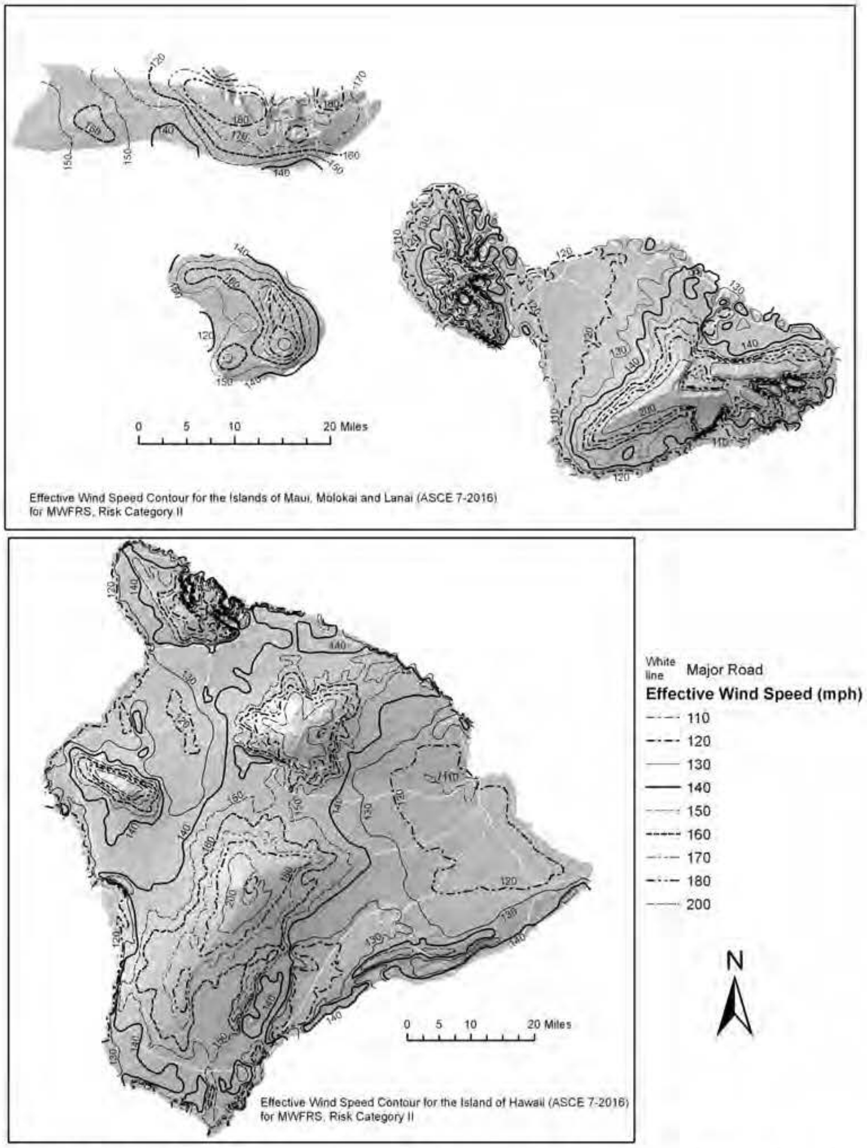
(Existing code figure not shown for clarity)



Notes:
1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation is permitted between contours. Point values are provided to aid with interpolation.
3. Islands, coastal areas, and land boundaries outside the last contour shall use the last wind speed contour.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 15% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00333, MRI = 300 Years).
6. Location-specific basic wind speeds shall be permitted to be determined using www.atcouncil.org/windspeed

Add new text as follows:

FIGURE 1609.3(5)
BASIC DESIGN WIND SPEEDS, V, FOR RISK CATEGORY II BUILDINGS AND OTHER STRUCTURES IN HAWAII

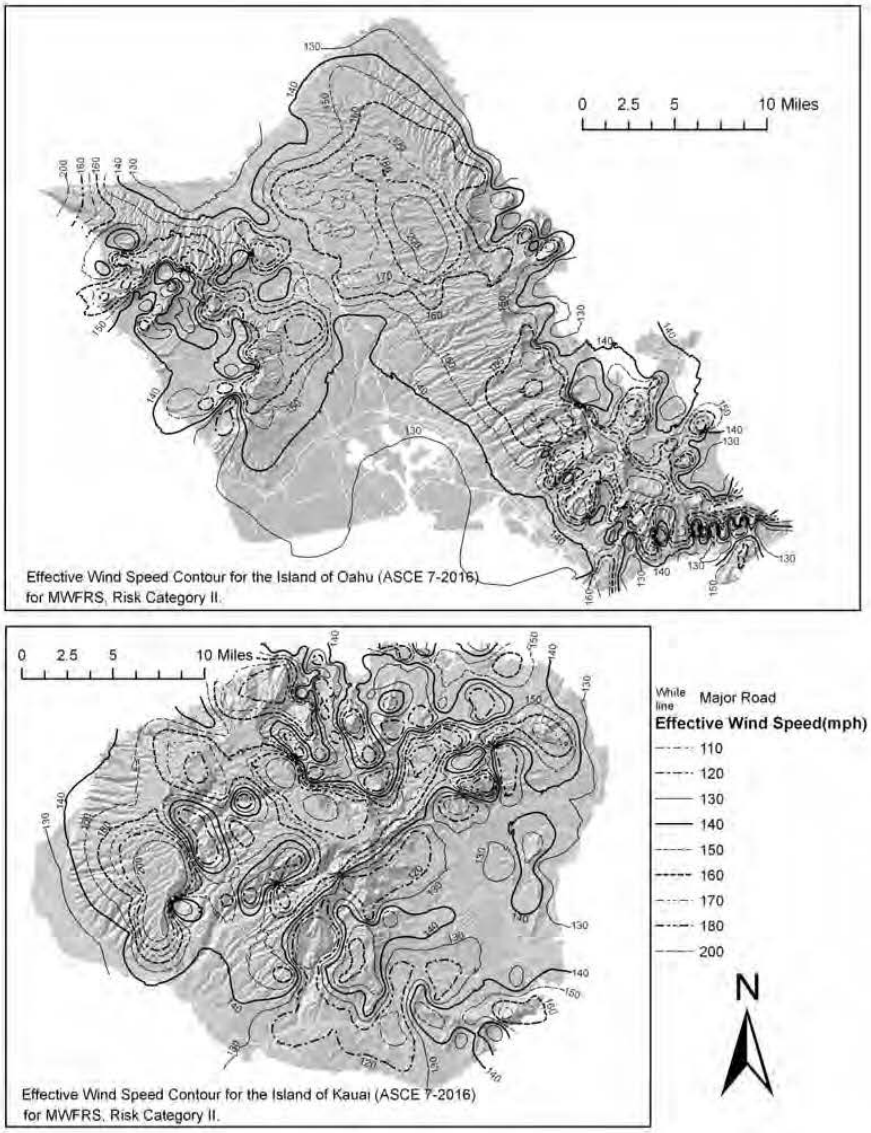


Basic Wind Speeds for Risk Category II Buildings and Other Structures (Hawaii).

Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. It is permitted to use the standard values of K_{zt} of 1.0 and K_d as given in Table 26.6-1
5. Ocean promontories and local escarpments shall be examined for unusual wind conditions.
6. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 Years).

FIGURE 1609.3(5)

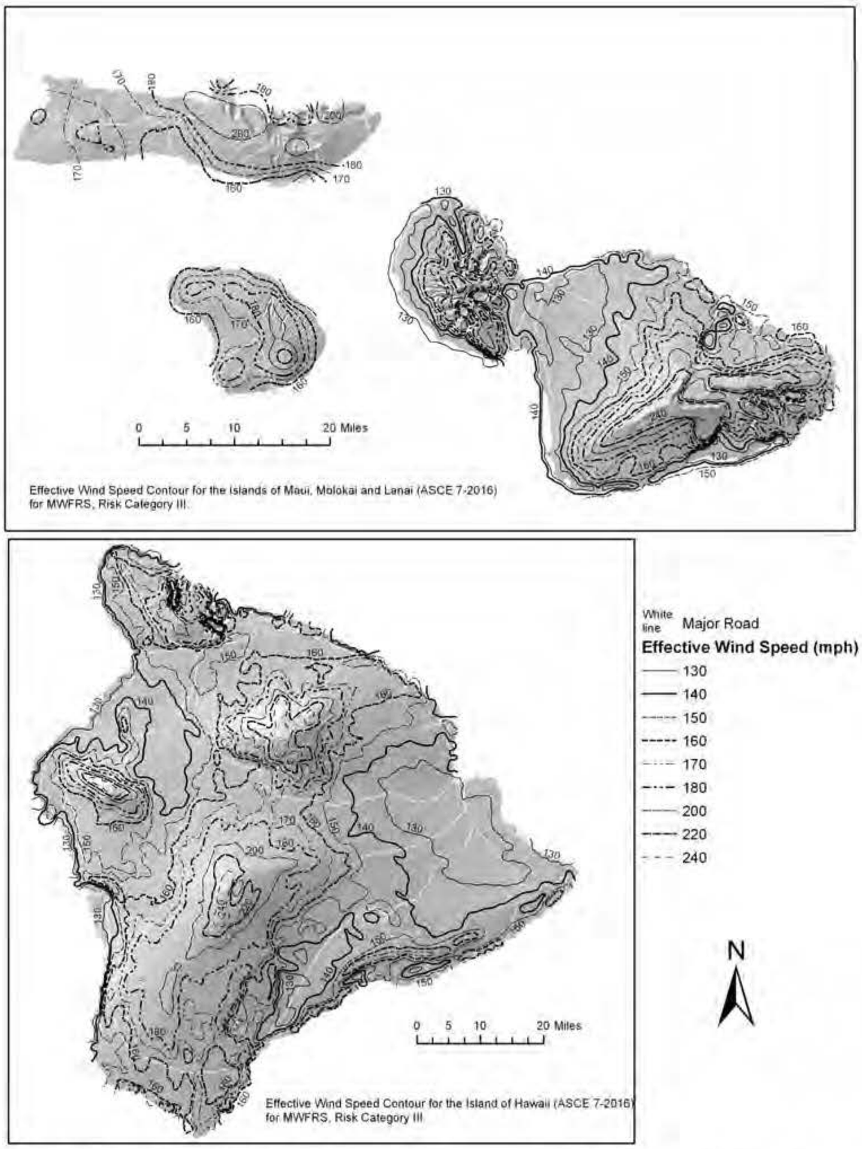


(continued) Basic Wind Speeds for Risk Category II Buildings and Other Structures (Hawaii).

Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. It is permitted to use the standard values of K_{zt} of 1.0 and K_d as given in Table 26.6-1
5. Ocean promontories and local escarpments shall be examined for unusual wind conditions.
6. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 Years).

FIGURE 1609.3(6)
BASIC DESIGN WIND SPEEDS, V, FOR RISK CATEGORY III BUILDINGS AND OTHER STRUCTURES IN HAWAII

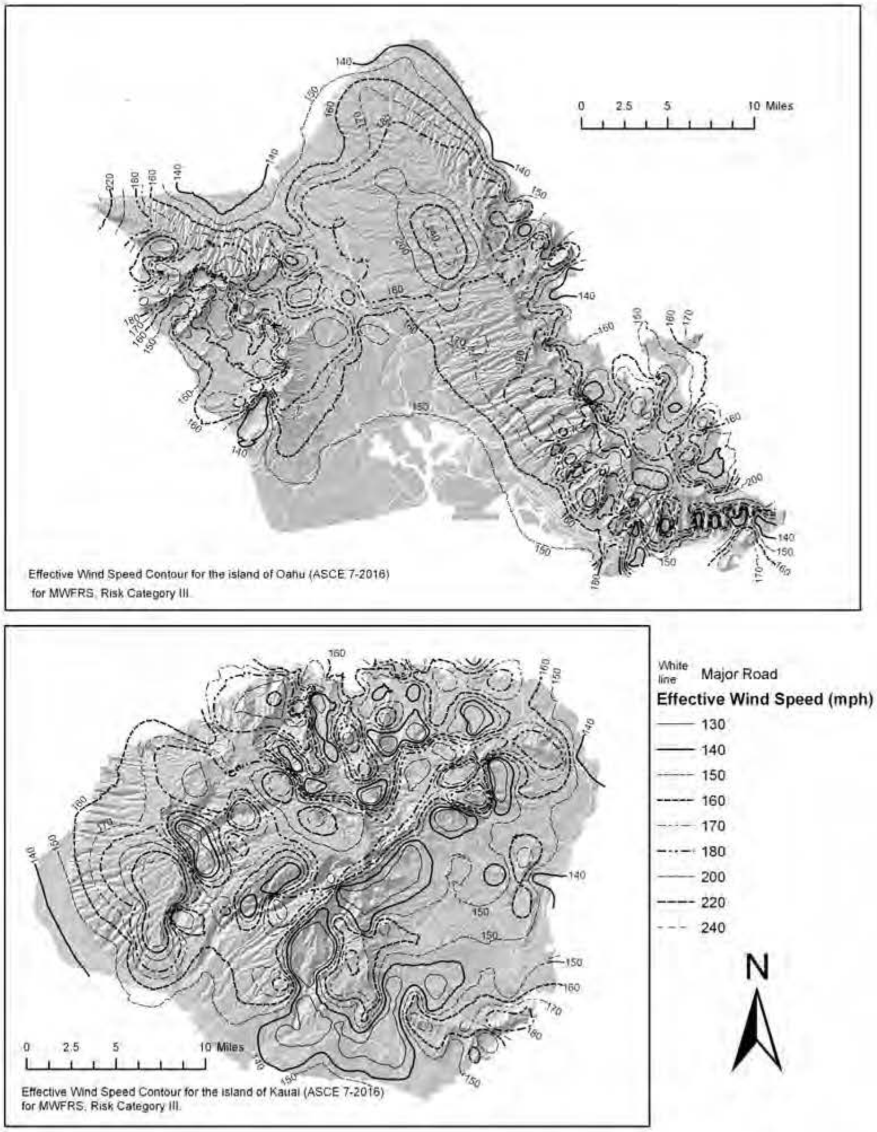


Basic Wind Speeds for Risk Category III Buildings and Other Structures (Hawaii).

Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. It is permitted to use the standard values of K_{zt} of 1.0 and K_d as given in Table 26.6-1
5. Ocean promontories and local escarpments shall be examined for unusual wind conditions.
6. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (Annual Exceedance Probability = 0.000588, MRI = 1700 Years).).

FIGURE 1609.3(6)

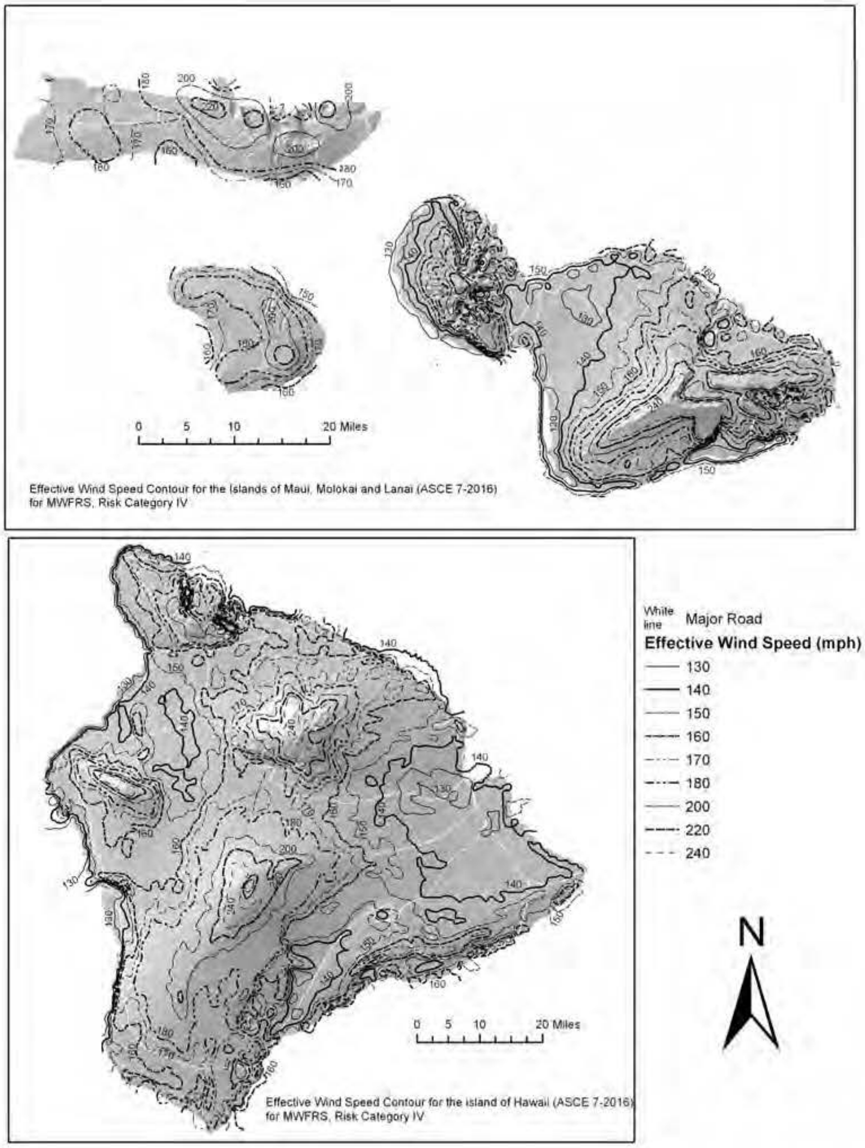


(continued) Basic Wind Speeds for Risk Category III Buildings and Other Structures (Hawaii).

Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. It is permitted to use the standard values of K_{zt} of 1.0 and K_d as given in Table 26.6-1
5. Ocean promontories and local escarpments shall be examined for unusual wind conditions.
6. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years (Annual Exceedance Probability = 0.000588, MRI = 1700 Years).).

FIGURE 1609.3(7)
BASIC DESIGN WIND SPEEDS, V, FOR RISK CATEGORY IV BUILDINGS AND OTHER STRUCTURES IN HAWAII

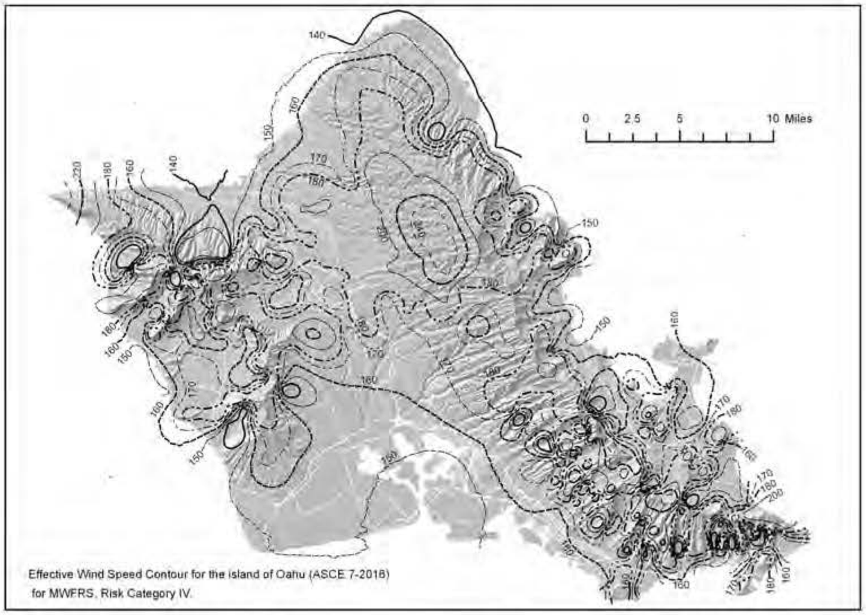


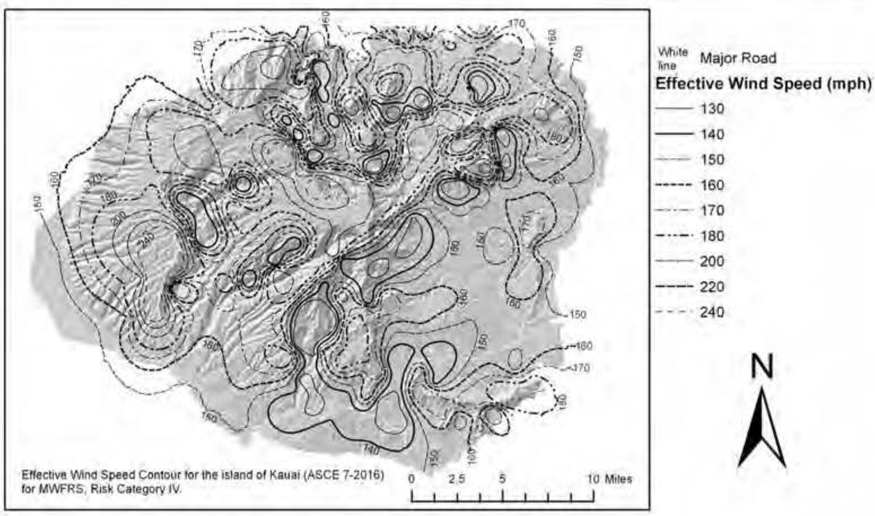
Basic Wind Speeds for Risk Category IV Buildings and Other Structures (Hawaii).

Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. It is permitted to use the standard values of K_{zt} of 1.0 and K_d as given in Table 26.6-1
5. Ocean promontories and local escarpments shall be examined for unusual wind conditions.
6. Wind speeds correspond to approximately a 1.7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.000333, MRI = 3000 Years).).

FIGURE 1609.3(7)



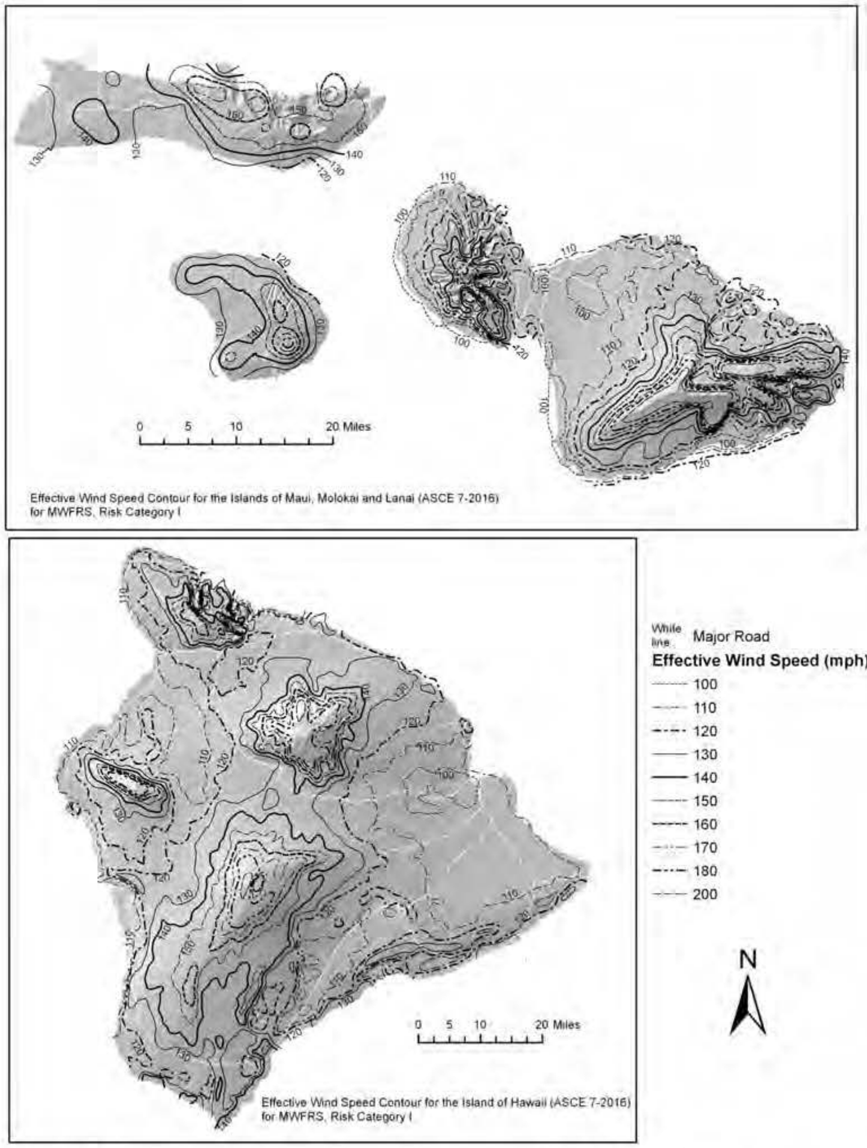


(c o n t i n u e d) Basic Wind Speeds for Risk Category IV Buildings and Other Structures (Hawaii).

Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. It is permitted to use the standard values of K_{zt} of 1.0 and K_d as given in Table 26.6-1
5. Ocean promontories and local escarpments shall be examined for unusual wind conditions.
6. Wind speeds correspond to approximately a 1.7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.000333, MRI = 3000 Years).).

FIGURE 1609.3(8)
BASIC DESIGN WIND SPEEDS, V, FOR RISK CATEGORY I BUILDINGS AND OTHER STRUCTURES IN HAWAII

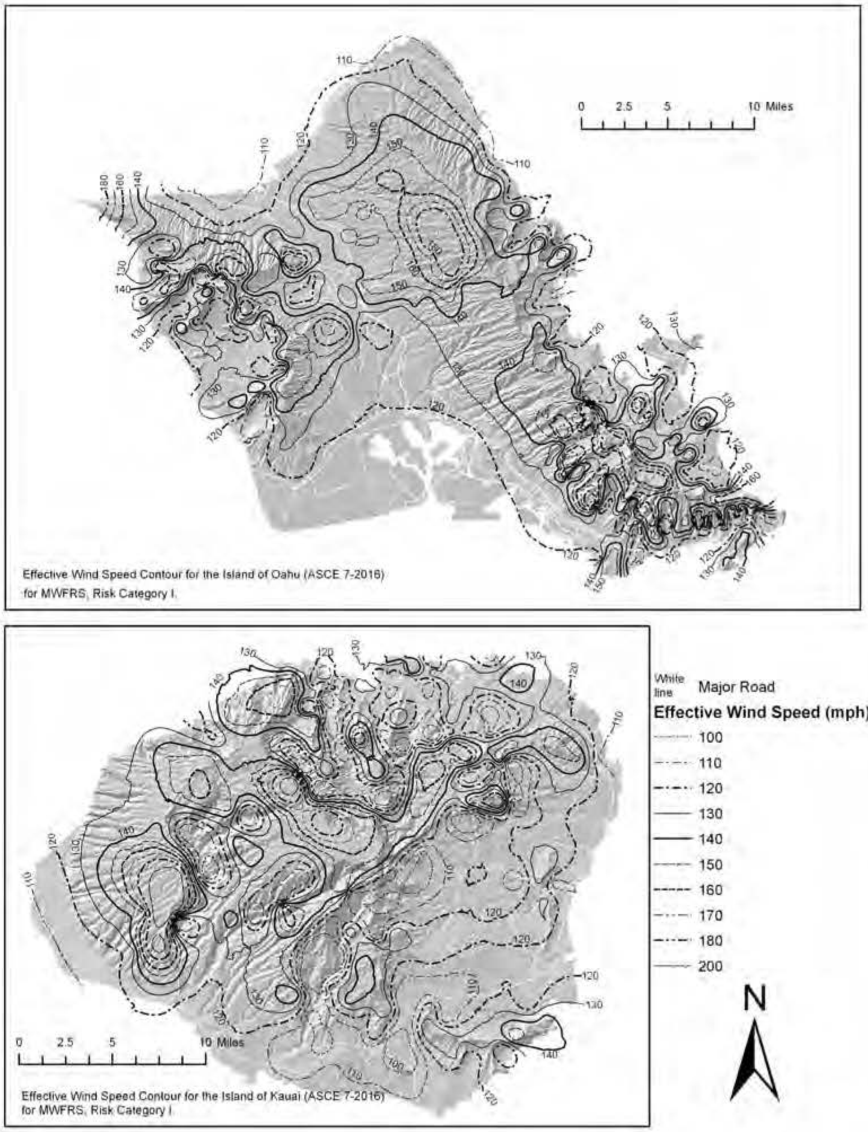


Basic Wind Speeds for Risk Category I Buildings and Other Structures (Hawaii).

Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. It is permitted to use the standard values of K_{zt} of 1.0 and K_d as given in Table 26.6-1
5. Ocean promontories and local escarpments shall be examined for unusual wind conditions.
6. Wind speeds correspond to approximately a 15% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00333, MRI = 300 Years).

FIGURE 1609.3(8)



(c o n t i n u e d) Basic Wind Speeds for Risk Category I Buildings and Other Structures (Hawaii).

Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. It is permitted to use the standard values of K_{zt} of 1.0 and K_d as given in Table 26.6-1
5. Ocean promontories and local escarpments shall be examined for unusual wind conditions.
6. Wind speeds correspond to approximately a 15% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00333, MRI = 300 Years).

Revise as follows:

1609.3.1 Wind speed conversion. When required, the ~~ultimate~~ basic design wind speeds of Figures 1609.3(1)~~, 1609.3(2) and 1609.3(3~~ through (8) shall be converted to ~~nominal~~ allowable stress design wind speeds, V_{asd}, using Table 1609.3.1 or Equation 16-33.

$$\cancel{L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)}$$

$$V_{asd} = V_{ult}\sqrt{0.6} \quad \textbf{(Equation 16-33)}$$

where:

V_{asd} = ~~Nominal~~ Allowable stress design wind speed applicable to methods specified in Exceptions 4 and 5 of Section 1609.1.1.

$V_{\cancel{ult}}$ = ~~Ultimate~~ Basic design wind speeds determined from Figures 1609.3(1)~~, 1609.3(2) or 1609.3(3~~ through (8).

TABLE 1609.3.1
WIND SPEED CONVERSIONS[a, b, c]

$V_{\cancel{ULT}}$	100	110	120	130	140	150	160	170	180	190	200
V_{asd}	78	85	93	101	108	116	124	132	139	147	155

For SI: 1 mile per hour = 0.44 m/s.

a. Linear interpolation is permitted.

b. V_{asd} = ~~nominal~~ allowable stress design wind speed applicable to methods specified in Exceptions 1 through 5 of Section 1609.1.1.

c. $V_{\cancel{ult}}$ = ~~ultimate~~ basic design wind speeds determined from Figure 1609.3(1)~~, 1609.3(2) or 1609.3(3~~ through (8).

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).
The changes proposed in all sections harmonizes terminology between the code and the loading standard. In all instances the word "ultimate" is changed to "basic" and the subscript "ult" is removed from the variable "V". Similarly, the word "nominal" is changed to "allowable stress" in all placed to be consistent with the terminology used in the loading standard.

The increase in distance in 1609.1.1.1 Applicability to 2 miles from 1 mile is also to correct the discrepancy between the code and ASCE 7.

The design wind speed maps have been updated to reflect the maps adopted into ASCE 7-16. During the development of the ASCE 7-16 standard the ASCE 7 Wind Load Subcommittee made substantial revision to the wind speed maps contained within the standard, and the number of maps went from three to eight. These revisions include the development of separate maps for Risk Categories III and IV structures; reconstruction of the special wind regions within the maps, correcting known deficiencies in the wind speed contours; and modification of the basic wind speed based on updated climatic and weather data. New hurricane contours in the northeastern states were developed based on updated hurricane models and the locations of the contours along the hurricane coast line were adjusted to reflect new research into the decay rate of hurricanes over land. New maps for the State of Hawaii were developed to eliminate it as a "special wind region" and to provide guidance on the wind patterns for the state that occur because of the unique topography there. Currently there are eight new maps for main wind force and component and cladding design in the ASCE 7-16 standard along with four new serviceability maps.

Cost Impact: Will not increase the cost of construction
The proposed map changes will decrease the cost of construction in the majority of the United States. The basic design wind speeds have been lowered at most locations on the new maps based on the latest data available, thus reducing the overall cost of construction. Along the hurricane coastline from Virginia to Texas, the wind speeds remain nearly unchanged from the current maps and thus the cost of construction will not change. There may be a very small increase in Category IV structures in some parts of the country, due to the new mean recurrence interval for Risk Category IV, which has now been separated from Risk Category III.
The basic wind speeds for all four Risk Category maps decrease very significantly west of the Continental Divide. For example, in much of coastal California wind speeds decrease by as much as 16%, 15%, and 11% from the previous maps for Risk Category II, III, and IV structures, respectively. Wind speeds in the Northern Great Plains states are similar to previous maps, and wind speeds in hurricane prone regions from Virginia to Texas remain nearly unchanged. In the rest of the continental United States south and east of the Great Plains, wind speeds decrease as much as 12%, 8%, and 4% for Risk Category II, III, and IV buildings respectively.For a comparatively small number of buildings, the design wind speeds for Risk Category IV increase slightly as a result of the split of Risk Category III and IV into separate maps with different mean recurrence intervals. The wind speeds for Risk Category IV buildings increase on the order of 5% in hurricane-prone regions from Virginia to Texas. The wind speeds for Risk Category IV buildings increase about 2% in much of Nebraska and the Dakotas and to a lesser extent in the adjacent states.

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).

S56-16 : 1603.1-GOUPIL11532

S57-16

IBC: 1603.1, 1603.1.3, 1603.1.9 (New).

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

2015 International Building Code

Revise 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~~ 1603.1.9 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~~ p_g.
> 3. Ultimate design wind speed, V_{ult}, (3-second gust), miles per hour (mph) (km/hr) and nominal design wind speed, V_{asd}, 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.
> 7. Rain load data.

1603.1.3 Roof snow load data. The ground snow load, ~~P~~ p_g, shall be indicated. In areas where the ground snow load, ~~P~~ p_g, exceeds 10 pounds per square foot (psf) (0.479 kN/m^2), the following additional information shall also be provided, regardless of whether snow loads govern the design of the roof:

1. Flat-roof snow load, ~~P~~ p_f.
2. Snow exposure factor, C_e.
3. Snow load importance factor, I_s.
4. Thermal factor, C_t.
5. Slope factor(s), C_s
6. Drift surcharge load(s), ~~P~~ p_d, where the sum of ~~P~~ p_d and ~~P~~ p_f exceeds 20 psf (0.96 kN/m^2).
7. Width of snow drift(s), *w*.

Add new text as follows:

1603.1.9 Roof rain load data. The following roof rain load parameters shall be shown regardless of whether the rain loads govern the design:

1. 1. Rain Load, *R* (psf) (kN/m^2)
2. Rain Intensity, *i* (in/hr) (cm/hr)

Reason: This change proposes to coordinate the IBC with the 2016 edition of the referenced loading standard *ASCE 7Minimum Design Loads and associated Criteria for Buildings and Other Structures* (ASCE 7-16). In particular, the snow loads variables should be lower case and the required slope factor should also be included in the required roof snow loads data. Also, rain load data should be included on the list of design loads required on construction documents.

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).

S57-16 : 1603.1-GOUPIL12738

S58-16

IBC: 1603.1.

Proponent : Gerald Gunny, City of Henderson Department of Building and Fire Safety, representing Southern Nevada Chapter International Code Council

2015 International Building Code

Revise 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 dead and live loads.
> 2. Ground snow load, P_g.
> 3. Ultimate design wind speed, V_{ult}, (3-second gust), miles per hour (mph) (km/hr) and nominal design wind speed, V_{asd}, 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.

Reason: The revised code section is a list of information to be placed on the construction documents for use with the conventional light-frame construction provisions of Section 2308. The estimated dead loads are necessary to use the span Tables in Section 2308. The estimated dead loads specified on the construction documents can also be confirmed by the plans examiner.

Cost Impact: Will not increase the cost of construction
The proposal will add an additional item to an otherwise required list of information on the construction documents and will not impact the cost of construction.

S58-16 : 1603.1-GUNNY11540

S59-16

IBC: 1603.1.4.

Proponent : Scott Douglas (sdouglasscott@gmail.com)

2015 International Building Code

Revise as follows:

1603.1.4 Wind design data. The following information related to wind loads shall be shown, regardless of whether wind loads govern the design of the lateral force-resisting system of the structure:

1. Ultimate design wind speed, V_{ult}, (3-second gust), miles per hour (km/hr) and nominal design wind speed, V_{asd}, as determined in accordance with Section 1609.3.1.
2. *Risk category*.
3. Wind exposure. Applicable wind direction if more than one wind exposure is utilized.
4. Applicable internal pressure coefficient.
5. Design wind pressures to be used for exterior component and cladding materials not specifically designed by the *registered design professional*responsible for the design of the structure, psf (kN/m^2).
6. <u>Topographic factor, K_{zt}.</u>

Reason: The topographic factor K_{zt} should be show n on the *construction documents* as it is of equal importance as the other w ind design data items currently required by Section 1603.1.4.

Cost Impact: Will not increase the cost of construction
The addition of the topographic factor to the w ind design data required to be show n on the *construction documents* w ill have no cost impact

S59-16 : 1603.1.4-DOUGLAS13145

S60-16

IBC: 1603.1.10 (New), 1603.1.11 (New), 1603.1.4, 1603.1.6, 1603.1.9 (New).

Proponent : Karl Rubenacker, Gilsanz Murray Steficek, representing Structural Engineer's Association of New York

2015 International Building Code

Revise as follows:

1603.1.4 Wind design data. The following information related to wind loads shall be shown, regardless of whether wind loads govern the design of the lateral force-resisting system of the structure:

1. Ultimate design wind speed, V_{ult}, (3-second gust), miles per hour (km/hr) and nominal design wind speed, V_{asd}, as determined in accordance with Section 1609.3.1.
2. *Risk category*.
3. Wind exposure. Applicable wind direction if more than one wind exposure is utilized.
4. Applicable internal pressure coefficient.
5. Design wind pressures to be used for exterior component and cladding materials not specifically designed by the *registered design professional*responsible for the design of the structure, psf (kN/m^2).
6. Design Base Shear

1603.1.6 Geotechnical information. The design load-bearing values of soils or rock under shallow foundations and/or the design load capacity of deep foundations shall be shown on the *construction documents*.

Add new text as follows:

1603.1.9 Partition loads The equivalent uniform partition loads, or in lieu of these, a statement to the effect that the design was predicated on actual partition loads.

1603.1.10 Superimposed dead loads The uniformly distributed superimposed dead loads used in the design.

1603.1.11 Other loads Other loads used in the design, including but not limited to the loads of machinery or equipment, which are of greater magnitude than the loads defined in the specified floor and roof loads, shall be indicated by their descriptions and locations.

Reason: The added requirements provide relevant structural design information that should be included on the drawings. They make the structural design more transparent and are also useful information for peer reviews and/or future work on the structure.

Cost Impact: Will not increase the cost of construction
Will not increase cost of construction, as it is simply information to be provided on the drawings.

S60-16 : 1603.1.4-RUBENACKER13439

S61-16

IBC: 1603.1.5.

Proponent : James Bela, representing Oregon Earthquakie Awareness (sasquake@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*~~ *Occupancy and Use categories*.
2. Seismic ~~importance~~ safety factor, I_e.
3. Mapped ~~spectral response acceleration parameters, S_S and S_1.~~ lateral design force (base shear) coefficient
4. *Site class*.
5. ~~Design spectral response acceleration parameters, S_{DS} and S_{D1}~~Basic seismic force-resisting system(s).
6. ~~*Seismic design category*~~Design base shear(s).
7. ~~Basic seismic force-resisting system(s~~ Seismic response coefficient(s), *CS*.
8. ~~Design base shear(s~~ Response modification coefficient(s), *R*.
9. ~~Seismic response coefficient(s), *CS*~~Analysis procedure used.
10. ~~Response modification coefficient(s), *R*.~~
11. ~~Analysis procedure used.~~

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

Bibliography: Cost Breakdow n of Nonstructural Building Elements
Taghavi, S. and Miranda, E. (2003). Cost Breakdow n of Nonstructural Building Elements, PEER Report 2003/05, Pacific 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 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://w w w .earthquakespectra.org/doi/abs/10.1193/1.4000032

Low -Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Holliday, L. and Kang, T. (2014). "Low -Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions." J. Perform. Constr. Facil., 10.1061/(ASCE)CF.1943-5509.0000630, 04014141.

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Read More: http://ascelibrary.org/doi/10.1061/%28ASCE%29CF.1943-5509.0000630

Homeow ner's Guide to Earthquake Safety
California Seismic Safety Commission, 2005 Edition, 49 p.
http://w w w .disclosuresource.com/dow nloads/earthquake.pdf

Retrofitting Questions and Answ ers
Earthquake Safety, Inc., 2015 (w eb based)
http://w w w .earthquakesafety.com/earthquake-retrofitting-faq.html

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

Earthquake Architecture w ebsite
http://w w w .iitk.ac.in/nicee/w cee/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

S61-16 : 1603.1.5-BELA12284

S62-16

IBC: 1603.1.8.

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

2015 International Building Code

Revise as follows:

1603.1.8 Special loads. Special loads that are applicable to the design of the building, structure or portions thereof, including but not limited to the loads of machinery or equipment, which are of greater magnitude than the loads defined in the specified floor and roof loads shall be ~~indicated along with~~ specified in the ~~specified section of this code that addresses the special loading condition.~~ construction drawings by their descriptions and locations

Reason: Machinery, equipment, planters, art structures and other elements impose loads that commonly exceed the capacity of the specified floor area loads. Structural engineers design these elements in specific locations with specific loads. This statement clarifies the communication of the designs direct to the builders and installers and allows the installer to provide feedback if the special load element exceeds the loads or requires a different location than was designed.

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 loads and locations, and it may speed up permitting and construction by ensuring the designer provides the information to the authority having jurisdiction and the contractor.

S62-16 : 1603.1.8-HUSTON13233

S63-16

IBC: 1604.1, 1604.3, 1604.4, 1604.8.2, 1604.9.

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~~ 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[a,b,c,h,i]

CONSTRUCTION	L or L_r	S or W [f]	$D + L$ [d,g]
Roof members: [e]			
Supporting plaster or stucco ceiling	l/360	l/360	l/240
Supporting nonplaster ceiling	l/240	l/240	l/180
Not supporting ceiling	l/180	l/180	l/120
Floor members	l/360	—	l/240
Exterior walls:			
With plaster or stucco finishes	—	l/360	—
With other brittle finishes	—	l/240	—
With flexible finishes	—	l/120	—
Interior partitions: [b]			
With plaster or stucco finishes	l/360	—	—

With other brittle finishes	*l*/240	—	—
With flexible finishes	*l*/120	—	—
Farm buildings	—	—	*l*/180
Greenhouses	—	—	*l*/120

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 1503.4 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 *1* /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~~ 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~~ 1.4.4 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.~~

Revise 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 14~~ 12, 13, 15, 17, and ~~Appendix 11A~~ 18 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).

S63-16 : 1604.1-GOUPIL12743

S64-16

IBC: 1604.3.

Proponent : Don Scott, representing National Council of Structural Engineering Associations (dscott@pcs-structural.com)

2015 International Building Code

Revise as follows:

TABLE 1604.3
DEFLECTION LIMITS[a,b,c,h,i]

CONSTRUCTION	*L*	*S or W* [f]	*D + L* [d,g]
Roof members: [e]			
Supporting plaster or stucco ceiling	*l*/360	*l*/360	*l*/240
Supporting nonplaster ceiling	*l*/240	*l*/240	*l*/180
Not supporting ceiling	*l*/180	*l*/180	*l*/120
Floor members	*l*/360	—	*l*/240
Exterior walls:			
With plaster or stucco finishes	—	*l*/360	—
With other brittle finishes	—	*l*/240	—
With flexible finishes	—	*l*/120	—
Interior partitions: [b]			
With plaster or stucco finishes	*l*/360	—	—
With other brittle finishes	*l*/240	—	—
With flexible finishes	*l*/120	—	—
Farm buildings	—	—	*l*/180
Greenhouses	—	—	*l*/120

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 1503.4 for roof drainage requirements.

f. The wind load ~~is~~ shall be permitted to be ~~taken~~ approximated as 0.42 times the "component and cladding" loads or directly calculated using the 10 year MRI wind speed 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 *1* /120.

i. For cantilever members, *l* shall be taken as twice the length of the cantilever.

Reason: Deflection limits are serviceability limits, and common practice is to use a 10 year MRI wind speed when computing service wind load. Prior to the incorporation of ASCE 7-10 into 2012 IBC, footnote f permitted a 0.7 reduction factor to be applied to the wind load that is an approximate conversion of a 50 year to a 10 year MRI wind load. This was similar to the wind load importance factors prior to ASCE 7-10 that converted the wind load based on a 50 year MRI to a 100 year MRI wind for Rick Category III and IV structures (I = 1.15) or converted the 50 year MRI to a 25 year MRI wind load for Risk Category I structures (I = 0.87). This 0.7 approximate conversion from 50 year to 10 year MRI wind was modified by a factor of 0.6 and became 0.42 when ASCE 7-10 was introduced into 2012 IBC that incorporated importance and load factors into the wind speed map. This conversion is actually only valid for Risk Category II structures and would be slightly conservative for Risk Category III and IV structures. Using the 10 year MRI wind speed map would be more accurate for these structures. Using the 10 year MRI wind speed map is also more accurate than the approximate conversion that is technically different for hurricane and non-hurricane regions.

This proposed revision clarifies the intent of footnote f and points the user to where a more accurate assessment of the 10 year service load can be determined. It also will draw attention to the other wind speed maps in the ASCE 7 Appendix C Commentary if a design professional wants to consider specifying a service wind load other than 10 year MRI, such as a 25 year MRI.

Cost Impact: Will not increase the cost of construction

There will be no impact on construction costs with this proposal as this proposal does not change the deflection limit, only clarifies the use of the proper wind criteria.

S64-16 : 1604.3-SCOTT12736

S65-16

IBC: 1604.3.

Proponent : Scott Douglas, representing Douglas Engineering (sdouglasscott@gmail.com)

2015 International Building Code

Revise as follows:

TABLE 1604.3
DEFLECTION LIMITS[a,b,c,h,i]

(Portions of table not shown remain unchanged)

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 1503.4 for roof drainage requirements.

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 *1* /120.

i. <u>*l* = Length of the member between supports.</u> For cantilever members, *l* shall be taken as twice the length of the cantilever.

Reason: "*l*" needs to be defined. It could also be defined in Section 1602.

Cost Impact: Will not increase the cost of construction
This addition is editorial only, thus this change will not increase the cost of construction.

S65-16 : TABLE 1604.3-DOUGLAS12992

S66-16

IBC: 1604.3.

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

2015 International Building Code

Revise as follows:

TABLE 1604.3
DEFLECTION LIMITS[a,b,c,h,i]

(Portions of table not shown remain unchanged)

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 1503.4 for roof drainage requirements.

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 <u>deflection due to the creep component of long-term </u>dead load <u>deflection </u>shall<u> be permitted to</u> 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 *1* /120.

i. For cantilever members, *l* shall be taken as twice the length of the cantilever.

Reason: Note **d** in Table 1604.3 says that the D+L column " only applies to the deflection due to the creep component of long-term dead load deflection plus the short-term live load deflection". Note **d** also provides the guidance necessary for concrete and wood systems to get the proper assessment of creep due to long-term dead load deflections.

According to AISC, Note **g** was added in a later code cycle than note **d** and attempted to add the corresponding guidance for steel systems. It has caused some confusion because it does not explicitly say it is addressing creep in steel, which is nonexistent. The following text in commentary could be considered: "Steel structural members do not have long-term dead load deflections because there is no creep in steel. Therefore, the check of this deflection limit is based only upon the short-term deflections due to the live load."

Cost Impact: Will not increase the cost of construction
This is clarification only, and will have no cost impact

S67-16

IBC: 1604.3.

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

2015 International Building Code

Revise as follows:

TABLE 1604.3
DEFLECTION LIMITS[a,b,c,h,i]

(Portions of table remain unchanged)

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~~ lumber, structural glued laminated timber, prefabricated wood I-joists, and structural composite lumber 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~~ lumber and glued laminated timber members installed or used at all other moisture conditions or cross laminated timber and wood structural panels 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 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 1503.4 for roof drainage requirements.

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 *1* /120.

i. For cantilever members, *l* shall be taken as twice the length of the cantilever.

Reason: Revisions are proposed to recognize different creep behavior of specific w ood products in accordance w ith the NDS. Specifically, creep deformation of seasoned lumber, structural glued laminated timber, prefabricated w ood I-joists, and structural composite lumber members that are installed and used in dry conditions can be approximated by calculation of immediate deflection resulting from the use of 0.5D. For seasoned lumber and structural glued laminated timber that are installed and used in w et conditions and unseasoned lumber used in any conditions, creep deformation is larger and can be approximated by the immediate deflection resulting from the use of 1.0D. For cross-laminated timber and w ood structural panels used in dry conditions, creep deformation can be approximated by the immediate deflection resulting from the use of 1.0D. The 0.5D and 1.0D approach in footnote d are associated and consistent w ith w ith NDS 3.5.2 creep factors of 1.5 and 2.0. The NDS creep factors represent the combined deformation resulting from the immediate deformation under dead load plus long-term creep deformation.

Cost Impact: Will not increase the cost of construction
This change correlates structural provisions w ith a new product in the applicable standard and w ill not increase the

cost of construction.

S67-16 : TABLE 1604.3-RICHARDSON12336

S68-16

IBC: 1604.3.

Proponent : Julie Ruth, representing American Architectural Manufacturers Association (julruth@aol.com)

2015 International Building Code

Revise as follows:

TABLE 1604.3
DEFLECTION LIMITS[a,b,c,h,i]

(Portions of table not shown remain unchanged)

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 1503.4 for roof drainage requirements.

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~~ in Table 1604.3. Where framing members support glass ~~in accordance with Section 2403 using~~ the deflection ~~limit therein~~ shall not be greater than 1/175 the length of span of the framing member, ~~the~~ when calculated using, 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 *1* /120.

i. For cantilever members, *l* shall be taken as twice the length of the cantilever.

Reason: This proposal replaces reference to Section 2403 with criteria directly in the footnote for framing supporting glass. Section 2403 provides deflection criteria based upon the requirements for glass design. Specifically, IF the framing does not deflect more than 1/175 within the length of the edge of glass being supported, the glass is considered to be firmly supported. This distinction is needed in the application of ASTM E1300, which is referenced in Chapter 24 for the design of glass.
Section 1604 addresses deflection limits of framing. This is a serviceability issue. For serviceability of exterior wall systems that are critical components of the building envelope, and therefore sensitive to deflection in the control of air and water infiltration, it is more appropriate that the full length of span of the framing member be considered. This proposal establishes criteria based upon the length of span rather than length of the edge of glass being supported.

The criteria of 1/175 of the length of span of the framing member at allowable stress design wind pressure (50 year MRI) has been used successfully by the architectural metals and glass industry for over 30 years. It takes into consideration other aspects of the exterior wall system, such as joints and sealants, along with the design of glass and other cladding materials.

The attached paper gives further information about the need for this proposed change.

https://cdpaccess.com/proposal/fileupload/get/1172

Cost Impact: Will increase the cost of construction
This proposal reflects w hat is considered to be "good practice" w ithin the exterior w all cladding industry. No additional cost w ill be incurred if "good practice" is already being follow ed. If it is not, some additional cost may occur. The additional cost, how ever, w ould only apply to the vertical framing members that are supporting glass. This is a relatively small portion of the overall cost of an exterior curtainw all system.

S68-16 : TABLE 1604.3-RUTH11932

S69-16

IBC: 1604.3, 1604.3.7 (New).

Proponent : Julie Ruth, representing American Architectural Manufacturers Association (julruth@aol.com)

2015 International Building Code

Revise as follows:

TABLE 1604.3
DEFLECTION LIMITS[a,b,c,h,i]

(Portions of table not shown remain unchanged)

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 1503.4 for roof drainage requirements.

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~~ in Table 1604.3. Where framing 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~~ not exceed that specified in Section 1604.3.7.

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 *1* /120.

i. For cantilever members, *l* shall be taken as twice the length of the cantilever.

Add new text as follows:

1604.3.7 Framing Supporting Glass The deflection of framing members supporting glass subjected to 0.6 times the "component and cladding" wind loads shall not exceed the following:

1. 1/175 of the length of span of the framing member, for framing members having a length not more than 13 foot 6 inches, or
2. 1/175 of the length of span of the framing member + 1/4 inch, for framing members having a length greater than 13 foot 6 inches.

Reason: This proposal replaces reference to Section 2403 for deflection limits on framing supporting glass with reference to a new section in Chapter 16.
Although the deflection limit given in Section 2403 is appropriate for glass design, and is similar to that given in the proposed new section, it does not address the deflection of the framing member over the entire length of its span. The later is a serviceability concern, which should more appropriately be addressed in Section 1504.

This proposal establishes the appropriate deflection limit for exterior w all framing members that are supporting glass. It is based upon criteria given in AAMA TIR-11 Maximum Allow able Deflection of Framing Systems for Building Cladding Components at Design Wind Loads. This criterion has been used successfilly by the fenestration industry for decades.

The attached paper discusses the need for this criteria is further detail.

https://cdpaccess.com/proposal/fileupload/get/1173

Cost Impact: Will increase the cost of construction
The criteria given is w ell established w ithin the fenestration industry. There is no cost impact for designers and contractors w ho are currently follow ing "good practice" w ith regards to curtainw all framing systems. For designers and contractors w ho are not follow ing good practice there may be additional cost associated w ith the vertical framing members that are supporting glass. This is a relatively small portion of the overall cost of an exterior curtainw all system.

S69-16 : TABLE 1604.3-RUTH11933

S70-16

IBC: 1604.3.3, 2203.2, 2207.1, 2207.1.1.

Proponent : Bonnie Manley, AISI, representing Steel Joist Institute (bmanley@steel.org)

2015 International Building Code

Revise as follows:

1604.3.3 Steel. The deflection of steel structural members shall not exceed that permitted by AISC 360, AISI S100, ASCE 8, SJI CJ~~, SJI JG, SJI K~~ or SJI ~~LH/DLH~~ 100, as applicable.

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~~ 100. 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 or AISI S220, as applicable.

2207.1 General. The design, manufacture and use of open-web steel joists and joist girders shall be in accordance with ~~one of the following Steel Joist Institute (~~ either SJI~~) specifications:~~

1. ~~SJI CJ~~
2. ~~SJI K~~
3. ~~SJI LH/DLH~~
4. ~~SJI JG~~

CJ or SJI 100, as applicable.

2207.1.1 Seismic design. Where required, the seismic design of buildings shall be in accordance with the additional provisions of Section 2205.2 or ~~2211.6~~ 2211.1.

Reference standards type: This contains both new and updated standards
Add new standard(s) as follows:
Add the following new standard:
100-15, *Standard Specification for K-Series, LH-Series, and DLH-Series Open Web Steel Joists and for Joist Girders, 2015*, 1604.3.3, 2203.2, 2207.1

Delete the following existing references:

~~JG—10, Standard Specification for Joist Girders,~~
~~K—10 , Standard Specification for Open Web Steel Joists, K-series,~~
~~LH/DLH—10, Standard Specification for Longspan Steel Joists, LH-series and Deep Longspan Steel Joists, DLH-series,~~

Reason: This proposal adopts the new ly completed 2015 edition (44th Edition) of the combined SJI-100, *Standard Specification for K-Series, LH-Series, and DLH-Series Open Web Steel Joists and for Joist Girders*, in the applicable code sections. Its publication represents a significant change in the presentation of the SJI Specifications. Previously, there w ere three separate specifications (all found in the 43rd Edition), covering K-Series, LH/DLH-Series, and Joist Girders, each one an independent ANSI standard. The new ly combined ANSI standard represents a major simplification for the specifying professional.
In addition to this overall change, below is a summary of substantive, notew orthy changes found in the new SJI 100:

- For concentrated loads, the 100 pound allow ance is now included in the specification, provided that certain conditions are met. For know n concentrated load locations, a new requirement is that the joist must be designed such that it does not required field applied w eb members.
- The reduction factor, Q, for crimped-end angle w eb members is now applicable to all crimped-end angles

intersecting at the first bottom chord panel point.

- For built-up web members comprised of two interconnected shapes, a modified slenderness ratio has been introduced.
- Changes have been made to the k factors for web and chord slenderness. The k factor has been reduced for out-of-plane slenderness of top and bottom chords, and the k factors for K-Series now match those of LH/DLH-Series joists.
- The K-Series (including KCS) bending exemption for interior panels of less than 24 inches has been removed.
- Joist Girder redundant web members in modified Warren web configuration that support direct loads have an additional design axial load of ½ of 1 % of the top chord axial force.
- Existing criteria for uncrimped single angle web members, which had previously only been published internally in the SJI Design Guides, are now included in the specification.
- The criteria for joint eccentricity have been merged to create criteria based upon the number of web components, but independent of the joist series.
- Criteria for bearing seat and bearing plate width, which had previously been only in the SJI Code of Standard Practice, has been added to the specification.
- The criteria for bearing seat depth, to achieve the end reaction farther over the support, has been redone for greater clarity.
- The existing "Minkoff" equation for determination of Erection bridging requirements has now been added to the specification.
- The bridging criteria is unchanged, but the previously separate K and LH tables have now been merged.
- Connection welds have been added to applicable bridging tables.
- The previous specification had almost no mention of seismic loads, so guidance has been added in Section 104.13.
- Welding during product assembly: This update brings SJI welding into compliance with AWS D1.1 and/or D1.3 with a modified acceptance criteria as permitted by AWS D1.1 Clause 6.8.

Cost Impact: Will increase the cost of construction
This code change proposal adopts the latest industry standard for steel joists. 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 steel joists.

Analysis: A review of the standard(s) proposed for inclusion in the code, SJI 100, 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.

S70-16 : 1604.3.3-MANLEY11133

S71-16

IBC: 1604.5.

Proponent : James Bela, representing Oregon Earthquakie 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 occup[ancy 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 inTable 1604.5 shall be used in lieu of ASCE 7, Table 1.5-1.

Reason:

- "Risk" is *subjective,ambiguous*, and *political* (or "w hat people w ant")
- "Occupancy" and "Use" are *objective* descriptions of row s I, II, III, and IV.

RISK is *forced* into this Table to imply that "w e have it altogether and that w e are, indeed, designing for risk." It is called RISK because of the use of Risk-Targeted" earthquake ground motions. But, let's be honest: w e really don't have it all together! It is here, because the code presently uses these "Risk-Targeted" earthquake ground motions. How ever, there is no supporting argument supporting its present use - the argument, if any, is based on *circular reas oning*, w herein 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 w e can replace the tensor (magnitude and directional) nature of real earthquake ground motion (magnitude, frequency content, duration, aftershocks) w ith a scalar quantiy (number value only); w hich, because it is fictitious, has little to do w ith assessing the true effects of scenario earthquakes that can impact the site. So-called "Risk-Targeted" earthquake ground motions w ere 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 w idely criticized for being non-transparent, subjective, blurring distinctions betw een assumptions and facts (and too often w ith a w eak understanding of the *limitations* of the model assumptions."

Furthermore, just *knowing* w hat the risk is . . . I mean w hat you *calculated* it to be; doesn't mean that you know "w hat 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 w ell-know n "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: <u>See also</u> BIBLIOGRAPHY in Proposal: **Figure 1613.3.1 RISK-TARGETED MCE$_R$**

Cost Breakdow n of Nonstructural Building Elements
Taghavi, S. and Miranda, E. (2003). Cost Breakdow n of Nonstructural Building Elements, PEER Report 2003/05, Pacific 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 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

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Low -Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Holliday, L. and Kang, T. (2014). "Low -Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions." J. Perform. Constr. Facil., 10.1061/(ASCE)CF.1943-5509.0000630, 04014141.

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Homeow ner's Guide to Earthquake Safety
California Seismic Safety Commission, 2005 Edition, 49 p.
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Retrofitting Questions and Answ ers
Earthquake Safety, Inc., 2015 (w eb based)
http://w w w .earthquakesafety.com/earthquake-retrofitting-faq.html

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

Earthquake Architecture w ebsite
http://w w w .iitk.ac.in/nicee/w cee/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.

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http://seaoc.org/bookstore/overview -perspective-1988-ubc-and-blue-book-applicable-1990-edition

1997 Uniform Building Code
http://w w w .iamcivilengineer.com/2014/08/dow nload-uniform-building-code-ubc-97.html

Robert E. Bachman and David R. Bonneville (2000)
The Seismic Provisions of the 1997 Uniform Building Code. Earthquake Spectra: February 2000, Vol. 16, No. 1, pp. 85-100.
http://w w w .earthquakespectra.org/doi/abs/10.1193/1.1586084?journalCode=eqsa

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 tw o-family dw ellings must be already built to w ithstand the lateral forces due to w ind; 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 tw o-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, 2013, 249 p. 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%.

S71-16 : 1604.5-BELA12867

S72-16

IBC: , 1604.5, 1615 (New), 202 (New).

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

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.

Bibliography: Chock, G., Robertson, I., Kriebel, D., Francis, M., and Nistor, I. (2013). *Tohoku, Japan, earthquake and tsunami of 2011:Performance of structures under tsunami loads,* ASCE, Reston, VA.
Chock, G. (2015). "The ASCE 7 Tsunami Loads and Effects Design Standard for the United States," Chapter 21, *Handbook of coastal disaster mitigation for engineers and planners,* Elsevier Science and Technology Books.

Robertson, I. N. (2016). *Tsunami Loads and Effects: Guide to the Tsunami Design Provisions of ASCE 7-16,* ASCE, Reston, VA.(in publication)

Chock, G., Yu, G., Thio, H.K., and Lynett, P.J. (2016). *Target Structural Reliability Analysis for Tsunami Hydrodynamic Loads of the ASCE 7 Standard*, Journal of Structural Engineering, ASCE, Reston, VA. (in publication)

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.

S72-16 : 1604.5-HAMBURGER11453

S73-16

IBC: 1604.5.

Proponent : James Bela, representing Oregon Earthquakie 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

~~RISK~~**OCCUPANCY AND USE CATEGORIES**	NATURE OF OCCUPANCY
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.
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.[a]
	• 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.~~[b]~~
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:

- 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$_R$**

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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 tw o-family dw ellings must be already built to w ithstand the lateral forces due to w ind; 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 tw o-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 flaw ed 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 alw ays 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, 2013, 249 p. NEHRP Consultants Joint Venture A partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering.

In general, w here costs might be increased, cost premiums above requirements for w ind tend to fall w ithin a range of +1-3%. For cases w here seismic requirements w ould be now additional to w hat previous codes either applied/neglected/failed to enforce, estimates probably w ould fall w ithin the range of 0.25 - 1%.

S73-16 : TABLE 1604.5-BELA12889

S74-16

IBC: 1604.5.

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

RISK CATEGORY	NATURE OF OCCUPANCY
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. <u>• Ground-mounted *photovoltaic panel systems* with no use underneath.</u>
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.[a]
	• 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.[b]
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.[b]
	• 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 emcompass buildings and structures that normally are unoccupied and that w ould result in negligible risk to the public should they fail."

4. Ground-mounted photovoltaic panel systems w ith no use underneath are not occupied. Facilities including ground-mounted photovoltaic panel systems are generally located w ithin a fenced area staffed by a small team of trained and qualified individuals w ho monitor performance and provide maintenance. Where structural failures have occurred, they have been triggered by w ind events. During w ind events, the trained and qualified staff can be expected to have hightened aw areness. 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 w ind turbine structures.

6. Ground-mounted photovoltaic panel systems represent a low er level of hazard to human life than w ind turbines in the event of failure. Therefore, it follow s that if ASCE recommends assignment of Risk Category II for w ind turbines, the low er Risk Category for ground-mounted solar is further justified.

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Cost Impact: Will not increase the cost of construction
Assignment as Risk Category I w ill not increase the cost of construction, and w ill avoid unnecessary increase in cost of construction ow ing to arbitrary assignments to higher Risk Categories.

S74-16 : TABLE 1604.5-CAIN10911

S75-16

IBC: 1604.5.

Proponent : John Williams, CBO, representing Adhoc Healthcare Committee (AHC@iccsafe.org); Carl Baldassarra, P.E., FSFPE, representing Code Technology Committee (CTC@iccsafe.org)

2015 International Building Code

Revise as follows:

TABLE 1604.5
RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

RISK CATEGORY	NATURE OF OCCUPANCY
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.
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, Condition 1 occupancies with ~~an occupant load of~~ 50 or more ~~resident~~ care recipients ~~but not having surgery or emergency treatment facilities.~~
	Group I-2, Condition 2 occupancies but not having emergency surgery or emergency treatment

	facilities.
	• Group I-3 occupancies.
	• Any other occupancy with an occupant load greater than 5,000.[a]
	• 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.[b]
IV	Buildings and other structures designated as essential facilities, including but not limited to:
	• Group I-2, Condition 2 occupancies having emergency surgery or emergency treatment facilities.
	Ambulatory care facilities having emergency 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.[b]

	• 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: Since Group I-2 has been split into two conditions in the 2015 I-Codes we are offering this clarification for which facilities this was intended to apply. Ambulatory care facilities or Group I-2 that only offer elective surgery should not be Category IV facilities. Category IV should be focused those occupancies that provide for emergency surgery and treatment of patients.
This is a joint proposal submitted by the ICC Ad Hoc Committee on Healthcare (AHC) and the ICC Code Technology Committee (CTC). The AHC was established by the ICC Board to evaluate and assess contemporary code issues relating to hospitals and ambulatory healthcare facilities. This is a joint effort between ICC and the American Society for Healthcare Engineering (ASHE), a subsidiary of the American Hospital Association, to eliminate duplication and conflicts in healthcare regulation. In 2014 and 2015 the ICC Ad Hoc Committee has held 4 open meetings and numerous Work Group meetings and conference calls for the current code development cycle which included members of the committees as well as any interested party to discuss and debate the proposed changes. Information on the AHC, including: meeting agendas; minutes; reports; resource documents; presentations; and all other materials developed in conjunction with the AHC effort can be downloaded from the AHC website at: AHC.
The two remaining CTC Areas of Study are Care Facilities and Elevator Lobbies/WTC Elevator issues. This proposal falls under the Care Facilities Area of Study. In 2014 and 2015 ICC CTC Committee has held 4 open meetings and numerous Work Group meetings and conference calls for the current code development cycle which included members of the committees as well as any interested party to discuss and debate the proposed changes. Information on the CTC, including: the sunset plan; meeting agendas; minutes; reports; resource documents; presentations; and all other materials developed in conjunction with the CTC effort can be downloaded from the CTC website CTC.

Cost Impact: Will not increase the cost of construction
This code change does not affect the actual requirements, rather it update the language to reflect current terminology and occupancy classes.

S75-16 : TABLE 1604.5-WILLIAMS11991

S76-16

IBC: 1604.5.1.

Proponent : Edward Kulik, representing Building Code Action Committee (bcac@iccsafe.org); Andrew Herseth, representing Federal Emergency Management Agency (andrew.herseth@fema.dhs.gov)

2015 International Building Code

Revise as follows:

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

> **Exception:** Where a *storm shelter* designed and constructed in accordance with ICC 500 is provided in a building, structure or portion thereof normally occupied for other purposes, the *risk category* for the normal occupancy of the building shall apply unless the *storm shelter* is a designated emergency shelter in accordance with Table 1604.5.

Reason: This code change proposal is intended to clarify the code, consistent with an ICC Committee Interpretation, IBC Interpretation 113-12, issued on January 25, 2013.

Risk categories are assigned to buildings to account for consequences and risks to human life (building occupants) in the event of a building failure. The intent is to assign higher risk categories, and hence higher design criteria, to buildings or structures that, if they experience a failure, would inhibit the availability of essential community services necessary to cope with an emergency situation and therefore have grave consequences to either the building occupants or the population around the building or structure that relies upon the provided services (such as a power station).

Community storm shelters are defined in the IBC and by the ICC-500 Standard for the Design and Construction of Storm Shelters (ICC 500) as shelters that either serve a non-residential use (i.e. not serving dwelling units) or serve dwelling units and provide a capacity exceeding 16 persons. This standard confirms that the area(s) of a building that have been constructed to the ICC 500 criteria have been specifically designed and constructed to provide life-safety protection for people seeking refuge from a high wind event.

ICC 500 compliant storm shelters are designed and constructed to account for extreme wind loads, have specific requirements for structural stability, vertical and horizontal load transfer, and egress that meet or exceed the basic requirements of the building code for property protection. Even if the storm shelter is not structurally separated from the host building, ICC 500 details the strength requirements for the members of the host building that connect to the storm shelter. Issues related to protection of occupants due to building collapse or failure have been considered by the ICC 500 and do not need to be addressed for the other portions of the facility.

ICC 500, Section 104.1: Rooms or spaces within other uses states: Where storm shelters are designated areas normally occupied for other purposes, the requirements of the applicable construction codes for the occupancy of the building shall apply unless otherwise stated in this standard.

Further, the storm shelter is a self-contained and defined space within the building that does not rely upon other portions of the building to provide life-safety protection from high winds, floods, or structural collapse. Hardening the other portions of the building that are outside the storm shelter or increasing the risk category for portions of the building that may be used to egress the space is not necessitated. The statements in Section 1604.5.1 regarding egress are to be applied when a building or portion thereof is being used to provide long-term, post-disaster response capabilities that would have considerable consequences to the community outside the occupied building and does not apply to ICC 500 compliant storm shelters.

The intent of the storm shelter is to provide short term, life safety in the event of a severe storm when the host building cannot. This protection provided by ICC 500 compliant storm shelters allows a building owner to provide a storm shelter in one portion of the structure as opposed to requiring the structure to meet the Risk Category IV provisions.

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**

Cost Impact: Will not increase the cost of construction
No increase in cost as this is a clarification only, based on the ICC Interpretations Committee IBC Interpretation 113-12.

S76-16 : 1604.5.1-KULIK10971

S77-16

IBC: 1605.1, 1605.2.1, 1605.3.2.

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 (ω) in the following equations shall be taken as 1.3. For other wind loads, (ω) 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, (ω) shall be taken as 1. When using these alternative load combinations to evaluate sliding, overturning and soil bearing at the soil-structure interface, the reduction of foundation overturning from Section 12.13.4 in ASCE 7 shall not be used. When using these alternative basic load combinations for proportioning foundations for loadings, which include seismic loads, the vertical seismic*load effect*,E_V, in Equation 12.4-4 of ASCE 7 is permitted to be taken equal to zero.

$D + L + (L_r \text{ or } S \text{ or } R)$	**(Equation 16-17)**
$D + L + 0.6\ \omega W$	**(Equation 16-18)**
$D + L + 0.6\ \omega W + S/2$	**(Equation 16-19)**
$D + L + S + 0.6\ \omega W/2$	**(Equation 16-20)**
$D + L + S + E/1.4$	**(Equation 16-21)**
$0.9D + E/1.4$	**(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/m^2) or less and roof live loads of 30 psf (1.44 kN/m^2) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.

Reason: This proposal is a coordination proposal to bring the 2018 IBC up to date with the provisions fo 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).

S77-16 : 1605.1-GOUPIL12914

S78-16

IBC: 1605.1, 1605.1.1, 1605.2, 1605.2.1, 1605.3, 1605.3.1, 1605.3.1.1, 1605.3.1.2, 1605.3.2, 1605.3.2.1, 1607.13.

Proponent : Ronald Hamburger, SIMPSON GUMPERTZ & HEGER (rohamburger@sgh.com)

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 of ASCE 7 where required by Section 12.2.5.2, 12.3.3.3 or 12.10.2.1 of ASCE 7. With the simplified procedure of ASCE 7 Section 12.14, the seismic load effects including overstrength factor in accordance with Section 12.14.3.2 of ASCE 7 shall be used.~~

~~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~~ Section 1605.2.

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:~~

~~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.~~

apply.

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~~ 2.3 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 ~~1605.3.1~~ 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 (ω) in the following equations shall be taken as 1.3. For other wind loads, (ω) 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, (ω) shall be taken as 1. When using these alternative load combinations to evaluate sliding, overturning and soil bearing at the soil-structure interface, the reduction of foundation overturning from Section 12.13.4 in ASCE 7 shall not be used. When using these alternative basic load combinations for proportioning foundations for loadings, which include seismic loads, the vertical seismic*load effect*,E_V, in Equation 12.4-4 of ASCE 7 is permitted to be taken equal to zero.

$D + L + (L_r \text{ or } S \text{ or } R)$	**(Equation 16-17)**
$D + L + 0.6\,\omega W$	**(Equation 16-18)**
$D + L + 0.6\,\omega W + S/2$	**(Equation 16-19)**
$D + L + S + 0.6\,\omega W/2$	**(Equation 16-20)**
$D + L + S + E/1.4$	**(Equation 16-21)**
$0.9D + E/1.4$	**(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/m^2) or less and roof live loads of 30 psf (1.44 kN/m^2) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.
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:~~

~~$1.4(D + F)$~~	~~**(Equation 16-1)**~~
~~$1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$~~	~~**(Equation 16-2)**~~
~~$1.2(D + F) + 1.6(L_r \text{ or } S \text{ or } R) + 1.6H + (f_1 L \text{ or } 0.5W)$~~	~~**(Equation 16-3)**~~
~~$1.2(D + F) + 1.0W + f_1 L + 1.6H + 0.5(L_r \text{ or } S \text{ or } R)$~~	~~**(Equation 16-4)**~~
~~$1.2(D + F) + 1.0E + f_1 L + 1.6H + f_2 S$~~	~~**(Equation 16-5)**~~
~~$0.9D + 1.0W + 1.6H$~~	~~**(Equation 16-6)**~~
~~$0.9(D + F) + 1.0E + 1.6H$~~	~~**(Equation 16-7)**~~

~~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.~~

~~**1605.2.1 Other loads.** Where flood loads, F_a, 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 in combination with other loads shall be determined 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:~~

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

~~**Exceptions:**~~

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

~~**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 accordance with Chapter 23.~~

~~**1605.3.1.2 Other loads.** Where flood loads, F_a, are to be considered in design, the load combinations of Section 2.4.2 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 2.4.4 of ASCE 7. Where an ice-sensitive structure is subjected to loads due to atmospheric icing, the load combinations of Section 2.4.3 of ASCE 7 shall be considered.~~

~~**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.~~

Revise as follows:

1607.13 Crane loads. The crane live load shall be the rated capacity of the crane. Design loads for the runway beams, including connections and support brackets, of moving bridge cranes and monorail cranes shall include the maximum wheel loads of the crane and the vertical impact, lateral and longitudinal forces induced by the moving crane. Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow loads or one-half of the wind load.

Reason: Since 2000, the IBC has contained three separate groups of load combinations included the follow ing:(1) Strength Load Combinations (1605.2); (2) Basic Allow able Stress Load Combinations (1605.3.1); and (3) Alternative Allow able Stress Load Combinations (1605.3.2). Tw o of these, the Strength Load Combinations and the Basic Allow able 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 betw een 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 w ill reduce the likelihood of design errors that many engineers have been making w hen applying the Basic Allow able Stress Combinations. The Alternative Allow able Stress Combinations permit the use of a 1/3 increase in allow able stresses w hen evaluating Load Combinations containing short-term transient loads including w ind and seismic. The Basic Allow able Stress Combinations do not do this, but instead, apply a factor fo 0.75 to the transient loads including live, snow , w ind, and seismic, w hen more than one of these loadings is considered simultaneously. The ASCE 7 combinations further permit increases in allow able stresses only w hen the material, such as w ood, 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 w hen using the Basie Allow able Stress Load Combinations because the strength of these materials does not have significant duration dependence. Unfortunately, and despite specific commentary w ithin the IBC to discourage this, many engineers routinely apply the 1/3 increase to all allow able stresses w hen designing using the Basic Allow able Stress Design Load Combinations. This creates a potentially dangerous situation in w hich safety margins of structures designed in this manner are substantially reduced.

By removing the transcription of the ASCE 7 Load Combinations from the IBC, in addition to avoiding duplication of nearly identical materials, w e hop to reduce the likelihood that designers w ill misapply the 1/3 increase factor applicable to the Alternate Allow able Stress Load Combinations Combinations. With the adoption of this proposal, the IBC w ill refer to ASCE 7 for the Strength and Basic Allow able Stress Load Combinations, w here there is no mention of the 1/3 increase factor. The Alternate Allow able Stress Load Combinations w ill remain in the IBC w ith 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 w ill remain available to them. The requirement that engineers reference ASCE 7 to determine the Strength and Basic Allow able 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 changed w ill 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* w ill 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).

S78-16 : 1605.1-HAMBURGER11551

S79-16

IBC: 1605.2.

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

2015 International Building Code

Revise as follows:

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)**
$1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$	**(Equation 16-2)**
$1.2(D + F) + 1.6(L_r \text{ or } S \text{ or } R) + 1.6H + (f_1L \text{ or } 0.5W)$	**(Equation 16-3)**
$1.2(D + F) + 1.0W + f_1L + 1.6H + 0.5(L_r \text{ or } S \text{ or } R)$	**(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)**

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.
3. <u>In equation 16-6, 1.0W is permitted to be reduced to 0.8W for the design of non-ballasted roof coverings complying with Section 1504.3.</u>

Reason: A load factor of 0.8, instead of 1.0 should be used for LRFD 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 300 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 0.8 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)(1) = 0.8.

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.

S79-16 : 1605.2-ENNIS12109

S80-16

IBC: 1605.3.1.

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

2015 International Building Code

Revise as follows:

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

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

Exceptions:

1. Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m^2) or less and roof live loads of 30 psf (1.44 kN/m^2) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.
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.
6. <u>In equation 16-15, 0.6W is permitted to be reduced to 0.5W for the design of non-ballasted roof coverings complying with Section 1504.3.</u>

Reason: A load factor of 0.5, instead of 0.6 should be used for allow able 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 w e are not targeting a 10-year service load as used for deflection design purposes, instead a 300 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 w ith life-expectancy of the component, first costs, and cost to replace. Designing for a 700-year return period w ind event w ith 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 w ind load factor of 0.5 instead w ill better ensure risk-consistent designs and encourage timely and economical roof replacements that should help improve overall roof covering performance. The difference in w ind speed betw een the 700-yr (Risk II map) and the 300-yr (Risk I map) is equivalent to a factor of approximately 1.2 on w ind load. This yields a corrected w ind 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.

S80-16 : 1605.3.1-ENNIS12110

S81-16

IBC: , 1605.4 (New), 1605.5 (New), 1605.6 (New), 1615 (New), 1615.1 (New), 1616 (New), 1616.1 (New), 1616.2 (New), 1616.3 (New), 1616.4 (New), 1616.5 (New), 1616.5.1 (New), 1617.1 (New), 1617.2 (New), 1617.3 (New), 1617.4 (New), 1617.5 (New), 1617.6 (New), 1617.7 (New), 1617.8 (New), 1617.8.1 (New), 1617.8.2 (New), 1617.8.3 (New), 1617.9 (New), 1909 (New), 1909.1 (New), 1909.2 (New), 1909.2.1 (New), 1909.2.1.1 (New), 1909.2.2 (New), 1909.2.3 (New), 1909.2.3.1 (New), 1909.2.4 (New), 1909.3 (New), 1909.3.1 (New), 1909.3.1.1 (New), 1909.3.1.2 (New), 1909.3.1.3 (New), 1909.3.2 (New), 1909.3.3 (New), 2112.2.1 (New), 2114 (New), 2114.1 (New), 2114.2 (New), 2114.2.1 (New), 2114.2.1.1 (New), 2114.2.1.2 (New), 2114.2.2 (New), 2114.2.3 (New), 2114.2.3.1 (New), 2212 (New), 2212.1 (New), 2212.2 (New), 2212.2.1 (New), 2212.2.2 (New), 2212.2.3 (New), 2212.2.4 (New), 2212.3 (New).

Proponent : Karl Rubenacker, Gilsanz Murray Steficek, representing Codes & Standards Committee, Structural Engineer's Association of New York (karl.rubenacker@gmsllp.com)

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 RiskCategory 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) **Equation16-24**

0.9D + Ak + 0.2W **Equation16-25**

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 Definiitions 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.~~

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 1617.8
Response Ratio and Rotation Limits [a]

Element	Response Ratio	Rotation
Concrete Slabs	$\mu < 10$	$\theta < 4^{\circ}$
Post-Tensioned Beams	$\mu < 2$	$\theta < 1.5^{\circ}$

Concrete Beams	μ < 20	θ < 6º
Concrete Columns	μ < 2	θ < 6º
Long Span Acoustical Deck	μ < 2	θ < 3º
Open Web Steel Joists	μ < 2	θ < 6º
Steel Beams	μ < 20	θ < 10º
Steel Columns	μ < 5	θ < 6º

For SI: 1 degree = 0.0 1745 rad.

a. 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

2112.2.1 Vertical Ties Column splices shall have an available tensile strength at least equal to the largest design gravity load reaction applied to the column at any floor level located within four floors below the splice.

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 de designed to avoid disproportionate collapse. Typical structures should have minimal prescriptive requirements. Significant structures should be designed w ith 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 outw eigh the very slight increase in construction cost.

S81-16 : 1605.4 (NEW)-RUBENACKER13488

S82-16

IBC: 1607.1.

Proponent : Scott Douglas, Douglas Engineering, representing Douglas Engineering (sdouglasscott@gmail.com)

2015 International Building Code

Revise as follows:

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_0, AND MINIMUM CONCENTRATED LIVE LOADS[g]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
1. Apartments (see residential)	—	—
2. Access floor systems		
Office use	50	2000
Computer use	100	2000
3. Armories and drill rooms	150[m]	—
4. Assembly areas		—
Fixed seats (fastened to floor)	60m	
Follow spot, projections and control rooms	50	
Lobbies	100m	
Movable seats	100m	
Stage floors	150m	
Platforms (assembly)	100m	
Other assembly areas	100m	
5. Balconies and decks[h]	Same as occupancy	—

	served	
6. Catwalks	40	300
7. Cornices	60	—
8. Corridors	100 Same as occupancy served except as indicated	—
First floor		
Other floors		
9. Dining rooms and restaurants	100m	—
10. Dwellings (see residential)	—	—
11. Elevator machine room and control room grating (on area of 2 inches by 2 inches)	—	300
12. Finish light floor plate construction (on area of 1 inch by 1 inch)	—	200
13. Fire escapes	100	—
On single-family dwellings only	40	
14. Garages (passenger vehicles only)	40m	Note a
Trucks and buses	See Section 1607.7	
15. Handrails, guards and grab bars	See Section 1607.8	
16. Helipads	See Section 1607.6	
17. Hospitals		
Corridors above first floor	80	1,000
Operating rooms, laboratories	60	1,000
Patient rooms	40	1,000

18. Hotels (see residential)	—	—
19. Libraries		
Corridors above first floor	80	1,000
Reading rooms	60	1,000
Stack rooms	150[b, m]	1,000
20. Manufacturing		
Heavy	250[m]	3,000
Light	125[m]	2,000
21. Marquees~~, except one and two family dwellings~~	~~75~~<u>20</u>[n]	—
22. Office buildings		
Corridors above first floor	80	2,000
File and computer rooms shall be designed for heavier loads based on anticipated occupancy	—	—
Lobbies and first-floor corridors	100	2,000
Offices	50	2,000

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
23. Penal institutions		—
Cell blocks	40	
Corridors	100	
24. Recreational uses:		—

Bowling alleys, poolrooms and similar uses	75 m	
Dance halls and ballrooms	100 m	
Gymnasiums	100 m	
Ice skating rink	250 m	
Reviewing stands, grandstands and bleachers	100 c, m	
Roller skating rink	100 m	
Stadiums and arenas with fixed seats (fastened to floor)	60 c, m	
25. Residential		—
One- and two-family dwellings		
Uninhabitable attics without storage i	10	
Uninhabitable attics with storage i, j, k	20	
Habitable attics and sleeping areas k	30	
~~Canopies, including marquees~~	~~20~~	
All other areas	40	
Hotels and multifamily		

dwellings					
Priv ate rooms and corridors serv ing them	40				
Public rooms m and corridors serv ing them	100				
26. Roof s					
All roof surf aces subject to main- tenance workers		300			
Awnings and canopies:					
Fabric construction supported by a skeleton structure	5 Nonreducible				
All other construction~~, except one and two f amily dwellings~~	20				
Ordinary f lat, pitched, and curv ed roof s (that are not occupiable)	20				
Primary roof members exposed to a work f loor					
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roof s ov er manuf acturing, storage warehouses, and repair garages		2,000	All other primary roof members		300
Occupiable roof s:					

Roof gardens	100	
Assembly areas	100 [m]	
All other similar areas	Note 1	Note 1
27. Schools		
Classrooms	40	1,000
Corridors above first floor	80	1,000
First-floor corridors	100	1,000
28. Scuttles, skylight ribs and accessible ceilings	—	200
29. Sidewalks, vehicular driveways and yards, subject to trucking	250 [d, m]	8,000 [e]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
30. Stairs and exits		
One- and two-family dwellings	40	300f
All other	100	300f
31. Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		—
Heavy	250m	
Light	125m	
32. Stores		

Retail		
First floor	100	1,000
Upper floors	75	1,000
Wholesale, all floors	125m	1,000
33. Vehicle barriers	See Section 1607.8.3	
34. Walkways and elevated platforms (other than exitways)	60	—
35. Yards and terraces, pedestrians	100^{m}	—

~~For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm^2,~~

~~1 square foot = 0.0929 m^2,~~

~~1 pound per square foot = 0.0479 kN/m^2, 1 pound = 0.004448 kN,~~

~~1 pound per cubic foot = 16 kg/m^3.~~

~~a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of this Table or the following concentrated loads: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 pounds acting on an area of 4$^1/_2$ inches by 4$^1/_2$ inches; (2) for mechanical parking structures without slab or deck that are used for storing passenger vehicles only, 2,250 pounds per wheel.~~

~~b. The loading applies to stack room floors that support nonmobile, double faced library book stacks, subject to the following limitations:~~

~~1. The nominal book stack unit height shall not exceed 90 inches;~~

~~2. The nominal shelf depth shall not exceed 12 inches for each face; and~~

~~3. Parallel rows of double faced book stacks shall be separated by aisles not less than 36 inches wide.~~

~~c. Design in accordance with ICC 300.~~

~~d. Other uniform loads in accordance with an approved method containing provisions for truck loadings shall be considered where appropriate.~~

~~e. The concentrated wheel load shall be applied on an area of 4.5 inches by 4.5 inches.~~

~~f. The minimum concentrated load on stair treads shall be applied on an area of 2 inches by 2 inches. This load need not be assumed to act concurrently with the uniform load.~~

~~g. Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design determined by the building official (see Section 1608).~~

~~h. See Section 1604.8.3 for decks attached to exterior walls.~~

~~i. Uninhabitable attics without storage are those where the maximum clear height between the joists and rafters is less than 42 inches, or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses. This live load need not be assumed to act concurrently with any other live load requirements.~~

~~j. Uninhabitable attics with storage are those where the maximum clear height between the joists and rafters is 42 inches or greater, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses.~~

~~The live load need only be applied to those portions of the joists or truss bottom chords where both of the following conditions are met:~~

~~i. The attic area is accessible from an opening not less than 20 inches in width by 30 inches in length that is located where the clear height in the attic is a minimum of 30 inches; and~~

~~ii. The slopes of the joists or truss bottom chords are no greater than two units vertical in 12 units horizontal.~~

~~The remaining portions of the joists or truss bottom chords shall be designed for a uniformly distributed concurrent live load of not less than 10 pounds per square foot.~~

~~k. Attic spaces served by stairways other than the pull down type shall be designed to support the minimum live load specified for habitable attics and sleeping rooms.~~

~~l. Areas of occupiable roofs, other than roof gardens and assembly areas, shall be designed for appropriate loads as approved by the building official. Unoccupied landscaped areas of roofs shall be designed in accordance with Section 1607.12.3.~~

~~m. Live load reduction is not permitted unless specific exceptions of Section 1607.10 apply.~~

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm^2,

1 square foot = 0.0929 m^2,

1 pound per square foot = 0.0479 kN/m^2, 1 pound = 0.004448 kN,

1 pound per cubic foot = 16 kg/m^3.

a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of this Table or the following concentrated loads: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 pounds acting on an area of $4^1/_2$ inches by $4^1/_2$ inches; (2) for mechanical parking structures without slab or deck that are used for storing passenger vehicles only, 2,250 pounds per wheel.

b. The loading applies to stack room floors that support nonmobile, double-faced library book stacks, subject to the following limitations:

1. The nominal book stack unit height shall not exceed 90 inches;

2. The nominal shelf depth shall not exceed 12 inches for each face; and

3. Parallel rows of double-faced book stacks shall be separated by aisles not less than 36 inches wide.

c. Design in accordance with ICC 300.

d. Other uniform loads in accordance with an approved method containing provisions for truck loadings shall be considered where appropriate.

e. The concentrated wheel load shall be applied on an area of 4.5 inches by 4.5 inches.

f. The minimum concentrated load on stair treads shall be applied on an area of 2 inches by 2 inches. This load need not be assumed to act concurrently with the uniform load.

g. Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design determined by the building official (see Section 1608).

h. See Section 1604.8.3 for decks attached to exterior walls.

i. Uninhabitable attics without storage are those where the maximum clear height between the joists and rafters is less than 42 inches, or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses. This live load need not be assumed to act concurrently with any other live load requirements.

j. Uninhabitable attics with storage are those where the maximum clear height between the joists and rafters is 42 inches or greater, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses.

The live load need only be applied to those portions of the joists or truss bottom chords where both of the following conditions are met:

i. The attic area is accessible from an opening not less than 20 inches in width by 30 inches in length that is located where the clear height in the attic is a minimum of 30 inches; and

ii. The slopes of the joists or truss bottom chords are no greater than two units vertical in 12 units horizontal.

The remaining portions of the joists or truss bottom chords shall be designed for a uniformly distributed concurrent live load of not less than 10 pounds per square foot.

k. Attic spaces served by stairways other than the pull-down type shall be designed to support the minimum live load specified for habitable attics and sleeping rooms.

l. Areas of occupiable roofs, other than roof gardens and assembly areas, shall be designed for appropriate loads as approved by the building official. Unoccupied landscaped areas of roofs shall be designed in accordance with Section 1607.12.3.

m. Live load reduction is not permitted unless specific exceptions of Section 1607.10 apply.

n. Marquees shall include any temporary signage or other variable loads in addition to the uniform live load specified.

Reason: This proposal harmanizes marquee, awning, and canopy live load criteria with current ASCE 7 live load criteria. The term "Marquee" is an archaic term as stated in ASCE 7 Commentary Section C4.3.1. In today's design and construction language the term "Marquee" has been replaced by the term "entry canopy". In modern construction there is often a continuous canopy along the front of multiple retail stores with an entry at each individual store. There is no justification for a change in canopy live load once the continuous canopy is less than 10 feet from an entry, or operable opening as per the current definition for a marquee in IBC Chapter 2. The definition for marquee can not be corrected until the next development cycle of Group A Code Change Proposals. Footnote n is proposed for addition to account for any special loads at a marquee (in reality an entry canopy in current design practice).

Cost Impact: Will not increase the cost of construction

The harmonization of marquee, awning, and canopy live load criteria with ASCE 7 will not increase, and may very minimally decrease the cost of construction.

S82-16 : 1607.1-DOUGLAS13471

S83-16

IBC: 1607.1.

Proponent : Michael Anthony, University of Michigan, representing University of Michigan (maanthon@umich.edu)

2015 International Building Code

Revise as follows:

TABLE 1607.1 (1607.1)
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS[g]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
1. Apartments (see residential)	—	—
2. Access floor systems		
Office use	50	2000
Computer use	100	2000
3. Armories and drill rooms	150[m]	—
4. Assembly areas		—
Fixed seats (fastened to floor)	60m	
Follow spot, projections and control rooms	50	
Lobbies	100m	
Movable seats	100m	
Stage floors	150m	
Platforms (assembly)	100m	
Other assembly areas	100m	
5. Balconies and decks[h]	Same as occupancy	—

	served	
6. Catwalks	40	300
7. Cornices	60	—
8. Corridors	100 Same as occupancy served except as indicated	—
First floor		
Other floors		
9. Dining rooms and restaurants	100m	—
10. Dwellings (see residential)	—	—
11. Elevator machine room and control room grating (on area of 2 inches by 2 inches)	—	300
12. Finish light floor plate construction (on area of 1 inch by 1 inch)	—	200
13. Fire escapes	100	—
On single-family dwellings only	40	
14. Garages (passenger vehicles only)	40m	Note a
Trucks and buses	See Section 1607.7	
15. Handrails, guards and grab bars	See Section 1607.8	
16. Helipads	See Section 1607.6	
17. Hospitals		
Corridors above first floor	80	1,000
Operating rooms, laboratories	60	1,000
Patient rooms	40	1,000

18. Hotels (see residential)	—	—
19. Libraries		
Corridors above first floor	80	1,000
Reading rooms	60	1,000
Stack rooms	150[b, m]	1,000
20 Laboratories Animal Research Facility Animal Research Facility with primates Aquatic Facilities Cagewash Frozen Storage Refrigeration Areas Operating Rooms Other laboratory uses	125 150 150 200 200 100 100	
20. Manufacturing		
Heavy	250[m]	3,000
Light	125[m]	2,000
21. Marquees, except one-and two-family dwellings	75	—
22. Office buildings		
Corridors above first floor	80	2,000
File and computer rooms shall be designed for heavier loads based on anticipated occupancy	—	—

Lobbies and first-floor corridors	100	2,000
Offices	50	2,000

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
23. Penal institutions		—
Cell blocks	40	
Corridors	100	
24. Recreational uses:		—
Bowling alleys, poolrooms and similar uses	75 m	
Dance halls and ballrooms	100 m	
Gymnasiums	100 m	
Ice skating rink	250 m	
Reviewing stands, grandstands and bleachers	100 c, m	
Roller skating rink	100 m	
Stadiums and arenas with fixed seats (fastened to floor)	60 c, m	
25. Residential		—
One- and two-family dwellings		
Uninhabitable attics without	10	

storage i		
Uninhabitable attics with storage i, j, k	20	
Habitable attics and sleeping areas k	30	
Canopies, including marquees	20	
All other areas	40	
Hotels and multifamily dwellings		
Private rooms and corridors serving them	40	
Public rooms m and corridors serving them	100	
26. Roofs		
All roof surfaces subject to main- tenance workers		300
Awnings and canopies:		
Fabric construction supported by a skeleton structure	5 Nonreducible	
All other construction, except one-and two-family dwellings	20	
Ordinary flat, pitched, and curved roofs (that are not	20	

occupiable)					
Primary roof members exposed to a work floor					
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages		2,000	All other primary roof members		300
Occupiable roofs:					
Roof gardens	100				
Assembly areas	100 [m]				
All other similar areas	Note 1	Note 1			
27. Schools					
Classrooms	40	1,000			
Corridors above first floor	80	1,000			
First-floor corridors	100	1,000			
28. Scuttles, skylight ribs and accessible ceilings	—	200			
29. Sidewalks, vehicular driveways and yards, subject to trucking	250 [d, m]	8,000 [e]			

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)

30. Stairs and exits		
One- and two-family dwellings	40	300f
All other	100	300f
31. Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		—
Heavy	250m	
Light	125m	
32. Stores		
Retail		
First floor	100	1,000
Upper floors	75	1,000
Wholesale, all floors	125m	1,000
33. Vehicle barriers	See Section 1607.8.3	
34. Walkways and elevated platforms (other than exitways)	60	—
35. Yards and terraces, pedestrians	100m	—

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm ^{2},

1 square foot = 0.0929 m ^{2},

1 pound per square foot = 0.0479 kN/m ^{2}, 1 pound = 0.004448 kN,

1 pound per cubic foot = 16 kg/m ^{3}.

a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of this Table or the following concentrated loads: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 pounds acting on an area of $4\,^1/_2$ inches by $4\,^1/_2$ inches; (2) for mechanical parking structures without slab or deck that are used for storing passenger vehicles only, 2,250 pounds per wheel.

b. The loading applies to stack room floors that support nonmobile, double-faced library book stacks, subject to the following limitations:

1. The nominal book stack unit height shall not exceed 90 inches;

2. The nominal shelf depth shall not exceed 12 inches for each face; and

3. Parallel rows of double-faced book stacks shall be separated by aisles not less than 36 inches wide.

c. Design in accordance with ICC 300.

d. Other uniform loads in accordance with an approved method containing provisions for truck loadings shall be considered where appropriate.

e. The concentrated wheel load shall be applied on an area of 4.5 inches by 4.5 inches.

f. The minimum concentrated load on stair treads shall be applied on an area of 2 inches by 2 inches. This load need not be assumed to act concurrently with the uniform load.

g. Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design determined by the building official (see Section 1608).

h. See Section 1604.8.3 for decks attached to exterior walls.

i. Uninhabitable attics without storage are those where the maximum clear height between the joists and rafters is less than 42 inches, or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses. This live load need not be assumed to act concurrently with any other live load requirements.

j. Uninhabitable attics with storage are those where the maximum clear height between the joists and rafters is 42 inches or greater, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses.

The live load need only be applied to those portions of the joists or truss bottom chords where both of the following conditions are met:

i. The attic area is accessible from an opening not less than 20 inches in width by 30 inches in length that is located where the clear height in the attic is a minimum of 30 inches; and

ii. The slopes of the joists or truss bottom chords are no greater than two units vertical in 12 units horizontal.

The remaining portions of the joists or truss bottom chords shall be designed for a uniformly distributed concurrent live load of not less than 10 pounds per square foot.

k. Attic spaces served by stairways other than the pull-down type shall be designed to support the minimum live load specified for habitable attics and sleeping rooms.

l. Areas of occupiable roofs, other than roof gardens and assembly areas, shall be designed for appropriate loads as approved by the building official. Unoccupied landscaped areas of roofs shall be designed in accordance with Section 1607.12.3.

m. Live load reduction is not permitted unless specific exceptions of Section 1607.10 apply.

Reason: Square-footage in the education and healthcare facilities industry used for laboratory research is significant. The absence of guidance on live loading for this use/occupancy class in the International Building Code is noteworthy. The specific information correlating use/occupancy/type of space for laboratories originates from a National Institutes of Health (NIH) Office of Research Facilities requirement for buildings that the NIH funds and is available at this link:
http://orf.od.nih.gov/PoliciesAndGuidelines/BiomedicalandAnimalResearchFacilitiesDesignPoliciesandGuidelines/DRMHT2StructuralLoadRequirements.aspx

From the NIH/ORF Table 5-2 we have extracted only the occupancy/use categories that apply to laboratories in our industry.

Remarks:

1. This table may not be appropriate for all non-NIH laboratories but it is a starting point. We were unable to locate the research (if any) that provided the technical substantiation for these numbers. We did, however, consult with many structural engineers during 2014 to discuss this proposal. Since the cdpAccess system did not process edits to the table, our tabulation is shown more fully here:

Laboratories / 100

Animal Research Facility / 125

Animal Research Facility With Primates / 150

Aquatic Facilities / 150

Cagewash / 200

Frozen Storage, Refrigeration Areas / 200

Operating Rooms / 100

The link to the NIH/ORF shows the formatted table we are copying.

3. We used an on-line kPa to psf unit converter and rounded up

4. University of Michigan Plant Operations is willing to provide impetus for research to get these numbers right and will coordinate with the ASCE Structural Engineering Institute

ASCE Structural Engineering Institute Table 4-1 which also does not contain requirements for laboratories. We have communicated with the Live Loading Subcommittee about what appears to be a gap and/or a correlation issue between IBC Table 1607 and ASCE SEI Minimum Distributed Live Load Table 4-1

Bibliography: The specific information correlating use/occupancy/type of space for laboratories originates from a National Institutes of Health (NIH) Office of Research Facilities requirement for buildings that the NIH funds and is available at these links:
http://orf2.od.nih.gov/orfphones.asp?division=OD

http://orf.od.nih.gov/PoliciesAndGuidelines/BiomedicalandAnimalResearchFacilitiesDesignPoliciesandGuidelines/DRMHT2StructuralLoadRequirements.aspx

From the NIH/ORF Table 5-2 we have extracted only the occupancy/use categories that apply to laboratories in our industry.

Significant professional expertise was provided by structural engineers at Wiss, Janney, Elstner Associates, Inc., Thornton Tomasetti, Purdue University and the University of Michigan

Cost Impact: Will not increase the cost of construction
Cost impact could go either way.
Concepts that recommend "designing for future uses" need to be approached with some care because of the likelihood that future uses that require higher structural loading design may never materialize. The standards and code of many building industry disciplines – electrical, mechanical, etc. -- are found to over-state the need the design for future use that never occurs; thereby stranding investment. If an Owner wants to change the occupancy/use class so significantly that the building's structure needs re-design, then the new project to re-purpose the use/occupancy classification should bear the cost of additional structural loading capability.

S83-16 : TABLE 1607.1-ANTHONY5121

S84-16

IBC: 1607.1.

Proponent : Scott Douglas, Douglas Engineering (sdouglasscott@gmail.com)

2015 International Building Code

Revise as follows:

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_0, AND MINIMUM CONCENTRATED LIVE LOADS[g]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
1. Apartments (see residential)	—	—
2. Access floor systems		
Office use	50	2000
Computer use	100	2000
3. Armories and drill rooms	150[m]	—
4. Assembly areas		—
Fixed seats (fastened to floor)	60m	
Follow spot, projections and control rooms	50	
Lobbies	100m	
Movable seats	100m	
Stage floors	150m	
Platforms (assembly)	100m	
Other assembly areas	100m	
5. Balconies and decks[h]	Same as occupancy served	—

5.1 Canopies	20	300
6. Catwalks	40	300
7. Cornices	60	—
8. Corridors	100 Same as occupancy served except as indicated	—
First floor		
Other floors		
9. Dining rooms and restaurants	100[m]	—
10. Dwellings (see residential)	—	—
11. Elevator machine room and control room grating (on area of 2 inches by 2 inches)	—	300
12. Finish light floor plate construction (on area of 1 inch by 1 inch)	—	200
13. Fire escapes	100	—
On single-family dwellings only	40	
14. Garages (passenger vehicles only)	40m	Note a
Trucks and buses	See Section 1607.7	
15. Handrails, guards and grab bars	See Section 1607.8	
16. Helipads	See Section 1607.6	
17. Hospitals		
<u>First-floor corridors</u>	<u>100</u>	
Corridors above first floor	80	1,000
Operating rooms, laboratories	60	1,000

Patient rooms	40	1,000
18. Hotels (see residential)	—	—
19. Libraries		
First-floor corridors	100	1,000
Corridors above first floor	80	1,000
Reading rooms	60	1,000
Stack rooms	150[b, m]	1,000
20. Manufacturing		
Heavy	250[m]	3,000
Light	125[m]	2,000
21. Marquees, except one-and two-family dwellings	75	—
22. Office buildings		
Corridors above first floor	80	2,000
File and computer rooms shall be designed for heavier loads based on anticipated occupancy	—	—
Lobbies and first-floor corridors	100	2,000
Offices	50	2,000

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
23. Penal institutions		—

Cell blocks	40	
Corridors	100	
24. Recreational uses:		—
Bowling alleys, poolrooms and similar uses	75 m	
Dance halls and ballrooms	100 m	
Gymnasiums	100 m	
Ice skating rink	250 m	
Reviewing stands, grandstands and bleachers	100 c, m	
Roller skating rink	100 m	
Stadiums and arenas with fixed seats (fastened to floor)	60 c, m	
25. Residential		—
One- and two-family dwellings		
Uninhabitable attics without storage i	10	
Uninhabitable attics with storage i, j, k	20	
Habitable attics and sleeping areas k	30	
Canopies, including	20	

marquees					
All other areas	40				
Hotels and multifamily dwellings					
Private rooms and corridors serving them	40				
Public rooms m and corridors serving them	100				
26. Roofs					
All roof surfaces subject to main- tenance workers		300			
Awnings and canopies:					
Fabric construction supported by a skeleton structure	5 Nonreducible				
All other construction	20				
Ordinary flat, pitched, and curved roofs (that are not occupiable)	20				
Primary roof members exposed to a work floor					
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair		2,000	All other primary roof members		300

garages		
Occupiable roofs:		
Roof gardens	100	
Assembly areas	100 [m]	
All other similar areas	Note 1	Note 1
27. Schools		
Classrooms	40	1,000
Corridors above first floor	80	1,000
First-floor corridors	100	1,000
28. Scuttles, skylight ribs and accessible ceilings	—	200
29. Sidewalks, vehicular driveways and yards, subject to trucking	250 [d, m]	8,000 [e]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
30. Stairs and exits		
One- and two-family dwellings	40	300f
All other	100	300f
31. Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		—
Heavy	250m	

Light	125m	
32. Stores		
Retail		
First floor	100	1,000
Upper floors	75	1,000
Wholesale, all floors	125m	1,000
33. Vehicle barriers	See Section 1607.8.3	
34. Walkways and elevated platforms (other than exitways)	60	—
35. Yards and terraces, pedestrians	100[m]	—

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm ^{2},

1 square foot = 0.0929 m ^{2},

1 pound per square foot = 0.0479 kN/m ^{2}, 1 pound = 0.004448 kN,

1 pound per cubic foot = 16 kg/m ^{3}.

a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of this Table or the following concentrated loads: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, a 3,000 ~~pounds~~ pound jack load acting on an area of 4 $^1/_2$ inches by 4 $^1/_2$ inches along any point between the center-line of the wheels along the axle; ~~(~~ (2) for mechanical parking structures without slab or deck that are used for storing passenger vehicles only, 2,250 pounds per wheel.

b. The loading applies to stack room floors that support nonmobile, double-faced library book stacks, subject to the following limitations:

1. The nominal book stack unit height shall not exceed 90 inches;
2. The nominal shelf depth shall not exceed 12 inches for each face; and
3. Parallel rows of double-faced book stacks shall be separated by aisles not less than 36 inches wide.

c. Design in accordance with ICC 300.

d. Other uniform loads in accordance with an approved method containing provisions for truck loadings shall be considered where appropriate.

e. The concentrated wheel load shall be applied on an area of 4.5 inches by 4.5 inches.

f. The minimum concentrated load on stair treads shall be applied on an area of 2 inches by 2 inches. This load need not be assumed to act concurrently with the uniform load.

g. Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design determined by the building official (see Section 1608).

h. See Section 1604.8.3 for decks attached to exterior walls.

i. Uninhabitable attics without storage are those where the maximum clear height between the joists and rafters is less than 42 inches, or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses. This live load need not be assumed to

act concurrently with any other live load requirements.

j. Uninhabitable attics with storage are those where the maximum clear height between the joists and rafters is 42 inches or greater, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses.

The live load need only be applied to those portions of the joists or truss bottom chords where both of the following conditions are met:

i. The attic area is accessible from an opening not less than 20 inches in width by 30 inches in length that is located where the clear height in the attic is a minimum of 30 inches; and

ii. The slopes of the joists or truss bottom chords are no greater than two units vertical in 12 units horizontal.

The remaining portions of the joists or truss bottom chords shall be designed for a uniformly distributed concurrent live load of not less than 10 pounds per square foot.

k. Attic spaces served by stairways other than the pull-down type shall be designed to support the minimum live load specified for habitable attics and sleeping rooms.

l. Areas of occupiable roofs, other than roof gardens and assembly areas, shall be designed for appropriate loads as approved by the building official. Unoccupied landscaped areas of roofs shall be designed in accordance with Section 1607.12.3.

m. Live load reduction is not permitted unless specific exceptions of Section 1607.10 apply.

Reason: Table 1607.1 Items 17 and 19 - First-level corridor live loading needs to be specified for Hospitals and Libraries just as it currently is specified for Office Buildings and Schools.
Footnote a - Clarity is required regarding the specified 3000 pound concentrated load for passanger vehicle garages. Some interpretations, such as Trus Joist / Weyerhaeuser Technical Bulletin TB-105 and ASCE 7 Commentary Section C4.4 regard the 3000 pound point load to be a single jack load at any location along the axle. Other interpretations, such as an article in Concrete International's September 2002 issue, regard the 3000 pound concentrated load to be a concentrated load at each wheel.

Bibliography:

- Technical Bulletin TB-105 - Truss Joist Weyerhaeuser - 2014 - *www.woodbywy.com/document/**tb-105**/*
- SUVs and Parking Structures: Is there a Weight Problem - Mary S. Smith and Anthony P. Chrest, *Concrete International,* September 2002, pgs 49-59 - http://www.walkerparking.com/wp-content/uploads/2013/02/smith_suv.pdf

Cost Impact: Will not increase the cost of construction
This clarification for both the first-floor corridor live load requirements and the concentrated load requirements for passenger vehicle garages are editorial changes and will not increase the cost of construction.

S84-16 : TABLE 1607.1-DOUGLAS13148

S85-16

IBC: 1607.1.

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

2015 International Building Code

Revise as follows:

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_0, AND MINIMUM CONCENTRATED LIVE LOADS[g]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
1. Apartments (see residential)	—	—
2. Access floor systems		
Office use	50	2000
Computer use	100	2000
3. Armories and drill rooms	150[m]	—
4. Assembly areas		—
Fixed seats (fastened to floor)	60m	
Follow spot, projections and control rooms	50	
Lobbies	100m	
Movable seats	100m	
Stage floors	150m	
Platforms (assembly)	100m	
Other assembly areas	100m	
	1.5 times the live	

5. Balconies and decks[h]	load for the area served. Not required to exceed 100 psf. ~~Same as occupancy served~~	—
6. Catwalks	40	300
7. Cornices	60	—
8. Corridors	100 Same as occupancy served except as indicated	—
First floor		
Other floors		
9. Dining rooms and restaurants	100[m]	—
10. Dwellings (see residential)	—	—
11. Elevator machine room and control room grating (on area of 2 inches by 2 inches)	—	300
12. Finish light floor plate construction (on area of 1 inch by 1 inch)	—	200
13. Fire escapes	100	—
On single-family dwellings only	40	
14. Garages (passenger vehicles only)	40m	Note a
Trucks and buses	See Section 1607.7	
15. Handrails, guards and grab bars	See Section 1607.8	
16. Helipads	See Section 1607.6	
17. Hospitals		
Corridors above first floor	80	1,000

Operating rooms, laboratories	60	1,000
Patient rooms	40	1,000
18. Hotels (see residential)	—	—
19. Libraries		
Corridors above first floor	80	1,000
Reading rooms	60	1,000
Stack rooms	150[b, m]	1,000
20. Manufacturing		
Heavy	250[m]	3,000
Light	125[m]	2,000
21. Marquees, except one-and two-family dwellings	75	—
22. Office buildings		
Corridors above first floor	80	2,000
File and computer rooms shall be designed for heavier loads based on anticipated occupancy	—	—
Lobbies and first-floor corridors	100	2,000
Offices	50	2,000

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm ^{2} ,

1 square foot = 0.0929 m ^{2} ,

1 pound per square foot = 0.0479 kN/m ^{2} , 1 pound = 0.004448 kN,

1 pound per cubic foot = 16 kg/m ^{3} .

a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of this Table or the following concentrated loads: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 pounds acting on an area of $4\,^{1}/_{2}$ inches by $4\,^{1}/_{2}$ inches; (2) for mechanical parking structures without slab or deck that are used for storing passenger vehicles only, 2,250 pounds per wheel.

b. The loading applies to stack room floors that support nonmobile, double-faced library book stacks, subject to the following limitations:

1. The nominal book stack unit height shall not exceed 90 inches;

2. The nominal shelf depth shall not exceed 12 inches for each face; and

3. Parallel rows of double-faced book stacks shall be separated by aisles not less than 36 inches wide.

c. Design in accordance with ICC 300.

d. Other uniform loads in accordance with an approved method containing provisions for truck loadings shall be considered where appropriate.

e. The concentrated wheel load shall be applied on an area of 4.5 inches by 4.5 inches.

f. The minimum concentrated load on stair treads shall be applied on an area of 2 inches by 2 inches. This load need not be assumed to act concurrently with the uniform load.

g. Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design determined by the building official (see Section 1608).

h. See Section 1604.8.3 for decks attached to exterior walls.

i. Uninhabitable attics without storage are those where the maximum clear height between the joists and rafters is less than 42 inches, or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses. This live load need not be assumed to act concurrently with any other live load requirements.

j. Uninhabitable attics with storage are those where the maximum clear height between the joists and rafters is 42 inches or greater, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses.

The live load need only be applied to those portions of the joists or truss bottom chords where both of the following conditions are met:

i. The attic area is accessible from an opening not less than 20 inches in width by 30 inches in length that is located where the clear height in the attic is a minimum of 30 inches; and

ii. The slopes of the joists or truss bottom chords are no greater than two units vertical in 12 units horizontal.

The remaining portions of the joists or truss bottom chords shall be designed for a uniformly distributed concurrent live load of not less than 10 pounds per square foot.

k. Attic spaces served by stairways other than the pull-down type shall be designed to support the minimum live load specified for habitable attics and sleeping rooms.

l. Areas of occupiable roofs, other than roof gardens and assembly areas, shall be designed for appropriate loads as approved by the building official. Unoccupied landscaped areas of roofs shall be designed in accordance with Section 1607.12.3.

m. Live load reduction is not permitted unless specific exceptions of Section 1607.10 apply.

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).
Table 1607.1 - Proposed modifications to Table 1607.1 modify the live loads on decks to 1.5 times the live load for the area served, but not required to exceed 100 psf. This proposed change will align the requirements of the code with the standard ASCE 7, which has included this provision in the 2010 edition. Given that balconies and decks can be places of assembly, it is reasonable that the required live load is not to exceed the specified the uniform load required for Assembly Areas.

Cost Impact: Will increase the cost of construction
The proposed changes will 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).

S85-16 : TABLE 1607.1-GOUPIL12930

S86-16

IBC: 1607.1.

Proponent : Edwin Huston, representing National Council of Structural Engineers' Associations (NCSEA) (huston@smithhustoninc.com); Dennis Richardson, representing American Wood Council (drichardson@awc.org); Karl Rubenacker, representing Codes & Standards Committee, Structural Engineer's Association of New York (karl.rubenacker@gmsllp.com)

2015 International Building Code

Revise as follows:

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_0, AND MINIMUM CONCENTRATED LIVE LOADS[g]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
1. Apartments (see residential)	—	—
2. Access floor systems		
Office use	50	2000
Computer use	100	2000
3. Armories and drill rooms	150[m]	—
4. Assembly areas		—
Fixed seats (fastened to floor)	60m	
Follow spot, projections and control rooms	50	
Lobbies	100m	
Movable seats	100m	
Stage floors	150m	
Platforms (assembly)	100m	
Other assembly areas		

	100m	
5. Balconies and decks[h]	~~Same as occupancy served~~ 1.5 times the live load for the area served. Not required to exceed 100 psf (4.79 kN/m2)	—
6. Catwalks	40	300
7. Cornices	60	—
8. Corridors	100 Same as occupancy served except as indicated	—
First floor		
Other floors		
9. Dining rooms and restaurants	100[m]	—
10. Dwellings (see residential)	—	—
11. Elevator machine room and control room grating (on area of 2 inches by 2 inches)	—	300
12. Finish light floor plate construction (on area of 1 inch by 1 inch)	—	200
13. Fire escapes	100	—
On single-family dwellings only	40	
14. Garages (passenger vehicles only)	40m	Note a
Trucks and buses	See Section 1607.7	
15. Handrails, guards and grab bars	See Section 1607.8	
16. Helipads	See Section 1607.6	

17. Hospitals		
Corridors above first floor	80	1,000
Operating rooms, laboratories	60	1,000
Patient rooms	40	1,000
18. Hotels (see residential)	—	—
19. Libraries		
Corridors above first floor	80	1,000
Reading rooms	60	1,000
Stack rooms	150[b, m]	1,000
20. Manufacturing		
Heavy	250[m]	3,000
Light	125[m]	2,000
21. Marquees, except one-and two-family dwellings	75	—
22. Office buildings		
Corridors above first floor	80	2,000
File and computer rooms shall be designed for heavier loads based on anticipated occupancy	—	—
Lobbies and first-floor corridors	100	2,000
Offices	50	2,000

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)

23. Penal institutions		—
Cell blocks	40	
Corridors	100	
24. Recreational uses:		—
Bowling alleys, poolrooms and similar uses	75 m	
Dance halls and ballrooms	100 m	
Gymnasiums	100 m	
Ice skating rink	250 m	
Reviewing stands, grandstands and bleachers	100 c, m	
Roller skating rink	100 m	
Stadiums and arenas with fixed seats (fastened to floor)	60 c, m	
25. Residential		—
One- and two-family dwellings		
Uninhabitable attics without storage i	10	
Uninhabitable attics with storage i, j, k	20	
Habitable attics and sleeping areas k	30	

Canopies, including marquees	20				
All other areas	40				
Hotels and multifamily dwellings					
Private rooms and corridors serving them	40				
Public rooms m and corridors serving them	100				
26. Roofs					
All roof surfaces subject to main- tenance workers		300			
Awnings and canopies:					
Fabric construction supported by a skeleton structure	5 Nonreducible				
All other construction, except one-and two-family dwellings	20				
Ordinary flat, pitched, and curved roofs (that are not occupiable)	20				
Primary roof members exposed to a work floor					
Single panel point of lower chord of roof trusses or any point along primary		2,000	All other primary roof members		300

structural members supporting roofs over manufacturing, storage warehouses, and repair garages		
Occupiable roofs:		
Roof gardens	100	
Assembly areas	100 [m]	
All other similar areas	Note 1	Note 1
27. Schools		
Classrooms	40	1,000
Corridors above first floor	80	1,000
First-floor corridors	100	1,000
28. Scuttles, skylight ribs and accessible ceilings	—	200
29. Sidewalks, vehicular driveways and yards, subject to trucking	250 [d, m]	8,000 [e]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
30. Stairs and exits		
One- and two-family dwellings	40	300f
All other	100	300f
31. Storage warehouses (shall be designed for heavier loads if		

required for anticipated storage)		—
Heavy	250m	
Light	125m	
32. Stores		
Retail		
First floor	100	1,000
Upper floors	75	1,000
Wholesale, all floors	125m	1,000
33. Vehicle barriers	See Section 1607.8.3	
34. Walkways and elevated platforms (other than exitways)	60	—
35. Yards and terraces, pedestrians	100m	—

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm ^{2},

1 square foot = 0.0929 m ^{2},

1 pound per square foot = 0.0479 kN/m ^{2}, 1 pound = 0.004448 kN,

1 pound per cubic foot = 16 kg/m ^{3}.

a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of this Table or the following concentrated loads: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 pounds acting on an area of $4\,^1/_2$ inches by $4\,^1/_2$ inches; (2) for mechanical parking structures without slab or deck that are used for storing passenger vehicles only, 2,250 pounds per wheel.

b. The loading applies to stack room floors that support nonmobile, double-faced library book stacks, subject to the following limitations:

1. The nominal book stack unit height shall not exceed 90 inches;

2. The nominal shelf depth shall not exceed 12 inches for each face; and

3. Parallel rows of double-faced book stacks shall be separated by aisles not less than 36 inches wide.

c. Design in accordance with ICC 300.

d. Other uniform loads in accordance with an approved method containing provisions for truck loadings shall be considered where appropriate.

e. The concentrated wheel load shall be applied on an area of 4.5 inches by 4.5 inches.

f. The minimum concentrated load on stair treads shall be applied on an area of 2 inches by 2 inches. This load need not be assumed to act concurrently with the uniform load.

g. Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to

the increased loads caused by drift buildup or a greater snow design determined by the building official (see Section 1608).

h. See Section 1604.8.3 for decks attached to exterior walls.

i. Uninhabitable attics without storage are those where the maximum clear height between the joists and rafters is less than 42 inches, or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses. This live load need not be assumed to act concurrently with any other live load requirements.

j. Uninhabitable attics with storage are those where the maximum clear height between the joists and rafters is 42 inches or greater, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses.

The live load need only be applied to those portions of the joists or truss bottom chords where both of the following conditions are met:

i. The attic area is accessible from an opening not less than 20 inches in width by 30 inches in length that is located where the clear height in the attic is a minimum of 30 inches; and

ii. The slopes of the joists or truss bottom chords are no greater than two units vertical in 12 units horizontal.

The remaining portions of the joists or truss bottom chords shall be designed for a uniformly distributed concurrent live load of not less than 10 pounds per square foot.

k. Attic spaces served by stairways other than the pull-down type shall be designed to support the minimum live load specified for habitable attics and sleeping rooms.

l. Areas of occupiable roofs, other than roof gardens and assembly areas, shall be designed for appropriate loads as approved by the building official. Unoccupied landscaped areas of roofs shall be designed in accordance with Section 1607.12.3.

m. Live load reduction is not permitted unless specific exceptions of Section 1607.10 apply.

Reason: For historical context, the 2006 IBC and 2005 ASCE 7-05 contained similar language in that balconies and decks were treated as different uses and had different uniform loading criteria. Then the IBC diverged from matching ASCE 7 in 2006 under S9-06/07 when the IBC combined the separate occupancy categories balconies and decks into one item, with the uniform loading set as the "Same as occupancy served" force level. ASCE 7-10 followed suit in combining balconies and decks as a single item, however the uniform loading was set at 1.5 times the live load for the area served, with an upper bound not required to be greater than 100 psf. To harmonize the ASCE and IBC and IRC live loading requirements, this proposal is using the ASCE 7 load requirements for the baseline minimum live loads on balconies and decks.

Cost Impact: Will increase the cost of construction
For an ASCE 7 compliant design there is no increase in loading and thus no change in construction cost. For an IBC/IRC compliant design the loading of balconies and decks will increase possibly increasing the cost of structural framing for the support of these structures.

S86-16 : 1607.1-HUSTON13588

S87-16

IBC: 1607.1.

Proponent : Jonathan Siu, City of Seattle Department of Construction & Inspections, representing Washington Association of Building Officials Technical Code Development Committee (jon.siu@seattle.gov)

2015 International Building Code

Revise as follows:

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_0, AND MINIMUM CONCENTRATED LIVE LOADS[g]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
1. Apartments (see residential)	—	—
2. Access floor systems		
Office use	50	2000
Computer use	100	2000
3. Armories and drill rooms	150~~m~~n	—
4. Assembly areas		—
Fixed seats (fastened to floor)	60m	
Follow spot, projections and control rooms	50	
Lobbies	100m	
Movable seats	100m	
Stage floors	150~~m~~n	
Platforms (assembly)	100m	
Other assembly areas	100m	

5. Balconies and decks[h]	Same as occupancy served	—
6. Catwalks	40	300
7. Cornices	60	—
8. Corridors	100 Same as occupancy served except as indicated	—
First floor		
Other floors		
9. Dining rooms and restaurants	100[m]	—
10. Dwellings (see residential)	—	—
11. Elevator machine room and control room grating (on area of 2 inches by 2 inches)	—	300
12. Finish light floor plate construction (on area of 1 inch by 1 inch)	—	200
13. Fire escapes	100	—
On single-family dwellings only	40	
14. Garages (passenger vehicles only)	40~~m~~o	Note a
Trucks and buses	See Section 1607.7	
15. Handrails, guards and grab bars	See Section 1607.8	
16. Helipads	See Section 1607.6	
17. Hospitals		
Corridors above first floor	80	1,000
Operating rooms, laboratories	60	1,000
Patient rooms	40	1,000

18. Hotels (see residential)	—	—
19. Libraries		
Corridors above first floor	80	1,000
Reading rooms	60	1,000
Stack rooms	$150^{b,\ \cancel{m}\underline{n}}$	1,000
20. Manufacturing		
Heavy	$250^{\cancel{m}\underline{n}}$	3,000
Light	$125^{\cancel{m}\underline{n}}$	2,000
21. Marquees, except one-and two-family dwellings	75	—
22. Office buildings		
Corridors above first floor	80	2,000
File and computer rooms shall be designed for heavier loads based on anticipated occupancy	—	—
Lobbies and first-floor corridors	100	2,000
Offices	50	2,000

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
23. Penal institutions		—
Cell blocks	40	
Corridors	100	

24. Recreational uses:		—	
Bowling alleys, poolrooms and similar uses	75 m		
Dance halls and ballrooms	100 m		
Gymnasiums	100 m		
Ice skating rink	250 ~~m~~n		
Reviewing stands, grandstands and bleachers	100 c, m		
Roller skating rink	100 m		
Stadiums and arenas with fixed seats (fastened to floor)	60 c, m		
25. Residential		—	
One- and two-family dwellings			
Uninhabitable attics without storage i	10		
Uninhabitable attics with storage i, j, k	20		
Habitable attics and sleeping areas k	30		
Canopies, including marquees	20		
All other areas	40		

Hotels and multifamily dwellings					
Private rooms and corridors serving them	40				
Public rooms m and corridors serving them	100				
26. Roofs					
All roof surfaces subject to main- tenance workers		300			
Awnings and canopies:					
Fabric construction supported by a skeleton structure	5[m] ~~Nonreducible~~				
All other construction, except one-and two-family dwellings	20				
Ordinary flat, pitched, and curved roofs (that are not occupiable)	20				
Primary roof members exposed to a work floor					
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages		2,000	All other primary roof members		300

Occupiable roofs:		
Roof gardens	100	
Assembly areas	100 [m]	
All other similar areas	Note 1	Note 1
27. Schools		
Classrooms	40	1,000
Corridors above first floor	80	1,000
First-floor corridors	100	1,000
28. Scuttles, skylight ribs and accessible ceilings	—	200
29. Sidewalks, vehicular driveways and yards, subject to trucking	250 [d, ~~m~~n]	8,000 [e]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
30. Stairs and exits		
One- and two-family dwellings	40	300f
All other	100	300f
31. Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		—
Heavy	250~~m~~n	
Light	125~~m~~n	

32. Stores		
Retail		
First floor	100	1,000
Upper floors	75	1,000
Wholesale, all floors	125~~m~~n	1,000
33. Vehicle barriers	See Section 1607.8.3	
34. Walkways and elevated platforms (other than exitways)	60	—
35. Yards and terraces, pedestrians	100[m]	—

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm^2,

1 square foot = 0.0929 m^2,

1 pound per square foot = 0.0479 kN/m^2, 1 pound = 0.004448 kN,

1 pound per cubic foot = 16 kg/m^3.

a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of this Table or the following concentrated loads: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 pounds acting on an area of $4\frac{1}{2}$ inches by $4\frac{1}{2}$ inches; (2) for mechanical parking structures without slab or deck that are used for storing passenger vehicles only, 2,250 pounds per wheel.

b. The loading applies to stack room floors that support nonmobile, double-faced library book stacks, subject to the following limitations:

1. The nominal book stack unit height shall not exceed 90 inches;

2. The nominal shelf depth shall not exceed 12 inches for each face; and

3. Parallel rows of double-faced book stacks shall be separated by aisles not less than 36 inches wide.

c. Design in accordance with ICC 300.

d. Other uniform loads in accordance with an approved method containing provisions for truck loadings shall be considered where appropriate.

e. The concentrated wheel load shall be applied on an area of 4.5 inches by 4.5 inches.

f. The minimum concentrated load on stair treads shall be applied on an area of 2 inches by 2 inches. This load need not be assumed to act concurrently with the uniform load.

g. Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design determined by the building official (see Section 1608).

h. See Section 1604.8.3 for decks attached to exterior walls.

i. Uninhabitable attics without storage are those where the maximum clear height between the joists and rafters is less than 42 inches, or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses. This live load need not be assumed to act concurrently with any other live load requirements.

j. Uninhabitable attics with storage are those where the maximum clear height between the joists and rafters is 42 inches or greater, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses.

The live load need only be applied to those portions of the joists or truss bottom chords where both of the following conditions are met:

i. The attic area is accessible from an opening not less than 20 inches in width by 30 inches in length that is located where the clear height in the attic is a minimum of 30 inches; and

ii. The slopes of the joists or truss bottom chords are no greater than two units vertical in 12 units horizontal.

The remaining portions of the joists or truss bottom chords shall be designed for a uniformly distributed concurrent live load of not less than 10 pounds per square foot.

k. Attic spaces served by stairways other than the pull-down type shall be designed to support the minimum live load specified for habitable attics and sleeping rooms.

l. Areas of occupiable roofs, other than roof gardens and assembly areas, shall be designed for appropriate loads as approved by the building official. Unoccupied landscaped areas of roofs shall be designed in accordance with Section 1607.12.3.

~~m. Live load reduction is not permitted unless specific exceptions of Section 1607.10 apply.~~

m. Live load reduction is not permitted.
n. Live load reduction is only permitted in accordance with Section 1607.10.1.2 or Item 1 of Section 1607.10.2.
o. Live load reduction is only permitted in accordance with Section 1607.10.1.3 or Item 2 of Section 1607.10.2.

Reason: This proposal clarifies which live loads are not permitted to be reduced, and more clearly align the IBC with ASCE 7.
The current footnote m in Table 1607.1 restricts the use of the live load reduction equations in Sections 1607.10.1 (basic) and 1607.10.2 (alternate), "unless specific exceptions of Section 1607.10 apply." This clause causes confusion for both engineers and building officials:

- Section 1607.10.1 states, "Except for uniform live loads at roofs, **all other** minimum uniformly distributed live loads...are permitted to be reduced.... [emphasis mine]"
- Item 3 of Section 1607.10.2 reads, "**For live loads not exceeding 100 psf**...the design live load for any structural member supporting 150 square feet...or more is permitted to be reduced.... [emphasis mine]"

Does the text in either section constitute a "specific exception" that permits all the live loads with footnote m to be reduced? We have had discussions with building officials and engineers in our association who have maintained that it does. However, in comparing Table 1607.10 with ASCE 7-10 Table 4-1, along with the corresponding texts in IBC Section 1607.10 and ASCE 7 Section 4.7, we believe this to be in error for the following reasons:

- If one believes the live load reduction equations can be used in all instances where footnote m appears, there is no longer any restriction on live load reductions, and footnote m is meaningless.
- Because its text is structured differently from the IBC, it is clear in ASCE 7-10 that the live load reduction for heavy live loads (> 100 psf) and passenger vehicle garages are allowed to be reduced only for members supporting two or more floors. Assembly loads are clearly not allowed to be reduced in ASCE 7-10. (See ASCE 7-10, Sections 4.7.3, 4.7.4, and 4.7.5.)
- Footnote m was introduced into the 2012 IBC via code change proposal S60-09/10, and the proponent's clear intent was to align the 2012 IBC with ASCE 7-10. Allowing an expansion of the application of live load reduction would cause a misalignment between the two documents.

In order to clear up the confusion, this proposal introduces two new footnotes to the table, and modifies footnote m.

In reading through Section 1607.10.1 regarding the basic live load reduction, the only "specific exceptions" appear to be those in 1607.10.1.2 (heavy live loads) and 1607.10.1.3 (passenger vehicle garages). Items 1 and 2 of the alternate live load reduction (1607.10.2) cover the same territory. These clearly correspond to Sections 4.7.3 and 4.7.4 in ASCE 7-10. It is therefore clear that there are no exceptions for the rest of the items to which footnote m is applied, including all assembly spaces (corresponding to Section 4.7.5 in ASCE 7-10).

The new footnote n takes care of heavy live loads--it starts by prohibiting live load reductions, but refers the user to the specifc sections that allow the reduction. New footnote o does the same for passenger vehicle garage loading. Footnote m is then modified so as to completely prohibit any live load reduction for the remaining items covered by the old footnote. This incidentally allows footnote m to be substituted for the "nonreducible" note in the line in the table for fabric awnings/canopies in Item 26 of Table 1607.1.

We believe this makes restrictions on the application of live load reductions much clearer for the users of the IBC.

Cost Impact: Will not increase the cost of construction
This is a clarification that does not result in an increase of design loads or cost of construction.

S87-16 : TABLE 1607.1-SIU11258

S88-16

IBC: 1607.4, 1607.9.3.

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 *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

S89-16

IBC: 1607.8, 1607.8.3.

Proponent : Edward Kulik, representing Building Code Action Committee (bcac@iccsafe.org)

2015 International Building Code

Revise as follows:

1607.8 Loads on handrails, guards, grab bars, <u>and</u> seats ~~and vehicle barriers~~. Handrails~~,~~ ~~*guards*, grab bars, accessible seats, accessible benches~~ and ~~vehicle barriers~~ <u>guards,</u> shall be designed and constructed for the structural loading conditions set forth in ~~this section~~ <u>Section 1607.8.1. Grab bars, shower seats, and accessible benches shall be designed and constructed for structural loading conditions set forth in Section 1607.8.2.</u>

~~1607.8.3~~ <u>1607.9</u> Vehicle barriers. *No change to text.*

Reason: This proposal is intended to be a clarification. The current language only brings up benches in dressing rooms. 2009 A117.1 require accessible benches in dressing rooms, locker rooms and steam rooms and saunas. The loads of 250 lbs. should be applied to grab bars and show er seats w herever they are provided. The load of 250 pounds should not be required for all benches in any dressing room, but should be required for accessible benches in all three locations.
The suggested language in 1607.8.2 is because the rooms are scoped in Chapter 11, but the benches themselves are specified in ICC A117.1. The need to be so specific is that if the requirement w as just for bench seats, it could be misinterpreted to be applicable to any bench seating, accessible or not, fixed or loose. The current language follow s the grouping of ASCE 7 w hich also includes fixed ladders.

Existing load requirements for vehicle barriers have been moved to be a separate section for clarity. These loads are related to impact on these barriers from vehicles as opposed to loads from an individual grasping a handrail or using a bench.

This proposal is submitted by the ICC Building Code Action Committee (BCAC). BCAC w as 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 w ere numerous Working Group meetings and conference calls for the current code development cycle, w hich included members of the committee as w ell as any interested party to discuss and debate the proposed changes. Related documentation and reports are posted on the BCAC w ebsite at: **BCAC**

Cost Impact: Will not increase the cost of construction
Need cost impact and substatiation.
As this is intended as a clarification, there w ill be no increase in construction cost.

S89-16 : 1607.8-KULIK10974

S90-16
Part I:
IBC: 1607.8.1.
Part II:
IRC: R301.5.

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC-STRUCTURAL CODE COMMITTEE. PART II WILL BE HEARD BY THE IRC-BUILDING CODE COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

Proponent : Charles Bajnai, representing Deck Code Coalition and Chesterfield County, VA; and North American Deck and Railing Association (NADRA) (bajnaic@chesterfield.gov)

Part I

2015 International Building Code

Revise as follows:

1607.8.1 Handrails and guards. Handrails and *guards* shall be designed to resist a linear load of 50 pounds per linear foot (plf) (0.73 kN/m) in accordance with Section 4.5.1 of ASCE 7. Glass handrail assemblies and *guards* shall also comply with Section 2407.

Exceptions:

1. For one- and two-family dwellings, ~~only~~ handrails shall be designed to resist the~~ single concentrated~~ load required by Section 1607.8.1.1~~ shall be applied~~.
2. Guards shall be designed to resist 200 pounds acting as a single concentrated load, applied perpendicular at any point along the top of the guard, in the vertically downward direction and the horizontally outward direction and 50 pounds acting as a single concentrated load, applied perpendicular at any point along the top of the guard, in the horizontally inward direction and the vertically upward direction. These loads shall be applied independent of one another, and are assumed not to occur with any other live load.
3. In Group I-3, F, H and S occupancies, for areas that are not accessible to the general public and that have an *occupant load* less than 50, the minimum load shall be 20 pounds per foot (0.29 kN/m).

Part II

2015 International Residential Code

Revise as follows:

TABLE R301.5
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS (in pounds per square foot)

USE	LIVE LOAD
Uninhabitable attics without storage[b]	10

Uninhabitable attics with limited storage[b,g]	20
Habitable attics and attics served with fixed stairs	30
Balconies (exterior) and decks[e]	40
Fire escapes	40
~~Guards and h~~Handrails[d]	200[h]
Guards[h, i]	200 outward 200 downward 50 inward 50 upward
Guard in-fill components[f]	50[h]
Passenger vehicle garages[a]	50[a]
Rooms other than sleeping rooms	40
Sleeping rooms	30
Stairs	40[c]

For SI: 1 pound per square foot = 0.0479 kPa, 1 square inch = 645 mm ^{2} ,1 pound = 4.45 N.

a. Elevated garage floors shall be capable of supporting a 2,000-pound load applied over a 20-square-inch area.

b. Uninhabitable *attics* without storage are those where the clear height between joists and rafters is not more than 42 inches, or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses. This live load need not be assumed to act concurrently with any other live load requirements.

c. Individual stair treads shall be designed for the uniformly distributed live load or a 300-pound concentrated load acting over an area of 4 square inches, whichever produces the greater stresses.

d. A single concentrated load applied in any direction at any point along the top.

e. See Section R507.1 for decks attached to *exterior walls* .

f. *Guard* in-fill components (all those except the handrail), balusters and panel fillers shall be designed to withstand a horizontally applied normal load of 50 pounds on an area equal to 1 square foot. This load need not be assumed to act concurrently with any other live load requirement.

g. Uninhabitable *attics* with limited storage are those where the clear height between joists and rafters is not greater than 42 inches, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses. The live load need only be applied to those portions of the joists or truss bottom chords where all of the following conditions are met:

1. The *attic* area is accessible from an opening not less than 20 inches in width by 30 inches in length that is located where the

clear height in the *attic* is not less than 30 inches.

2. The slopes of the joists or truss bottom chords are not greater than 2 inches vertical to 12 units horizontal.

3. Required insulation depth is less than the joist or truss bottom chord member depth.

The remaining portions of the joists or truss bottom chords shall be designed for a uniformly distributed concurrent live load of not less than 10 pounds per square foot.

h. Glazing used in handrail assemblies and *guards* shall be designed with a safety factor of 4. The safety factor shall be applied to each of the concentrated loads applied to the top of the rail, and to the load on the in-fill components. These listed loads shall be determined independent of one another, and loads are assumed not to occur with any other live load.

i. Single concentrated load applied perpendicular at any point along the top of the guard. The listed loads shall be applied independent of one another, and assumed not to occur with any other live load.

Reason:

Part I:
WHAT: For guards, this code change amends the IBC Live Load Table from requiring a 200# single concentrated load applied in any direction to a 200# single concentrated load applied in the outward direction, and in the downward direction, and a 50# single concentrated load applied in the upward direction, and the inward direction.

Note: This proposal does not revise load requirements for *handrails*. Handrails, especially on stairs, may be subject to inward and / or upward loads.

WHY: The consensus of the Deck Code Coalition (DCC) is that the current live load requirement in the IBC for guards is too conservative and irrational when considering potential fall-preventing forces in the inward and upward directions.

HISTORY: This proposal is significant because these load requirements have been in the model codes for a long time:

1975 BOCA, Section 709.4 Railings: Railings around stairwells, balconies and other floor openings, both exterior and interior, shall be designed to resist a load of at least two hundred (200) pounds applied in any direction at any point of the top rail and also a vertical and a horizontal thrust of fifty (50) pounds per lineal foot applied at the top railing. The concentrated load and distributed loads need not be assumed to act concurrently..."

1993 BOCA, Section 1615.8.1: "Guards shall be designed and constructed for a concentrated load of 200 pounds applied at any point and in any direction along the top railing member."

2015 IBC, Table 1607.1 #15: Handrails, guards and grab bars, refers the reader to Section 1607.8.1: Handrails and guards and its exception #1 for residential applications.

2015 IBC, Section 1607.8.1, exception 1: For one- and two-family dwellings, only the single concentrated load required by Section 1608.1.1 shall be applied.

2015 IBC, Section 1607.8.1.1. Concentrated load. Handrails and guards shall be designed to resist a concentrated load of 200 pounds in accordance with Section 4.5.1 of ASCE 7.

TESTING STANDARDS: When exploring the **intent** of the IBC, current testing practices for guard systems which comply with the IBC, and which are accepted by designers, manufacturers, ICC, and code officials should be considered, and include:

1. **ICC ES AC273** "Acceptance Criteria for Handrails and Guards",
2. **ICC ES AC174** "Acceptance Criteria for Deck Board Span Ratings and Guardrail Systems (Guards and Handrails)",
3. **ASTM D7032** "Standard Specification for Establishing Performance Ratings for Wood-Plastic Composite Deck Boards and Guardrail Systems (Guards or Handrails).

AC273 and AC174 provide acceptance criteria that require guard *systems* to be tested for outward loads with a 2.5X factor of safety (200 lbs x 2.5 safety factor equals testing at 500 lbs force applied at the top of the guard in the *weakest locations*). These ICC ES acceptance criteria do not require test forces to be applied in the inward or upward direction.

ASTM D7032 is the standard referenced in the IBC and which exterior plastic composite guards are required to comply. The guard testing requirements are consistent with the acceptance criteria in AC273 and AC174.

SAFETY: Clearly, the broadly accepted intent of the IBC (as indicated by the acceptance criteria) is that guards are to be designed to withstand outward and downward forces where the biggest injuries can be expected.

CONCLUSION: The conclusion in the mind of the committee is that Section 1607.8.1 is too conservative in terms of load direction. The acceptance criteria and testing standards have been developed by experts and reflect the perceived intent of the IBC. Approval of this proposal will:

1) Align the requirements of the IBC with the established test procedures of ASTM D7032, AC273, and AC174, and

2) Provide explicit minimum load requirements for guards regarding outward, downward, inward, and upward forces.

The Deck Code Coalition (DCC) is a diverse group of stakeholders, including building officials, industry associations, product manufacturers, design professionals, and academia who have worked since the 2012 IBC code development cycle in an effort to consolidate and improve deck construction methods from across the country.

Our goals are threefold:

1. Consolidate existing code scattered throughout the IRC under the newly expanded Section R507. Being able to easily locate all deck related code provisions in one section equally serves the builder, code official and design professional to a safer, code-conforming deck.
2. Create realistic, fact-based, prescriptive solutions to fill critical gaps in the current deck code. Many parts of existing deck code rely on subjective interpretations by the reader leading to an inconsistent approach to meeting minimum code.
3. Maintain and promote a safer deck structure without unduly burdening the builder. In all cases the DCC want to offer safe minimum requirements without stifling the creativity of the design professional or builder.

Figure 6

Part II:

WHAT: For guards, this code change amends the IRC Live Load Table from requiring a 200# single concentrated load applied in any direction to a 200# single concentrated load applied in the outward direction, and in the downward direction, and a 50# single concentrated load applied in the upward direction, and the inward direction.

Note: This proposal does not revise load requirements for *handrails*. Handrails, especially on stairs, may be subject to inward and / or upward loads.

WHY: The consensus of the Deck Code Coalition (DCC) is that the current live load requirement in the IRC for guards is too conservative and irrational when considering potential fall-preventing forces in the inward and upward directions.

HISTORY: This proposal is significant because these load requirements have been in the model codes for a long time:

> **1975 BOCA, Section 709.4 Railings**: Railings around stairwells, balconies and other floor openings, both exterior and interior, shall be designed to resist a load of at least two hundred (200) pounds applied in any direction at any point of the top rail and also a vertical and a horizontal thrust of fifty (50) pounds per lineal foot applied at the top railing. The concentrated load and distributed loads need not be assumed to act concurrently..."

1993 BOCA, Section 1615.8.1: "Guards shall be designed and constructed for a concentrated load of 200 pounds applied at any point and in any direction along the top railing member."

2015 IBC, Table 1607.1 #15: Handrails, guards and grab bars, refers the reader to Section 1607.8.1: Handrails and guards and its exception #1 for residential applications.

2015 IBC, Section 1607.8.1, exception 1: For one- and two-family dwellings, only the single concentrated load required by Section 1608.1.1 shall be applied.

2015 IBC, Section 1607.8.1.1. Concentrated load. Handrails and guards shall be designed to resist a concentrated load of 200 pounds in accordance with Section 4.5.1 of ASCE 7.

TESTING PROCEDURES: When exploring the **intent** of the IRC, current testing practices for guard systems which comply with the IRC, and which are accepted by designers, manufacturers, ICC, and code officials should be considered, and include:

1. **ICC ES AC273** "Acceptance Criteria for Handrails and Guards",
2. **ICC ES AC174** "Acceptance Criteria for Deck Board Span Ratings and Guardrail Systems (Guards and Handrails)",
3. **ASTM D7032** "Standard Specification for Establishing Performance Ratings for Wood-Plastic Composite Deck Boards and Guardrail Systems (Guards or Handrails).

AC273 and AC174 provide acceptance criteria that require guard *systems* to be tested for outward loads with a 2.5X factor of safety (200 lbs x 2.5 safety factor equals testing at 500 lbs force applied at the top of the guard in the *weakest locations*). These ICC ES acceptance criteria do not require test forces to be applied in the inward or upward direction.

ASTM D7032 is the standard referenced in the IRC and which exterior plastic composite guards are required to comply. The guard testing requirements are consistent with the acceptance criteria in AC273 and AC174.

Figure 6

SAFETY: Clearly, the broadly accepted intent of the IRC (as indicated by the acceptance criteria) is that guards are to be designed to withstand outward and downward forces where the biggest injuries can be expected.

CONCLUSION: The conclusion in the mind of the committee is that R301.5 is too conservative in terms of load direction. The acceptance criteria and testing standards have been developed by experts and reflect the perceived intent of the IRC. Approval of this proposal will:

1) Align the requirements of the IRC with the established test procedures of ASTM D7032, AC273, and AC174, and

2) Provide explicit minimum load requirements for guards regarding outward, downward, inward, and upward forces.

The Deck Code Coalition (DCC) is a diverse group of stakeholders, including building officials, industry associations, product manufacturers, design professionals, and academia who have worked since the 2012 IRC code development cycle in an effort to consolidate and improve deck construction methods from across the country.

Our goals are threefold:

1. Consolidate existing code scattered throughout the IRC under the newly expanded Section R507. Being able to easily locate all deck related code provisions in one section equally serves the builder, code official and design professional to a safer, code-conforming deck.
2. Create realistic, fact-based, prescriptive solutions to fill critical gaps in the current deck code. Many parts of existing deck code rely on subjective interpretations by the reader leading to an inconsistent approach to meeting minimum code.
3. Maintain and promote a safer deck structure without unduly burdening the builder. In all cases the DCC want

to offer safe minimum requirements without stifling the creativity of the design professional or builder.

Cost Impact: Will increase the cost of construction

Cost may increase for guard systems that are tested as ICC-ES and ASTM would need to update their acceptance criteria and standards.

S90-16 : 1607.8.1-BAJNAI10536

S91-16

IBC: 1607.8.3.1 (New).

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.

S91-16 : 1607.8.3.1 (NEW)-RUBENACKER13473

S92-16

IBC: 1607.10.1.4 (New), 1607.10.1.5 (New), 1607.9.5 (New).

Proponent : Karl Rubenacker, Gilsanz Murray Steficek, representing Codes & Standards Committee, Structural Engineer's Association of New York (karl.rubenacker@gmsllp.com)

2015 International Building Code

Add new text as follows:

1607.9.5 Assembly structures. Seating areas in grandstands, stadiums, and similar assembly structures shall be designed to resist the simultaneous application of a horizontal swaying load of not less than 24 pounds per foot (36 kg/m) of seats applied in a direction parallel to the row of the seats, and of not less than 10 pounds per foot (15 kg/m) of seats in a direction perpendicular to the row of the seats. When this load is used in combination with wind for outdoor structures, the wind load shall be one-half of the design wind load.

1607.10.1.4 Special occupancies. Live loads of 100 psf (4.79 kN/m2) or less at areas where fixed seats are located shall not be reduced in public assembly occupancies or in areas used for retail or wholesale sales.

1607.10.1.5 Special structural elements. Live loads shall not be reduced for one-way slabs except as permitted in Section 1609.9.1.1. Live loads shall not be reduced for calculating shear stresses at the head of columns in flat slab or flat plate construction.

Reason: These design conditions are helpful to engineers designing affected structures.

Cost Impact: Will not increase the cost of construction
Will not increase cost of construction, these are just additional design checks.

S92-16 : 1607.9.5 (NEW)-RUBENACKER13474

S93-16

IBC: 1607.12.3.1.

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

S94-16

IBC: 1607.1, 1607.12.4.

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

2015 International Building Code

Revise as follows:

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_0, AND MINIMUM CONCENTRATED LIVE LOADS[g]

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
1. Apartments (see residential)	—	—
2. Access floor systems		
Office use	50	2000
Computer use	100	2000
3. Armories and drill rooms	150[m]	—
4. Assembly areas		—
Fixed seats (fastened to floor)	60m	
Follow spot, projections and control rooms	50	
Lobbies	100m	
Movable seats	100m	
Stage floors	150m	
Platforms (assembly)	100m	
Other assembly areas	100m	
5. Balconies and decks[h]	Same as occupancy served	—

6. Catwalks	40	300
7. Cornices	60	—
8. Corridors	100 Same as occupancy served except as indicated	—
First floor		
Other floors		
9. Dining rooms and restaurants	100[m]	—
10. Dwellings (see residential)	—	—
11. Elevator machine room and control room grating (on area of 2 inches by 2 inches)	—	300
12. Finish light floor plate construction (on area of 1 inch by 1 inch)	—	200
13. Fire escapes	100	—
On single-family dwellings only	40	
14. Garages (passenger vehicles only)	40m	Note a
Trucks and buses	See Section 1607.7	
15. Handrails, guards and grab bars	See Section 1607.8	
16. Helipads	See Section 1607.6	
17. Hospitals		
Corridors above first floor	80	1,000
Operating rooms, laboratories	60	1,000
Patient rooms	40	1,000
18. Hotels (see residential)	—	—

19. Libraries		
Corridors above first floor	80	1,000
Reading rooms	60	1,000
Stack rooms	150[b, m]	1,000
20. Manufacturing		
Heavy	250[m]	3,000
Light	125[m]	2,000
21. Marquees, except one-and two-family dwellings	75	—
22. Office buildings		
Corridors above first floor	80	2,000
File and computer rooms shall be designed for heavier loads based on anticipated occupancy	—	—
Lobbies and first-floor corridors	100	2,000
Offices	50	2,000

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
23. Penal institutions		—
Cell blocks	40	
Corridors	100	
24. Recreational uses:		—
Bowling alleys, poolrooms	75 m	

and similar uses		
Dance halls and ballrooms	100 m	
Gymnasiums	100 m	
Ice skating rink	250 m	
Reviewing stands, grandstands and bleachers	100 c, m	
Roller skating rink	100 m	
Stadiums and arenas with fixed seats (fastened to floor)	60 c, m	
25. Residential		—
One- and two-family dwellings		
Uninhabitable attics without storage i	10	
Uninhabitable attics with storage i, j, k	20	
Habitable attics and sleeping areas k	30	
Canopies, including marquees	20	
All other areas	40	
Hotels and multifamily dwellings		

Priv ate rooms and corridors serv ing them	40				
Public rooms m and corridors serv ing them	100				
26. Roof s					
All roof surf aces subject to main- tenance workers		300			
Awnings, <u>sun control and shading dev ices,</u> and canopies:					
Fabric construction supported by a skeleton structure	5 Nonreducible				
All other construction, except one-and two-f amily dwellings	20				
Ordinary f lat, pitched, and curv ed roof s (that are not occupiable)	20				
Primary roof members exposed to a work f loor					
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roof s ov er manuf acturing, storage warehouses, and repair garages		2,000	All other primary roof members		300

Occupiable roofs:		
Roof gardens	100	
Assembly areas	100 m	
All other similar areas	Note 1	Note 1
27. Schools		
Classrooms	40	1,000
Corridors above first floor	80	1,000
First-floor corridors	100	1,000
28. Scuttles, skylight ribs and accessible ceilings	—	200
29. Sidewalks, vehicular driveways and yards, subject to trucking	250 d, m	8,000 e

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
30. Stairs and exits		
One- and two-family dwellings	40	300f
All other	100	300f
31. Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		—
Heavy	250m	
Light	125m	
32. Stores		

Retail		
First floor	100	1,000
Upper floors	75	1,000
Wholesale, all floors	125m	1,000
33. Vehicle barriers	See Section 1607.8.3	
34. Walkways and elevated platforms (other than exitways)	60	—
35. Yards and terraces, pedestrians	100[m]	—

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm ^{2} ,

1 square foot = 0.0929 m ^{2} ,

1 pound per square foot = 0.0479 kN/m ^{2} , 1 pound = 0.004448 kN,

1 pound per cubic foot = 16 kg/m ^{3} .

a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of this Table or the following concentrated loads: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 pounds acting on an area of 4 $^1/_2$ inches by 4 $^1/_2$ inches; (2) for mechanical parking structures without slab or deck that are used for storing passenger vehicles only, 2,250 pounds per wheel.

b. The loading applies to stack room floors that support nonmobile, double-faced library book stacks, subject to the following limitations:

1. The nominal book stack unit height shall not exceed 90 inches;

2. The nominal shelf depth shall not exceed 12 inches for each face; and

3. Parallel rows of double-faced book stacks shall be separated by aisles not less than 36 inches wide.

c. Design in accordance with ICC 300.

d. Other uniform loads in accordance with an approved method containing provisions for truck loadings shall be considered where appropriate.

e. The concentrated wheel load shall be applied on an area of 4.5 inches by 4.5 inches.

f. The minimum concentrated load on stair treads shall be applied on an area of 2 inches by 2 inches. This load need not be assumed to act concurrently with the uniform load.

g. Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design determined by the building official (see Section 1608).

h. See Section 1604.8.3 for decks attached to exterior walls.

i. Uninhabitable attics without storage are those where the maximum clear height between the joists and rafters is less than 42 inches, or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses. This live load need not be assumed to act concurrently with any other live load requirements.

j. Uninhabitable attics with storage are those where the maximum clear height between the joists and rafters is 42 inches or greater, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses.

The live load need only be applied to those portions of the joists or truss bottom chords where both of the following conditions are

met:

i. The attic area is accessible from an opening not less than 20 inches in width by 30 inches in length that is located where the clear height in the attic is a minimum of 30 inches; and

ii. The slopes of the joists or truss bottom chords are no greater than two units vertical in 12 units horizontal.

The remaining portions of the joists or truss bottom chords shall be designed for a uniformly distributed concurrent live load of not less than 10 pounds per square foot.

k. Attic spaces served by stairways other than the pull-down type shall be designed to support the minimum live load specified for habitable attics and sleeping rooms.

l. Areas of occupiable roofs, other than roof gardens and assembly areas, shall be designed for appropriate loads as approved by the building official. Unoccupied landscaped areas of roofs shall be designed in accordance with Section 1607.12.3.

m. Live load reduction is not permitted unless specific exceptions of Section 1607.10 apply.

1607.12.4 Awnings and canopies. Awnings , sun control and shading devices, and canopies shall be designed for uniform live loads as required in Table 1607.1 as well as for snow loads and wind loads as specified in Sections 1608 and 1609.

Reason: With demands for increased energy efficiency, the use of sun controls and shading devices are increasing. The devices are attached to the exterior of the building. The code provides no guidance for the loads to be used in designing the attachment to the building and to the devices themselves. This code proposal adds sun control and shading devices to the requirements for awnings and canopies. There have been many failures discovered in the field due to snow and wind loads, as well as attachment failures.

Cost Impact: Will not increase the cost of construction
There is no increase in cost to design with loads to prevent failures.

S94-16 : 1607.12.4-EDWARDS13390

S95-16

IBC: 1607.12.5.1.

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 betw een 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 w ith 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 w ill not change the cost of construction because there is no change to technical requirements. These are editorial changes only.

S95-16 : 1607.12.5.1-CAIN13370

S96-16

IBC: 1607.12.5.1.

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 to be covered by solar photovoltaic panels or modules shall be designed for the roof live load, L_r, assuming that the photovoltaic panels or modules are not present. The roof photovoltaic live load in 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 where the clear space between the panels and the rooftop is not more than ~~24~~ 42 inches (~~610~~ 1067 mm) shall be considered inaccessible. Roof surfaces not covered by photovoltaic panels shall be designed for the roof live load.

Reason: This proposal seeks to revise the threshold for offset of live load from 24 inches to 42 inches. The Structural Engineers Association of California (SEAOC) PV Systems committee has conducted lengthy conversations on this topic, and is in agreement that 42 inches is a more reasonable threshold than 24 inches. The following excerpt is from *Recommended Design Live Loads for Rooftop Solar Arrays,* by Colin Blaney, S.E. of ZFA Structural Engineers and Ron LaPlante, S.E. of the California Division of State Architect, Structural Safety (DSA SS).

> The other significant question was whether or not live loads should be included under raised low to medium profile PV panel systems to account for either maintenance worker loads or other special loads. This would include loads such as storage, including temporary stacking of re-roofing materials. This question was discussed at length along with the term "inaccessible" that is used in DSA IR 16-8 and 2015 IBC Section 1607.12.5.1. The discussion centered around whether it is meant to be used to identify the local space below a PV system or the entire roof top area that may be only "accessible" through a locked stair tower or hatch. After lengthy deliberations, the Task Group agreed to set a clear height cut off of 42". Under arrays taller than the cut off, uniform and concentrated live loads must be considered on the covered roof areas where the below-panel dimension exceeds this limit in addition to the uncovered areas. The 42" cut off is similar to uninhabitable attic spaces where storage loads need not be considered per 2012 IBC Table 1607.1 footnotes i & j. The Task Group also felt that it would be highly unlikely that any building owners or maintenance workers would store items of any significance below such systems as they would be exposed to the weather and could easily block roof drainage. It was also agreed that the term "inaccessible" should not apply to general roof access when discussed in context of eliminating roof live loads from consideration.

Footnotes i and j of 2015 IBC Table 1607.1 read as follows:

> i. Uninhabitable attics without storage are those where the maximum clear height between the joists and rafters is less than 42 inches, or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses. This live load need not be assumed to act concurrently with any other live load requirements.
>
> j. Uninhabitable attics with storage are those where the maximum clear height between the joists and rafters is 42 inches or greater, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses.

This proposal strives to establish a reasonable threshold for live load. Roof live load is defined as: "A load on a roof produced: 1) During maintenance by workers, equipment and materials; 2) During the life of the structure by movable objects such as planters or other small decorative appurtenances that are not occupancy related; 3) By the use and occupancy of the roof such as for roof gardens or assembly areas." In this definition, item 1 is most relevant for the space beneath photovoltaic panel systems. It is unlikely a person will be beneath a photovoltaic panel system for maintenance if the clear vertical height is 24 inches or less. This would require a maintenance worker to slide under the panels on back or belly, and would not allow working space. It is more reasonable to assume a person could be beneath the system for maintenance if the clear vertical height is 42 inches or greater.

Bibliography: Recommended Design Live Loads for Rooftop Solar Arrays, Colin Blaney S.E. and Ronald W. LaPlante S.E., SEAOC Conference Proceedings, 2013.

Cost Impact: Will not increase the cost of construction
This proposal w ill not increase the cost of construction, as it relaxes the live load threshold to be consistent w ith attic requirements.

S96-16 : 1607.12.5.1-CAIN13391

S97-16

IBC: 1607.12.5.1.

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

2015 International Building Code

Delete and substitute as follows:

1607.12.5.1 Roof live load. ~~Roof surfaces to be covered by solar photovoltaic panels or modules shall be designed for the roof live load, L_r, assuming that the photovoltaic panels or modules are not present. The roof photovoltaic live load in 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 where the clear space between the panels and the rooftop is not more than 24 inches (610 mm) shall be considered inaccessible. Roof surfaces not covered by photovoltaic panels shall be designed for the roof live load.~~
Roof surfaces to be covered by solar photovoltaic panels or modules shall be designed for the roof live load, L_r, assuming that the photovoltaic panels or modules are not present. The roof photovoltaic live load in 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 where the clear space between the panels and the rooftop is not more than 42 inches (1067 mm) shall be considered inaccessible. Roof surfaces not covered by photovoltaic panels shall be designed for the roof live load.

Reason: When section 1607.12.5 w as added to the IBC last cycle there w as a recognized need for the inclusion of photovoltaic into the loading requirements. The adopted language of 1607.12.5 w as a joint effort betw een the representatives of the solar industry, Structural Engineers Association of California (SEAOC), National Council of Structural Engineers Associations (NCSEA) w ith input from the National Association of Home Builders (NAHB). The final language w as a compromise betw een the various interested parties w ith the recognition that the language w as a first step in incorporating structural provisions for photovoltaic panel systems into Chapter 16.
The 24 inch height in section 1607.12.5.1 w as selected as an arbitrary distance betw een the panels and roof surface for the purposes of considering the area inaccessible for the application of roof live loading. After further consideration of this dimension this proposal is to simply increase the height to 42 inches. The selection of 42 inches w ill be for consistency w ith another similar situation in w hich there is a defining limit of w here storage needs to be considered. For comparison w ith IBC Table 1607.1 item 25 footnote i attics w ithout storage (Table 1607.1 footnote i), a proposed increase to 42 inches.

Cost Impact: Will not increase the cost of construction
Increasing the allow able height from 24 inches to 42 inches w ill decrease the number of projects w here live loading on top of the solar systems w ill need to be accounted. This may slightly decrease the cost of construction, how ever it is likely to have a negligible impact on cost.

S97-16 : 1607.12.5.1-HUSTON13268

S98-16

IBC: 1607.12.5.2.1 (New).

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/m^2).

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 1607.12.5.2.1 Photovoltaic panels installed on open grid roof structures. Awareness of building sustainability in recent years has led to unprecedented growth rate in design and implementation of rooftop solar photovoltaic (PV) systems. Engineers have made various assumptions relating to the vertical support of rooftop solar PV systems particularly at existing structures where reserve vertical capacity is often limited. Among the many assumptions made is whether full or partial roof live loads must be used or whether no live loads need to be considered at areas covered by new PV systems. This issue has been studied by a Live Load Task Group under the directive of the Structural Engineers Association of California (SEAOC) Solar PV System Subcommittee. Recommendation leading to a proposal that has been finalized and accepted into ASCE 7.In the ASCE 7-16 Main Committee ballot, new provisions on Rooftop Solar Arrays based primarily on SEAOC PV-2-2012 document received overwhelming approval action. This proposal on live load requirements will complement the new provision for solar PV system wind load design. Together the live load and wind provisions will help solidify ASCE 7 as the single source design loading information for both the design professionals as well as the industry related to solar photovoltaic system.

Reference:

Blaney, Colin and LaPlante, Ron (2013), "Recommended Design Live Loads for Rooftop Solar Arrays," SEAOC Convention Proceedings, 2013, pp. 264 – 278.

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).

S98-16 : 1607.12.5.2.1 (NEW)-GOUPIL12940

S99-16

IBC: 1607.12.5.3.

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

2015 International Building Code

Delete and substitute as follows:

1607.12.5.3 ~~Photovoltaic panels or modules installed as an independent structure~~ Freestanding photovoltaic panel systems. ~~Solar photovoltaic panels or modules that are independent structures and do not have accessible/occupied space underneath are not required to accommodate a roof photovoltaic live load, provided the area under the structure is restricted to keep the public away. All other loads and combinations in accordance with Section 1605 shall be accommodated.~~

~~Solar photovoltaic panels or modules that are designed to be the roof, span to structural supports and have accessible/occupied space underneath shall have the panels or modules and all supporting structures designed to support a roof photovoltaic live load, as defined in Section 1607.12.5.1 in combination with other applicable loads. Solar photovoltaic panels or modules in this application are not permitted to be classified as "not accessible" in accordance with Section 1607.12.5.1.~~

Design loads for freestanding, ground mounted *photovoltaic panel systems* with no use underneath need not include live load. all other loads and load combinations in accordance with Section 1605 shall be considered.

Photovoltaic panel systems mounted on raised support structures with open grid framing and no deck, and with occupied space underneath, shall have the supporting structure designed to support a reducible live load, in combination with other applicable loads.

Reason: Section 1607.12 w as created w ith proposal S72-12 for the 2015 IBC. This proposal is intended to clarify the language of Section 1607.12.5.3, by using defined terms and by simplifying the language. The first paragraph is for ground mounted photovoltaic panel systems w ith no use underneath. For example, fixed-tilt ground mounts or single axis trackers. These systems have no live load, and the language is clarified to indicate live load is not required. The second paragraph is revised to clarify that it applies to raised support structures such as parking lot canopy structures or solar trellises. Live load can be reduced on these structures w hen they have open grid framing and no rood deck.

Cost Impact: Will not increase the cost of construction
This proposal w ill not increase the cost of construction, as it does not change any technical requirements.

S99-16 : 1607.12.5.3-CAIN13508

S100-16

IBC: 706.2, 1607.14, 1607.14.2 (New).

Proponent : Paul Coats, PE CBO, representing American Wood Council (pcoats@awc.org)

2015 International Building Code

Revise 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 shall be deemed to comply with this section.

1607.14 Interior walls and partitions. Interior walls and partitions that exceed 6 feet (1829 mm) in height, including their finish materials, shall have adequate strength and stiffness to resist the loads to which they are subjected but not less than ~~a horizontal~~ an out-of-plane lateral load of 5 psf (0.240 kN/m^2) for Allowable Stress Design or 8 psf (0.384 kN/m^2) for Strength Design, applied from either direction.

Add new text as follows:

1607.14.2 Fire walls. Fire walls and their supports shall be designed and constructed to resist, under fire conditions, the required gravity loads and the out-of-plane lateral loads in 1607.14. The required gravity loads and the out-of-plane lateral loads need not be taken as concurrent loads.

Reason: This change addresses 3 issues. First, the current 5 psf lateral load required for all interior walls and partitions has historically been applied to allowable stress design (ASD) and the change provides an equivalent 8 psf lateral load for strength design (LRFD). Second, this proposal clarifies that fire walls in 706.2 must also comply with the lateral load required for all interior walls and partitions in 1607.14. Finally, a new section 1607.14.2 is proposed that clarifies that a fire wall and its supports must be designed for both the required gravity loads and the out-of-plane loads, but not concurrently.

Addition of these provisions will ensure proper design of fire walls for non-fire conditions and will maintain recognition of NFPA 221 fire walls designed and tested under full gravity loads in accordance with ASTM E119. This change also provides lateral design loads for evaluation of structural stability during the fire event. It is clarified that gravity and lateral loads do not need to be considered concurrently, consistent with the gravity-only basis of ASTM E119 tested fire walls and, therefore, ensures continued acceptability of NFPA 221 fire walls. ASTM E119 wall fire tests and, associated analysis methods, are the basis of fire resistance ratings for existing NFPA 221 fire walls.

Cost Impact: Will not increase the cost of construction
This change provides clarification with respect to the proper gravity and lateral loads and non-concurrence of these loads as they apply fire walls. This change may actually reduce cost by providing a method of ensuring currently-accepted NFPA 221 fire walls are still permitted without additional testing and engineering analysis.

S100-16 : 1607.14-COATS13298

S101-16

IBC: 1607.14.2 (New), 706.2 (New).

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, 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 w alls for structural loads. This code change w as submitted as public comment to FS28-15 that w as disapproved in Group A in Long Beach. This code change addresses the design of fire w alls and does not address the loading on fire w alls during an E119 fire test. The code change is proposed in Chapter 16 w ith a cross reference in Section 706.2, w here the other structural loads are defined. The engineers w ho are going to be required to design these w alls for structural loading w ill 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 Allow able Stress loading. The second sentence in the new Section 1607.14.2 then gives guidance to the design engineer w hat to use for Strength Design loading (w hich is w hat the vast majority of the loads in the code are calibrated to) and w hat 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 w here earthquake loads are not required and w here the conditions in the exception in ASCE 7 Section 13.5.8.1 and a design is required.

This in no w ay conflicts w ith or changes the requirements in NFPA 221--it is merely a conversion from one type of loading (Allow able 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 w ill all be coordinated. Without this code change, even if an engineer or code official w ere 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 w hat kind of load it is (allow able stress or strength design), or w hat 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 w alls tend to be large buildings w hose structures are designed by an engineer. The additional cost of designing the fire w alls 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 w alls tend to be large buildings w hose structures are designed by an engineer. The additional cost of designing the fire w alls for this load is not significant.

S101-16 : 1607.14.2 (NEW)-FATTAH11489

S102-16

IBC: 1608.2, 1608.3.

Proponent : Ronald Hamburger, SIMPMSON GUMPERTZ & HEGER, representing SELF (rohamburger@sgh.com)

2015 International Building Code

Revise as follows:

1608.2 Ground snow loads. The ground snow loads to be used in determining the design snow loads for roofs shall be determined in accordance with ASCE 7~~ or Figure 1608.2 for~~ . **Exception:** Where gound snow loads are specifically established by the ~~contiguous United States and Table 1608.2 for Alaska. Site-specific case studies~~ building official, those ground snow loads shall be ~~made in areas designated "CS" in Figure 1608.2~~ used. ~~Ground snow loads for sites at elevations above the limits indicated in Figure 1608.2 and for all sites within the CS areas shall be *approved*. Ground snow load determination for such sites shall be based on an extreme value statistical analysis of data available in the vicinity of the site using a value with a 2-percent annual probability of being exceeded (50-year mean recurrence interval). Snow loads are zero for Hawaii, except in mountainous regions as *approved* by the *building official*.~~

Delete without substitution:

~~1608.2~~
~~GROUND SNOW LOADS, p_g , FOR ALASKAN LOCATIONS~~

~~LOCATION~~	~~POUNDS PER SQUARE FOOT~~	~~LOCATION~~	~~POUNDS PER SQUARE FOOT~~	~~LOCATION~~	~~POUNDS PER SQUARE FOOT~~
~~Adak~~	~~30~~	~~Galena~~	~~60~~	~~Petersburg~~	~~150~~
~~Anchorage~~	~~50~~	~~Gulkana~~	~~70~~	~~St. Paul Islands~~	~~40~~
~~Angoon~~	~~70~~	~~Homer~~	~~40~~	~~Seward~~	~~50~~
~~Barrow~~	~~25~~	~~Juneau~~	~~60~~	~~Shemya~~	~~25~~
~~Barter Island~~	~~35~~	~~Kenai~~	~~70~~	~~Sitka~~	~~50~~
~~Bethel~~	~~40~~	~~Kodiak~~	~~30~~	~~Talkeetna~~	~~120~~
~~Big Delta~~	~~50~~	~~Kotzebue~~	~~60~~	~~Unalakleet~~	~~50~~
~~Cold Bay~~	~~25~~	~~McGrath~~	~~70~~	~~Valdez~~	~~160~~
~~Cordova~~	~~100~~	~~Nenana~~	~~80~~	~~Whittier~~	~~300~~

~~Fairbanks~~	~~60~~	~~Nome~~	~~70~~	~~Wrangell~~	~~60~~
~~Fort Yukon~~	~~60~~	~~Palmer~~	~~50~~	~~Yakutat~~	~~150~~

~~For SI: 1 pound per square foot = 0.0479 kN/m^2.~~

~~FIGURE 1608.2~~
~~GROUND SNOW LOADS, p_g, FOR THE UNITED STATES (psf)~~

~~FIGURE 1608.2~~
~~GROUND SNOW LOADS, p_g, FOR THE UNITED STATES (psf)~~

Revise as follows:

1608.3 Ponding instability. Susceptible bays of roofs shall be evaluated for ponding instability in accordance with ~~Section 7.11~~ Chapter 7 and Chapter 8 of ASCE 7.

Reason: Many years ago legacy building codes specified snow loading using simple procedures with reference to a snow loading map that specified design pressures on the basis of geographic location. Other factors that affected snow loads were neglected, including the propensity for snow to drift and the accumulated weight due to melting and refreezing. Over the years, roofs designed to these simple procedures proved vulnerable to snow-induced collapse. With time, the referenced loading standard *ASCE 7 Minimum Design Loads for Buildings and Other Structures* developed more complex snow load computation procedures that included consideration not only of regional climatic conditions, as represented by the ground snow load maps, but also surface elevation, geometry, and topography, as well as wind exposure, thermal conditions, and adjacency of taller structures. The International Building Code has continued to reproduce the ASCE 7 ground snow load map within the code, however the code references the ASCE 7 standard for the other parameters necessary to compute snow loads including the calculation procedure. This creates a condition in which unwary designers are able to reference the snow load map contained in the code and compute snow loads without reference to the provisions in ASCE 7, and in the process neglect the many other important factors and procedures necessary to properly determine safe design snow loads.
In its continuing effort to improve the reliability of the nation's built environment, 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 load data for individual cities in regions with highly variable climatic conditions associated with mountain and other factors. This data was assembled over many years through the efforts of the regional experts and the state Structural Engineers Associations (SEAs) with specialized knowledge in the local climatic conditions. The data was then vetted by the ASCE 7 Snow Loads Subcommittee for appropriate and consistent procedures. The ASCE 7 committee believes that duplication of this supplemental data within the IBC would inappropriately reinforce the incorrect assumption that safe design snow loads can be directly computed from the maps and data in the code without reference to the full design procedure contained in the provisions of ASCE 7. Rather than perpetuate this potentially dangerous condition further, this proposal completely removes the map from the code and points to the referenced design standard ASCE 7 for all required snow load data and computation procedures.

Complementing the publication of the 2016 edition of ASCE 7, the snow maps and associated data currently available in the code will be made available separately from purchasing the full standard so that building officials and other will still have access to this specific data.

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 snow loads 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 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).

S102-16 : 1608.2-HAMBURGER11549

S103-16

IBC: 1608.2, 1608.3.

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)

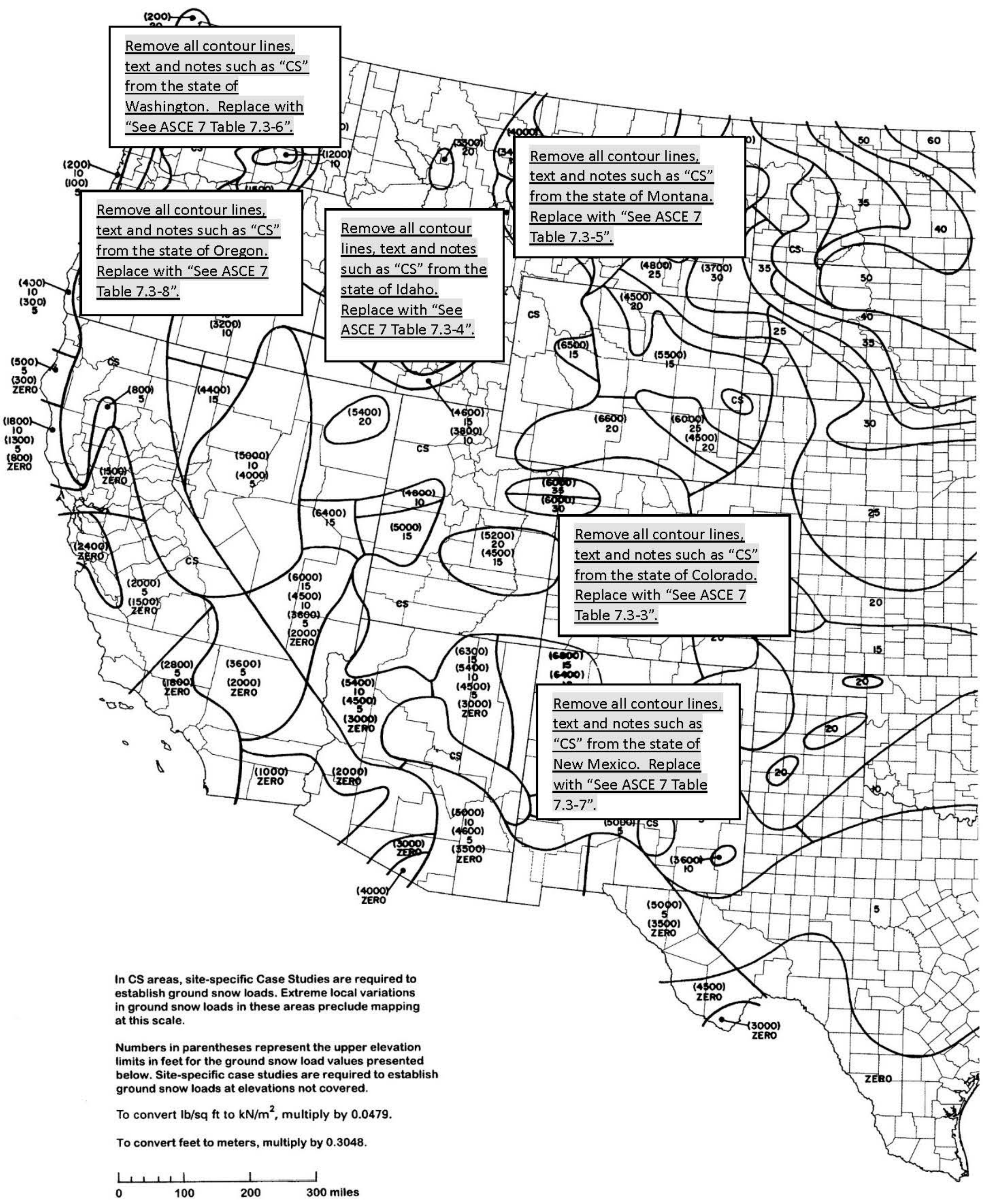


FIGURE 1608.2 Ground Snow Loads, p_g, for the United States (lb/ft^2)

NOTES: For state tables, see Chapter 7 of ASCE 7: See Table 7.3-3 for Colorado; See Table 7.3-4 for Idaho; See Table 7.3-5 for Montana; See Table 7.3-6 for Washington; See Table 7.3-7 for New Mexico; See Table 7.3-8 For Oregon

1608.3 Ponding instability. Susceptible bays of roofs shall be evaluated for ponding instability in

accordance with ~~Section 7.11~~ Chapter 7 and Chapter 8 of ASCE 7.

Reason: This proposed changes to Section 1608 w ill harmonize the provision in the code w ith 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 w ith an extensive database of ground snow loading data for individual cites in regions w ith highly variable climatic conditions associated w ith mountains and other factors. This data w as assembled over a period of many years through the efforts of regional experts and structural engineering associations w ith specialized know ledge in local climatic conditions and vetted by the Committee as having follow ed appropriate and consistent procedures. The revised map indicates w hich states have supplemental data w ithin the ASCE 7-16 standard.

Cost Impact: Will not increase the cost of construction
The proposed changes w ill not impact the cost of construction. The proposed changes w ill 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 w ith the referenced loading standard *ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. ASCE 7 w ill 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 ot he 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).

S103-16 : FIGURE 1608.2-GOUPIL12659

S104-16

IBC: 1609.1.1.

Proponent : Don Scott, representing National Council of Structural Engineering Associations (dscott@pcs-structural.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.
4. Designs using NAAMM FP 1001.
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. Wind loads on storm shelters shall be determined in accordance with ICC 500.

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: Section 423 Storm Shelters, which includes wind load criteria for storm shelters by reference to ICC 500, was first included in IBC 2009. Section 1609.1 is the IBC section that defines wind loads for buildings and structures, but it does not currently include specific criteria for storm shelters. Table 1604.5 includes hurricane and other emergency shelters in risk category IV. Since Chapter does not include any other requirements for storm shelter wind loads, it could be misinterpreted that risk category IV wind speeds are appropriate for storm shelters. Risk category IV wind speeds are based on a mean recurrence interval (MRI) of 3000 years, while ICC-500 wind speeds are based on a MRI of 10,000 years for hurricane shelters and a MRI of approximately 20,000 to 1,000,000 for tornados. This code change clarifies that ICC 500 wind loads are required for storm shelters.

Cost Impact: Will not increase the cost of construction
The cost of construction will not increase by clarifying storm shelter wind loads in Chapter 16, since wind loads for storm shelters are already required to be determined in accordance with ICC 500 by section 423.

S104-16 : 1609.1.1-SCOTT12737

S105-16

IBC: 1609.1.1.

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.
4. Designs using NAAMM FP 1001.
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_p) values given in these figures are significantly greater (in magnitude) than those given in previous versions (2010 and earlier)...".
The GC_p 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_p values were significantly higher in some cases. However, the GC_p 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_{pi}), topographic factor (K_{zt}), and velocity pressure exposure coefficient (K_z). The ASCE 7 debate did not focus on whether the roof component and cladding GC_p 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_p value increase from -0.9 to -2.0. If, 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_p 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_p revisions, but we will start with GC_p 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_p. 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.

S105-16 : 1609.1.1-SHOEMAKER13097

S106-16

IBC: 1609.1.2.

Proponent : Don Scott, representing National Council of Structural Engineering Associations (dscott@pcs-structural.com)

2015 International Building Code

Revise as follows:

1609.1.2 Protection of openings. In *wind-borne debris regions*, glazing in buildings shall be impact resistant or protected with an impact-resistant covering meeting the requirements of an *approved* impact-resistant standard or ASTM E 1996 and ASTM E 1886 referenced herein as follows:

1. Glazed openings located within 30 feet (9144 mm) of grade shall meet the requirements of the large missile test of ASTM E 1996.
2. Glazed openings located more than 30 feet (9144 mm) above grade shall meet the provisions of the small missile test of ASTM E 1996.

Exceptions:

1. Wood structural panels with a minimum thickness of ~~7~~ 23/~~16~~32 inch (~~11.1~~ 18.2 mm) and maximum panel span of 8 feet (2438 mm) shall be permitted for opening protection in buildings with a mean roof height of 33 feet (10 058 mm) or less that are classified as a Group R-3 or R-4 occupancy. Panels shall be precut so that they shall be attached to the framing surrounding the opening containing the product with the glazed opening. Panels shall be predrilled as required for the anchorage method and shall be secured with the attachment hardware provided. Attachments shall be designed to resist the components and cladding loads determined in accordance with the provisions of ASCE 7, with corrosion-resistant attachment hardware provided and anchors permanently installed on the building. Attachment in accordance with Table 1609.1.2 with corrosion-resistant attachment hardware provided and anchors permanently installed on the building is permitted for buildings with a mean roof height of 45 feet (13 716 mm) or less where V_{asd} determined in accordance with Section 1609.3.1 does not exceed 140 mph (63 m/s).
2. Glazing in *Risk Category* I buildings, including greenhouses that are occupied for growing plants on a production or research basis, without public access shall be permitted to be unprotected.
3. Glazing in *Risk Category* II, III or IV buildings located over 60 feet (18 288 mm) above the ground and over 30 feet (9144 mm) above aggregate surface roofs located within 1,500 feet (458 m) of the building shall be permitted to be unprotected.

Reason: Testing has shown that 7/16 inch panels do not meet the large missile tests of ASTM D 1996 and ASTM E 1886. These tests have been summarized in the "Hazard Mitigation Study for the Hawaii Hurricane Relief Fund, December 7, 2001 by Applied Research Associates, Inc." and the "Wind-borne Debris Test Facility and Safe Room Testing", by Dr. Ian Robertson, Gary Chock and Stephen Peters, for the University of Hawaii and the Hawaii State Civil Defense.

Cost Impact: Will increase the cost of construction
The initital cost of the opening protection with be increased by the material cost increase.

S107-16

IBC: 1609.5.2.

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. <u>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.</u>

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 300 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.

S107-16 : 1609.5.2-ENNIS11972

S108-16

IBC: 1609.6, 1609.6.1, 1609.6.1.1, 1609.6.2, 1609.6.3, 1609.6.4, 1609.6.4.1, 1609.6.4.2, 1609.6.4.3, 1609.6.4.4, 1609.6.4.4.1.

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 diaphragms.~~
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:~~

$\cancel{C_{net}}$	=	~~Net-pressure coefficient based on~~ $\cancel{K_d [(G)(C_p) - (GC_{pi})]}$~~, in accordance with Table 1609.6.2.~~
$\cancel{G}$	=	~~Gust-effect factor for rigid structures in accordance with ASCE 7 Section 26.9.1.~~
$\cancel{K_d}$	=	~~Wind directionality factor in accordance with ASCE 7 Table 26-6.~~
$\cancel{P_{net}}$	=	~~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.~~

$\cancel{P_{net} = 0.00256V^2 K_z C_{net} K_{zt}}$ ~~**(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_z and K_{zt}.** Velocity pressure exposure coefficient, K_z, 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_{zt} and K_z shall be based on height z.~~
2. ~~For leeward and sidewalls, and for windward and leeward roofs, K_{zt} and K_z shall be based on mean roof height h.~~

~~**1609.6.4.3 Determination of net pressure coefficients, C_{net}.** 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_{net}.~~

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~~
~~NET PRESSURE COEFFICIENTS, C_{net}[a, b]~~

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 7provisiosn and that engineers in the eastern United States did not have problems suing 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 fro 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 ot 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).

S108-16 : 1609.6-HAMBURGER11908

S109-16

IBC: 1609.6.2.

Proponent : Don Scott, representing National Council of Structural Engineering Associations (dscott@pcs-structural.com)

2015 International Building Code

Revise as follows:

TABLE 1609.6.2
NET PRESSURE COEFFICIENTS, C_{net}[a, b]

STRUCTURE OR PART THEREOF	DESCRIPTION		C_{net} FACTOR			
	Walls:		Enclosed		Partially enclosed	
			+ Internal pressure	- Internal pressure	+ Internal pressure	- Internal pressure
	Windward wall		0.43	0.73	0.11	1.05
	Leeward wall		-0.51	-0.21	-0.83	0.11
	Sidewall		-0.66	-0.35	-0.97	-0.04
	Parapet wall	Windward	1.28		1.28	
		Leeward	-0.85		-0.85	
	Roofs:		Enclosed		Partially enclosed	
	Wind perpendicular to ridge		+ Internal pressure	- Internal pressure	+ Internal pressure	- Internal pressure
	Leeward roof or flat roof		-0.66	-0.35	-0.97	-0.04
	Windward roof slopes:					
	Slope	Condition 1	-1.09	-0.79	-1.41	-0.47
		Condition 2	-0.28	0.02	-0.60	0.34

1. Main windforce-resisting frames and systems	Slope = 4:12 (18°)	Condition 1	-0.73	-0.42	-1.04	-0.11
		Condition 2	-0.05	0.25	-0.37	0.57
	Slope = 5:12 (23°)	Condition 1	-0.58	-0.28	-0.90	0.04
		Condition 2	0.03	0.34	-0.29	0.65
	Slope = 6:12 (27°)	Condition 1	-0.47	-0.16	-0.78	0.15
		Condition 2	0.06	0.37	-0.25	0.68
	Slope = 7:12 (30°)	Condition 1	-0.37	-0.06	-0.68	0.25
		Condition 2	0.07	0.37	-0.25	0.69
	Slope = 9:12 (37°)	Condition 1	-0.27	0.04	-0.58	0.35
		Condition 2	0.14	0.44	-0.18	0.76
	Slope = 12:12 (45°)		0.14	0.44	-0.18	0.76
	Wind parallel to ridge and flat roofs		-1.09	-0.79	-1.41	-0.47
	Nonbuilding Structures: Chimneys, Tanks and Similar Structures:					
				h/D		
				1	7	25
	Square (Wind normal to face)			0.99	1.07	1.53
	Square (Wind on diagonal)			0.77	0.84	1.15
	Hexagonal or octagonal			0.81	0.97	1.13
	Round			0.65	0.81	0.97
	Open signs and lattice frameworks			Ratio of solid to gross area		
					0.1 to 0.29	0.3 to 0.7

	Flat	1.45	1.30	1.16
	Round	0.87	0.94	1.08

STRUCTURE OR PART THEREOF	DESCRIPTION		C_{net} FACTOR	
	Roof elements and slopes		Enclosed	Partially enclosed
	Gable of hipped configurations (Zone 1)			
	Flat			
	Positive	10 square feet or less	~~0.58~~0.75	~~0.89~~1.06
		100 square feet or more	0.41	0.72
	Negative	10 square feet or less	~~1.00~~ 1.85	~~1.32~~ 2.17
		100 square feet or more	~~0.92~~ 1.68	~~1.23~~ 2.00
	Overhang: Flat			
	Negative	10 square feet or less	~~1.45~~ 2.28	
		100 square feet or more	~~1.36~~ 1.77	
		500 square feet or more	~~0.94~~ 1.68	
	6:12 (27°)			
2. Components and cladding not in areas of discontinuity—roofs	Positive	10 square feet or less	0.92	1.23
		100 square feet or	~~0.83~~0.58	~~1.15~~0.89

and overhangs		more		
	Negative	10 square feet or less	~~-1.00~~ 1.68	~~-1.32~~ 2.00
		100 square feet or more	-0.83	-1.15
	Monosloped configurations (Zone 1)		Enclosed	Partially enclosed
	Flat			
	Positive	10 square feet or less	0.49	0.81
		100 square feet or more	0.41	0.72
	Negative	10 square feet or less	-1.26	-1.57
		100 square feet or more	-1.09	-1.40
	Tall flat-topped roofs h > 60 feet		Enclosed	Partially enclosed
	Flat			
	Negative	10 square feet or less	-1.34	-1.66
		500 square feet or more	-0.92	-1.23
3. Components and cladding in areas of discontinuity—roofs and overhangs (continued)	Gable or hipped configurations at ridges, eaves and rakes (Zone 2)			
	Flat			
	Positive	10 square feet or less	~~0.58~~0.75	~~0.89~~1.06
		100 square feet or more	0.41	0.72
	Negative	10 square feet or less	~~-1.68~~ 2.28	~~-2.00~~ 2.59

		100 square feet or more	~~-1.17~~ 1.30	~~-1.49~~ 1.62
	Overhang for Slope Flat			
	Negative	10 square feet or less	~~-1.87~~ 3.13	
		100 square feet or more	~~-1.87~~ 2.11	
	6:12 (27°)		Enclosed	Partially enclosed
	Positive	10 square feet or less	0.92	1.23
		100 square feet or more	~~0.83~~0.58	~~1.15~~0.89
	Negative	10 square feet or less	~~-1.17~~ 2.49	~~-1.49~~ 2.76
		100 square feet or more	-1.00	-1.32
	Overhang for 6:12 (27°)			
	Negative	10 square feet or less	~~-1.70~~ 3.13	
		500 square feet or more	~~-1.53~~ 1.68	

STRUCTURE OR PART THEREOF	DESCRIPTION		C_{net} FACTOR	
	Roof elements and slopes		Enclosed	Partially enclosed
	Monosloped configurations at ridges, eaves and rakes (Zone 2)			
	Flat			
		10 square feet or less	0.49	0.81

	Positive	100 square feet or more	0.41	0.72
	Negative	10 square feet or less	-1.51	-1.83
		100 square feet or more	-1.43	-1.74
	Tall flat topped roofs *h* > 60 feet		Enclosed	Partially enclosed
	Flat			
	Negative	10 square feet or less	-2.11	-2.42
		500 square feet or more	-1.51	-1.83
	Gable or hipped configurations at corners (Zone 3) ~~See ASCE 7 Figure 30.4-2B Zone 3~~			
	Flat		Enclosed	Partially enclosed
	Positive	10 square feet or less	~~0.58~~0.75	~~0.89~~1.06
		100 square feet or more	0.41	0.72
	Negative	10 square feet or less	~~-2.53~~ 3.21	~~-2.85~~ 3.53
		100 square feet or more	~~-1.85~~ 1.62	~~-2.17~~ 2.00
	Overhang for Slope Flat			
3. Components and	Negative	10 square feet or less	~~-3.15~~ 4.15	

cladding in areas of discontinuity—roofs and overhangs		100 square feet or more	~~-2.13~~ 2.28	
	6:12 (27°)			
	Positive	10 square feet or less	0.92	1.23
		100 square feet or more	~~0.83~~0.58	~~1.15~~0.89
	Negative	10 square feet or less	~~-1.17~~ 3.21	~~-1.49~~ 3.53
		100 square feet or more	-1.00	-1.32
	Overhang for 6:12 (27°)		Enclosed	Partially enclosed
	Negative	10 square feet or less	~~-1.70~~ 3.89	
		100 square feet or more	~~-1.53~~ 1.68	
	Monosloped Configurations at corners (Zone 3) ~~See ASCE 7 Figure 30.4 5B Zone 3~~			
	Flat			
	Positive	10 square feet or less	0.49	0.81
		100 square feet or more	0.41	0.72
	Negative	10 square feet or less	-2.62	-2.93
		100 square feet or more	-1.85	-2.17

	Tall flat topped roofs *h* > 60 feet		Enclosed	Partially enclosed
	Flat			
	Negative	10 square feet or less	-2.87	-3.19
		500 square feet or more	-2.11	-2.42
4. Components and cladding not in areas of discontinuity—walls and parapets (continued)	Wall Elements: *h* ≤ 60 feet (Zone 4) ASCE 7 Figure 30.4-1		Enclosed	Partially enclosed
	Positive	10 square feet or less	1.00	1.32
		500 square feet or more	0.75	1.06
	Negative	10 square feet or less	-1.09	-1.40
		500 square feet or more	-0.83	-1.15
	Wall Elements: h > 60 feet (Zone 4) See ASCE 7 Figure 30.6-1 Zone 4			
	Positive	20 square feet or less	0.92	1.23
		500 square feet or more	0.66	0.98

STRUCTURE OR PART THEREOF	DESCRIPTION		C_{net} FACTOR	
	Negative	20 square feet or less	-0.92	-1.23
		500 square feet or	-0.75	-1.06

4. Components and cladding not in areas of discontinuity—walls and parapets		more		
	Parapet Walls			
	Positive		2.87	3.19
	Negative		-1.68	-2.00
5. Components and cladding in areas of discontinuity—walls and parapets	Wall elements: $h \leq 60$ feet (Zone 5) ASCE 7 Figure 30.4-1		**Enclosed**	**Partially enclosed**
	Positive	10 square feet or less	1.00	1.32
		500 square feet or more	0.75	1.06
	Negative	10 square feet or less	-1.34	-1.66
		500 square feet or more	-0.83	-1.15
	Wall elements: $h > 60$ feet (Zone 5) See ASCE 7 Figure 30.6-1 Zone 4			
	Positive	20 square feet or less	0.92	1.23
		500 square feet or more	0.66	0.98
	Negative	20 square feet or less	-1.68	-2.00
		500 square feet or more	-1.00	-1.32
	Parapet walls			
	Positive		3.64	3.95
	Negative		-2.45	-2.76

For SI: 1 foot = 304.8 mm, 1 square foot = $0.0929 m^2$, 1 degree = 0.0175 rad.

a. Linear interpolation between values in the table is permitted.

b. Some C_{net} 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 desifn 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

S110-16

IBC: 1611.1, 1611.2.

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 the ~~primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet~~ 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.

$R = 5.2(d_S + d_h)$ **(Equation 16-36)**

For SI: $R = 0.0098(d_s + d_h)$

where:

d_h	=	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_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).
R	=	Rain load on the undeflected roof, in psf (kN/m ~~2~~ ^{2}). 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 - Hydrometerorological Design Studies Center (http://hdsc.nws.noaa.gov/hdsc/pfds/index.html) for

precipitation intensity (inches per hour) based on the 100-year mean re-occurrence interval.

Graber, S. D. (2009). "Rain loads and flow attenuation on roofs." *J. Architectural Eng.*, 15(3), 91–101.

Cost Impact: Will increase the cost of construction
The proposed changes may impact the design of roofs w here the secondary (overflow) system w as previously based on an unconservative hydraulic head from a low er rainfall intensity. The changes harmonizes the roof load design for the structure w ith the expectations for the design of the roof drainage system. This proposal coordinate the IBC w ith the referenced loading standard *ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures,* w hich w as updated to be consistent w ith the International Plumbing Code provisions. ASCE 7 w ill 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.)

S110-16 : 1611.1-GOUPIL12688

S111-16

IBC: 1611.1, 1611.2, 1611.3.

Proponent : Ronald Hamburger, SIMPSON GUMPERTZ & HEGER, representing SELF (rohamburger@sgh.com)

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~~ in accordance with the ~~primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet~~ 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 hourly rainfall rate indicated in Figure 1611.1 or on other rainfall rates determined from *approved* local weather data.~~

~~$R = 5.2(d_s + d_h)$ **(Equation 16-36)**~~

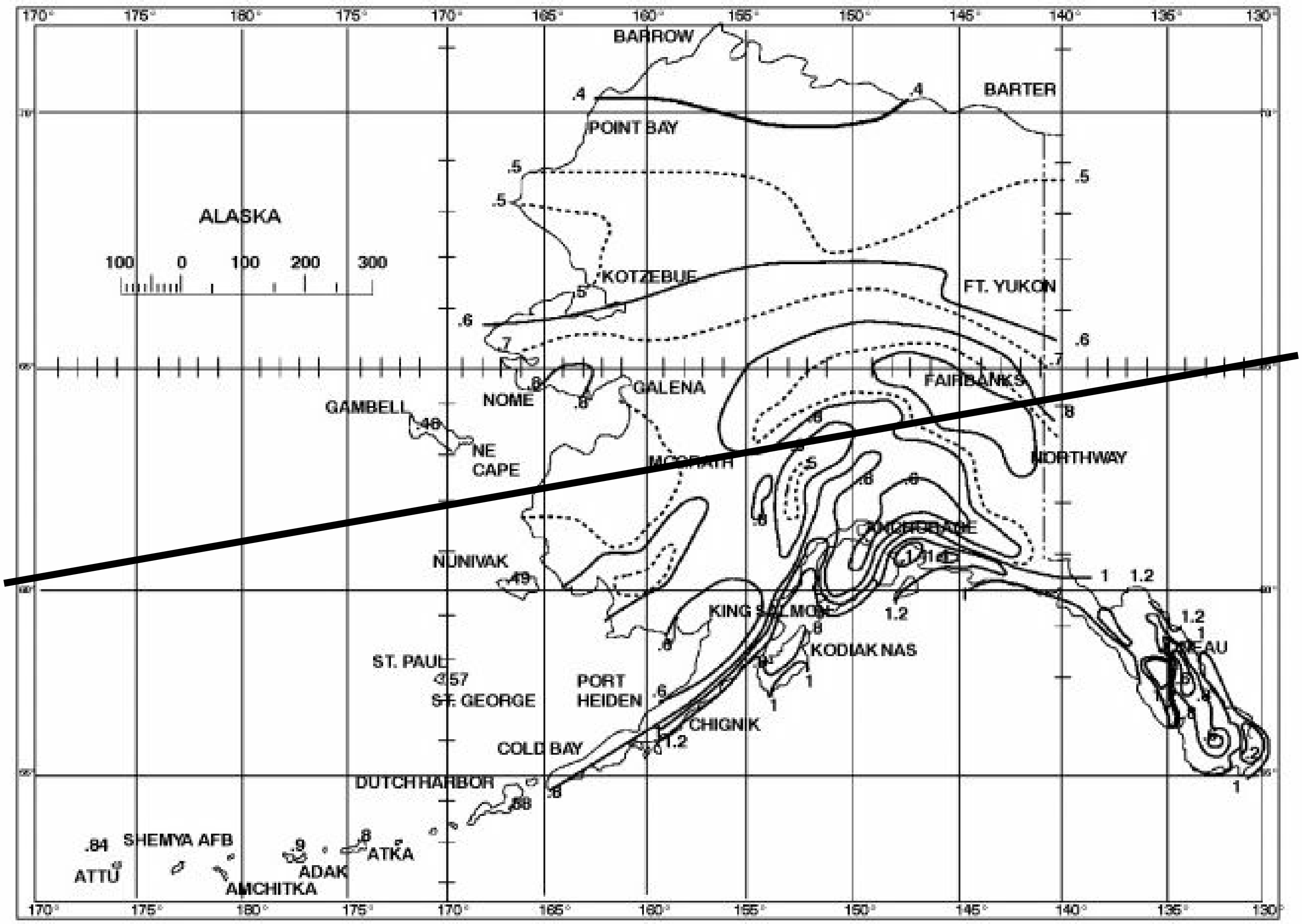
~~For SI: $R = 0.0098(d_s + d_h)$~~

~~where:~~

~~d_h~~	~~=~~	~~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_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).~~
~~R~~	~~=~~	~~Rain load on the undeflected roof, in psf (kN/m^2). 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.~~

Delete without substitution:

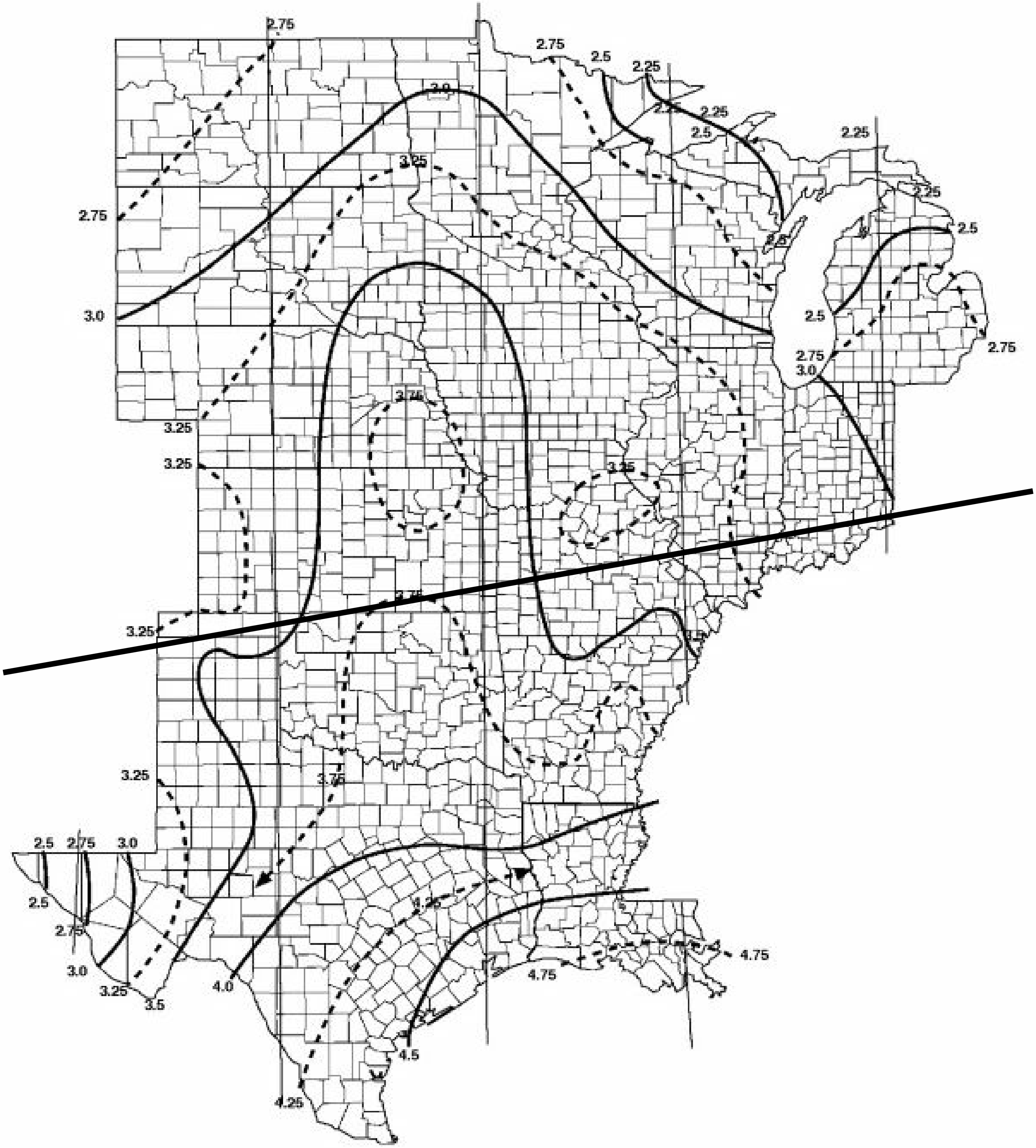
~~1611.1~~
~~100-YEAR, 1-HOUR RAINFALL (INCHES) ALASKA~~



~~For SI: 1 inch = 25.4 mm.~~

~~Source: National Weather Service, National Oceanic and Atmospheric Administration, Washington, DC.~~

~~1611.1~~
~~100-YEAR, 1-HOUR RAINFALL (INCHES) CENTRAL UNITED STATES~~



~~For SI: 1 inch = 25.4 mm.~~

~~Source: National Weather Service, National Oceanic and Atmospheric Administration, Washington, DC.~~

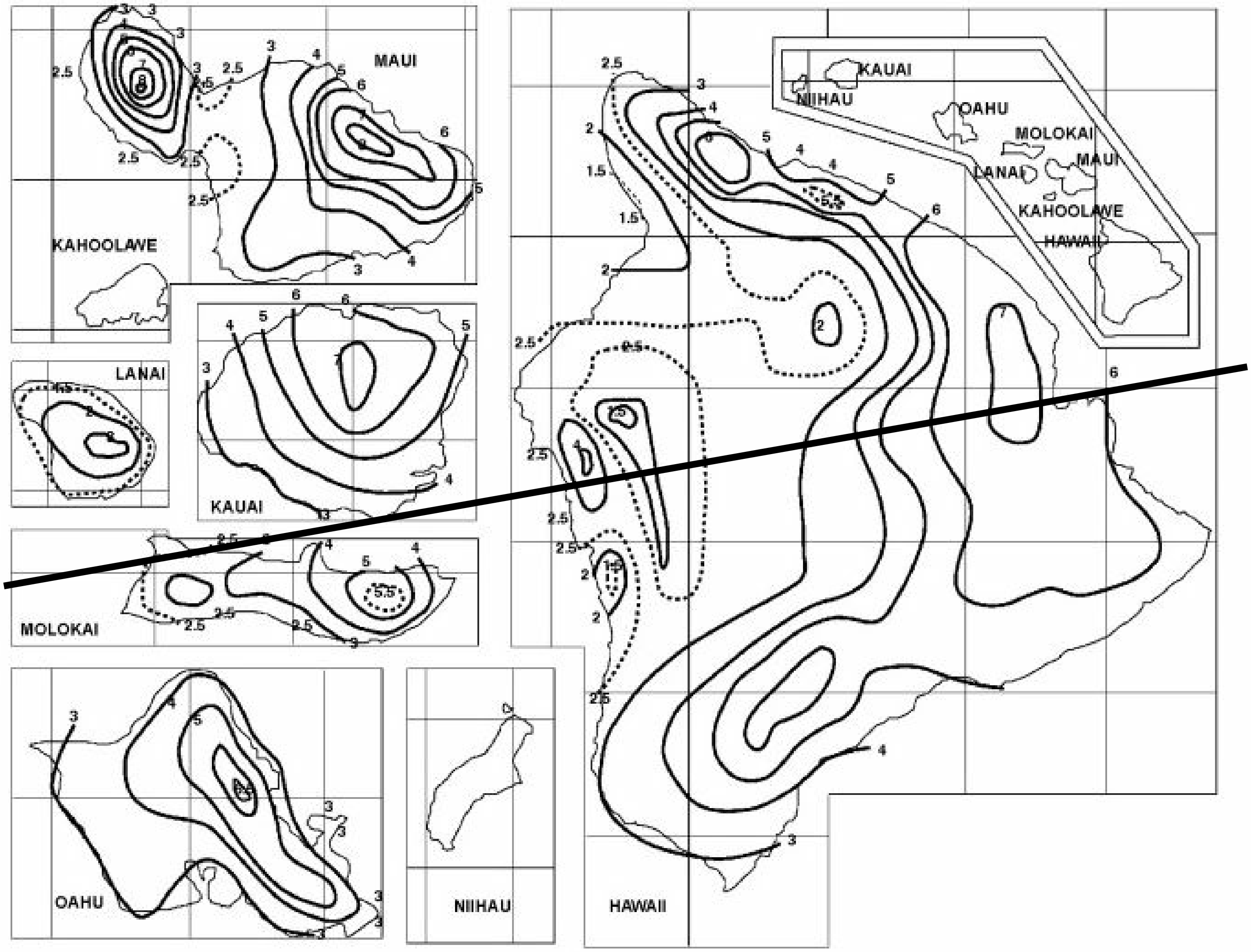
~~1611.1~~

~~100-YEAR, 1-HOUR RAINFALL (INCHES) EASTERN UNITED STATES~~

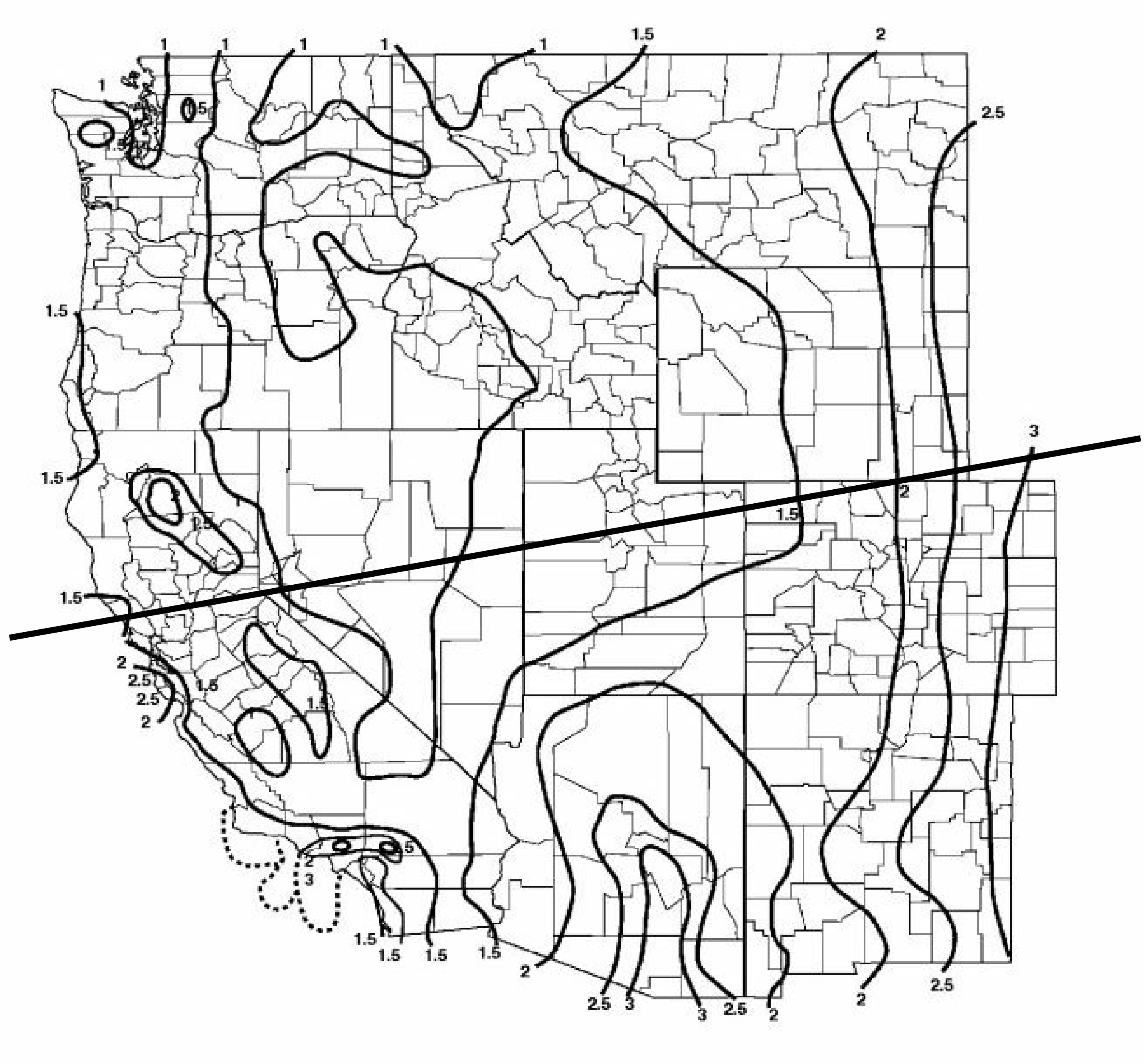
~~For SI: 1 inch = 25.4 mm.~~

~~Source: National Weather Service, National Oceanic and Atmospheric Administration, Washington, DC.~~

~~1611.1~~
~~100-YEAR, 1-HOUR RAINFALL (INCHES) HAWAII~~



~~1611.1~~
~~100-YEAR, 1-HOUR RAINFALL (INCHES) WESTERN UNITED STATES~~



~~For SI: 1 inch = 25.4 mm.~~

~~Source: National Weather Service, National Oceanic and Atmospheric Administration, Washington, DC.~~

~~For SI: 1 inch = 25.4 mm.~~

~~Source: National Weather Service, National Oceanic and Atmospheric Administration, Washington, DC.~~

Revise as follows:

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.

1611.3 Controlled drainage. Roofs equipped with hardware to control the rate of drainage shall be equipped with a secondary drainage system ~~at a higher elevation that limits accumulation of water on the roof above that elevation. Such roofs shall be designed to sustain the load of rainwater that will accumulate on them to the elevation of the secondary drainage system plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow determined from Section 1611.1. Such roofs shall also be checked for ponding~~

~~instability~~ in accordance with ~~Section 1611.2~~ the requirements of Chapter 8 of ASCE 7.

Reason: Section 1611 duplicates material contained in ASCE 7 Chapter 8. Since 2006, the IBC has specified such design procedures for most other loads, through adoptions of ASCE 7 by reference than transcribing the requirements into the body of IBC. Computation of adequacy to resist rain loads requires sophisticated second order analysis techniques that are only performed by civil and structural engineers, as opposed to other building code users. These engineers routinely refer directly to the ASCE 7 standard for other loading requirements and it does not create a hardship on the users to extend this practice to Rain Loads as w ell. This proposal removes duplicative material, replacing it w ith a reference to the ASCE 7 standard, in the process minimizing the potential for conflicts betw een the standard and the code.

Cost Impact: Will not increase the cost of construction
The proposed changes w ill not impact the cost of construction. This proposal is a re-organization of the pointers in the IBC to refer to the rain loads in the referenced loading standard ASCE 7. *ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures* w ill 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).

S111-16 : 1611.1-HAMBURGER12684

S112-16

IBC: 1613.3.3.

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

2015 International Building Code

Delete without substitution:

TABLE 1613.3.3 (2)
VALUES OF SITE COEFFICIENT F_V[a]

~~SITE CLASS~~	~~MAPPED SPECTRAL RESPONSE ACCELERATION AT 1 SECOND PERIOD~~				
	~~$S_1 \leq 0.1$~~	~~$S_1 = 0.2$~~	~~$S_1 = 0.3$~~	~~$S_1 = 0.4$~~	~~$S_1 \geq 0.5$~~
~~A~~	~~0.8~~	~~0.8~~	~~0.8~~	~~0.8~~	~~0.8~~
~~B~~	~~1.0~~	~~1.0~~	~~1.0~~	~~1.0~~	~~1.0~~
~~C~~	~~1.7~~	~~1.6~~	~~1.5~~	~~1.4~~	~~1.3~~
~~D~~	~~2.4~~	~~2.0~~	~~1.8~~	~~1.6~~	~~1.5~~
~~E~~	~~3.5~~	~~3.2~~	~~2.8~~	~~2.4~~	~~2.4~~
~~F~~	~~Note b~~	~~Note b~~	~~Note b~~	~~Note b~~	~~Note b~~

~~a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, S_1.~~

~~b. 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]

~~SITE CLASS~~	~~MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIOD~~				
	~~$S_S \leq 0.25$~~	~~$S_S = 0.50$~~	~~$S_S = 0.75$~~	~~$S_S = 1.00$~~	~~$S_S \geq 1.25$~~
~~A~~	~~0.8~~	~~0.8~~	~~0.8~~	~~0.8~~	~~0.8~~
~~B~~	~~1.0~~	~~1.0~~	~~1.0~~	~~1.0~~	~~1.0~~
~~C~~	~~1.2~~	~~1.2~~	~~1.1~~	~~1.0~~	~~1.0~~

~~D~~	~~1.6~~	~~1.4~~	~~1.2~~	~~1.1~~	~~1.0~~
~~E~~	~~2.5~~	~~1.7~~	~~1.2~~	~~0.9~~	~~0.9~~
~~F~~	~~Note b~~	~~Note b~~	~~Note b~~	~~Note b~~	~~Note b~~

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, S_S.

b. Values shall be determined in accordance with Section 11.4.7 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 processl.

References:

1988 Uniform Building Code
http://shop.iccsafe.org/1988-uniform-building-code-download-1.html
http://www.amazon.com/Uniform-Building-Code-1988-Edition/dp/B003EB5BB0

1990 SEAOC BLUE BOOK
http://seaoc.org/bookstore/overview-perspective-1988-ubc-and-blue-book-applicable-1990-edition

1997 Uniform Building Code
http://www.iamcivilengineer.com/2014/08/download-uniform-building-code-ubc-97.html

Robert E. Bachman and David R. Bonneville (2000)
The Seismic Provisions of the 1997 Uniform Building Code. Earthquake Spectra: February 2000, Vol. 16, No. 1, pp. 85-100.
http://www.earthquakespectra.org/doi/abs/10.1193/1.1586084?journalCode=eqsa

Bibliography: See also BIBLIOGRAPHY in Proposal: Figure 1613.3.1 RISK-TARGETED MCER
Cost Breakdown of Nonstructural Building Elements
Taghavi, S. and Miranda, E. (2003). Cost Breakdown of Nonstructural Building Elements, PEER Report 2003/05, Pacific 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 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://www.earthquakespectra.org/doi/abs/10.1193/1.4000032

Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Holliday, L. and Kang, T. (2014). "Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions." J. Perform. Constr. Facil., 10.1061/(ASCE)CF.1943-5509.0000630, 04014141.

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
California Seismic Safety Commission, 2005 Edition, 49 p.
http://www.disclosuresource.com/downloads/earthquake.pdf

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

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

Earthquake Architecture w ebsite
http://w w w .iitk.ac.in/nicee/w cee/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 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-dow n betw een map editions, this could effect cost accordingly, but I estimate that this w ould be minor.

This proposal may or may not affect the cost of construction. This is (1) because detached one- and tw o-family dw ellings must be already built to w ithstand the lateral forces due to w ind; 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 tw o-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 flaw ed 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 alw ays 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, 2013, 249 p. NEHRP Consultants Joint Venture A partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering.

In general, w here costs might be increased, cost premiums above requirements for w ind tend to fall w ithin a range of +1-3%. For cases w here seismic requirements w ould be now additional to w hat previous codes either applied/neglected/failed to enforce, estimates probably w ould fall w ithin the range of 0.25 - 1%.

S112-16 : 1613-BELA12959

S113-16

IBC: 1613.1.

Proponent : James Bela, self, representing Oregon Earthquakie 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_S, is less than 0.4 g.~~

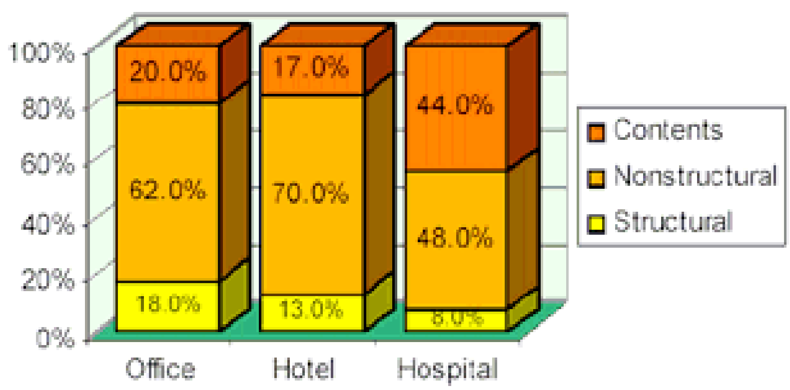
~~2. The seismic force-resisting system of wood-frame buildings that conform to the provisions of Section 2308 are not required to be analyzed as specified in this section.~~

~~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.~~

1. 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.
2. Agricultural storage structures intended only for incidental human occupancy.
3. 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



Cost breakdown of office buildings, hotels and hospitals

Bibliography: Cost Breakdown of Nonstructural Building Elements
Taghavi, S. and Miranda, E. (2003). Cost Breakdown of Nonstructural Building Elements, PEER Report 2003/05, Pacific Earthquake Engineering Rese3arch Center, U.C. Berkeley, 96 p.

Performance of Nonstructural Components during the 27 February 2010 Chile Earthquake.
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Holliday, L. and Kang, T. (2014). "Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions." J. Perform. Constr. Facil., 10.1061/(ASCE)CF.1943-5509.0000630, 04014141.
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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
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1988 Uniform Building Code
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http://www.amazon.com/Uniform-Building-Code-1988-Edition/dp/B003EB5BB0

1990 SEAOC BLUE BOOK
http://seaoc.org/bookstore/overview-perspective-1988-ubc-and-blue-book-applicable-1990-edition

1997 Uniform Building Code
http://www.iamcivilengineer.com/2014/08/download-uniform-building-code-ubc-97.html

Robert E. Bachman and David R. Bonneville (2000)
The Seismic Provisions of the 1997 Uniform Building Code. Earthquake Spectra: February 2000, Vol. 16, No.

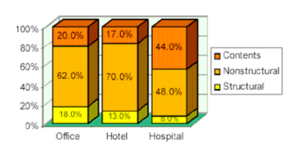
1, pp. 85-100.
http://w w w .earthquakespectra.org/doi/abs/10.1193/1.1586084?journalCode=eqsa

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 tw o-family dw ellings must be already built to w ithstand the lateral forces due to w ind; 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 tw o-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 flaw ed 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 alw ays 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, 2013, 249 p.
NEHRP Consultants Joint Venture A partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering.

In general, w here costs might be increased, cost premiums *above* requirements for w ind tend to fall w ithin a range of +1-3%. For cases w here seismic requirements w ould be now additional to w hat previous codes either applied/neglected/failed to enforce, *estimates* probably w ould fall w ithin the range of 0.25 - 1%.



S113-16 : 1613.1-BELA11816

S114-16

IBC: 1613.1, 1613.3.2, 1613.3.3, 1613.4, 1613.4.1, 1613.5, 1613.5.1, 1613.6.

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~~*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:

$S_{MS} = F_a S_s$ **(Equation 16-37)**

$S_{M1} = F_v S_1$ **(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.

~~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.~~

TABLE 1613.3.3 (2)
VALUES OF SITE COEFFICIENT F_V[a]

SITE CLASS	MAPPED RISK TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) SPECTRAL RESPONSE ACCELERATION PARAMETER AT 1-SECOND PERIOD					
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	S_1 ~~>=~~= 0.5	S_1>=0.6
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8~~1.0~~	0.8~~1.0~~	0.8~~1.0~~	0.8~~1.0~~	0.8 ~~1.0~~	0.8
C	1.5~~1.7~~	1.5~~1.6~~	1.5	1.5~~1.4~~	1.5~~1.3~~	1.4
D	2.4	2.2[c] ~~2.0~~	2.0[c]~~1.8~~	1.9[c]~~1.6~~	1.8[c]~~1.5~~	1.7[c]
E	4.2~~3.5~~	3.3[c]~~3.2~~	2.8[c]	2.4[c]	2.2[c]~~2.4~~	2.0[c]
F	Note b	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, S_1 .

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.

TABLE 1613.3.3 (1)
VALUES OF SITE COEFFICIENT F_a[a]

SITE CLASS	MAPPED RISK-TARGETED MACIMUM CONSIDERED EARTHQUAKE (MCE_R) SPECTRAL RESPONSE ACCELERATION PARAMETER AT SHORT PERIOD					
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	S_S ~~≥~~ = 1.25	$S_S \ge 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9 ~~1.0~~	0.9 ~~1.0~~	0.9 ~~1.0~~	0.9 ~~1.0~~	0.9 ~~1.0~~	0.9
C	1.3 ~~1.2~~	1.3 ~~1.2~~	1.2 ~~1.1~~	1.2 ~~1.0~~	1.2 ~~1.0~~	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4 ~~2.5~~	1.7	1.3 ~~1.2~~	Note b ~~0.9~~	Note b ~~0.9~~	Note b
F	Note b	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, S_S .

b. Values shall be determined in accordance with Section 11.4.7 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 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.~~

Reason:
This proposal is a coordination proposal to bring the 2018 IBC up to date w ith 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 w ithin ASCE 7-16, Appendix 11A w as 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 w ere inherently tied to the attenuation relationships that w e 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 w ell as IBC 2012 and th 2015 based on an updated set of attenuation relationships know n as the NGA equations. The old site class coefficients are not appropriate for use w ith 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 stud of the appropriate site class coefficients to use w ith 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 w ith 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_{DS} and S_{D1} parameters is unconservative for the design of long period buildings (T > 1 second) located on Site Class D or softer sites, w hen 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 w ill not impact the cost of construction. This proposal coordinates the IBC w ith 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).

S114-16 : 1613.1-GOUPIL12307

S115-16

IBC: 1613.1, 1613.2, 1613.3, 1613.3.1, 1613.3.2, 1613.3.3, 1613.3.4, 1613.3.5, 1613.3.5.1, 1613.3.5.2, 1613.4, 1613.4.1, 1613.5, 1613.5.1, 1613.6.

Proponent : Ronald Hamburger, SIMPSON GUMPERTZ & HEGER (rohamburger@sgh.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 Chapters 11, ~~excluding Chapter 14~~ 12, 13, 15, 17, and ~~Appendix 11A~~ 18, as applicable. The ~~*seismic design category*~~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.

1613.2 Definitions. The following terms are defined in Chapter 2:

~~**DESIGN EARTHQUAKE GROUND MOTION.**~~

~~**ORTHOGONAL.**~~

~~**RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATION.**~~

SEISMIC DESIGN CATEGORY.

~~**SEISMIC FORCE-RESISTING SYSTEM.**~~

~~**SITE CLASS.**~~

~~**SITE COEFFICIENTS.**~~

1613.3 Seismic ground motion values. Seismic ground motion ~~values~~ parameters S_S, S_1, S_{MS}, S_{M1}, S_{DS}, and S_{D1} and design acceleration response spectra shall be determined in accordance with ~~this section~~ ASCE 7 Chapter 11. The site specific ground motion procedures of ASCE 7 Chapter 21 shall be permitted to be used for any structures and shall be used where required by ASCE 7 Chapter 11.

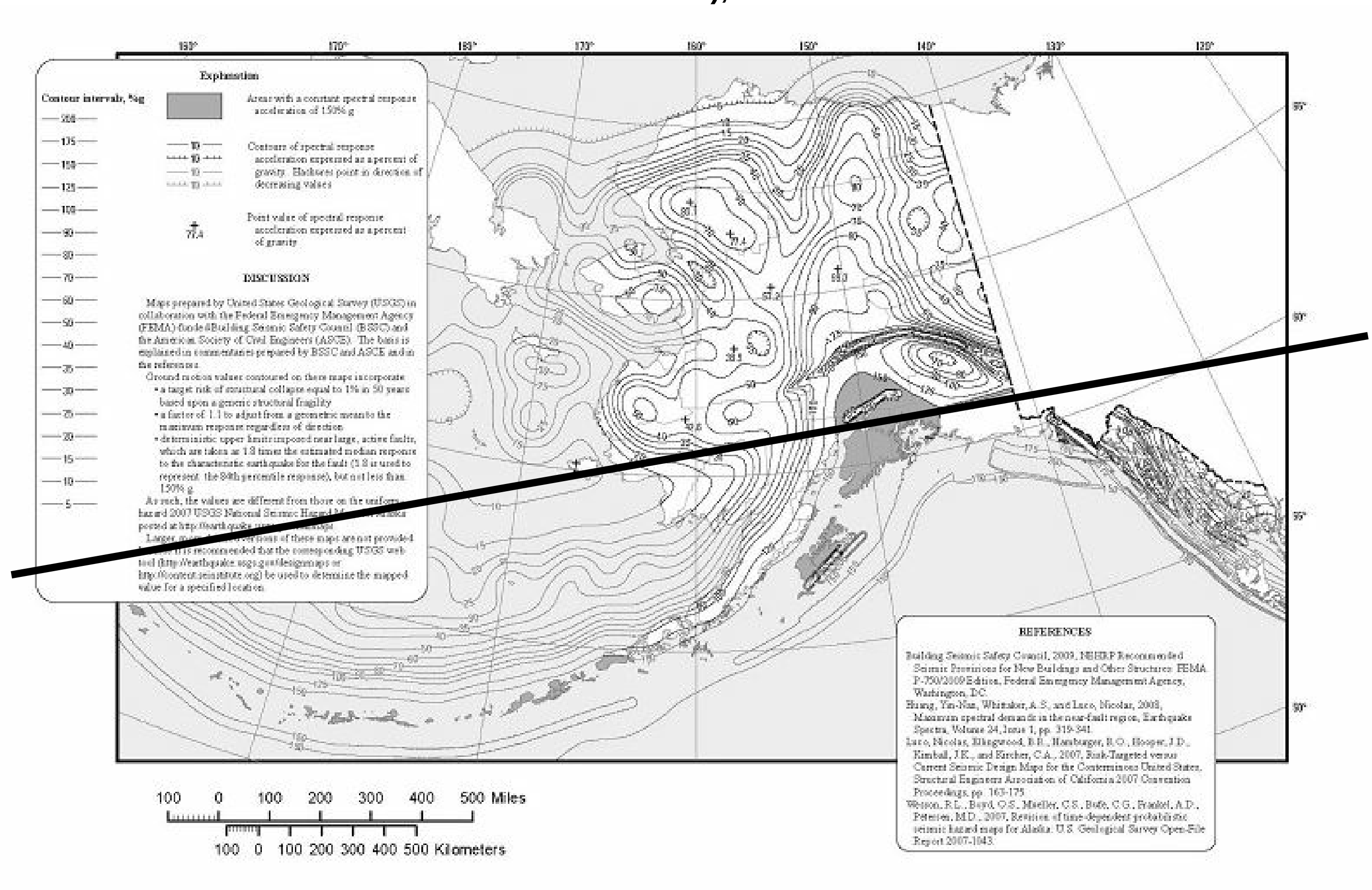
EXCEPTION: Where parameters S_S, S_1, S_{MS}, S_{M1}, S_{DS} and S_{D1} or design acceleration response spectra have been specifically established by the *building official*, those values shall be used.

Delete without substitution:

~~**1613.3.1 Mapped acceleration parameters.** The parameters S_S and S_1 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). Where S_1 is less than or equal to 0.04 and S_S is less than or equal to 0.15, the structure is permitted to be assigned *Seismic Design Category* A.~~

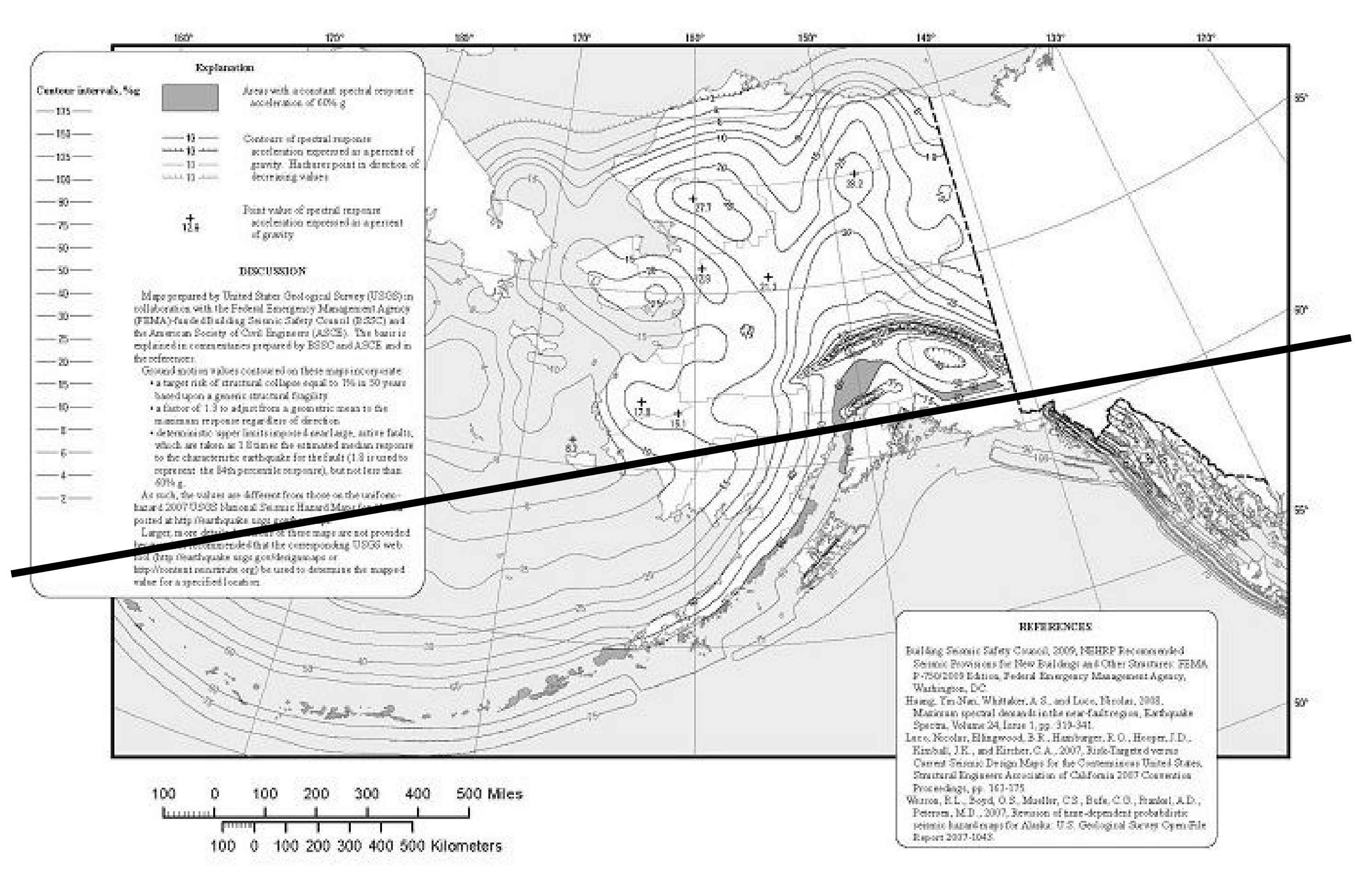
~~1613.3.1~~ (4)

~~RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR ALASKA OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B~~



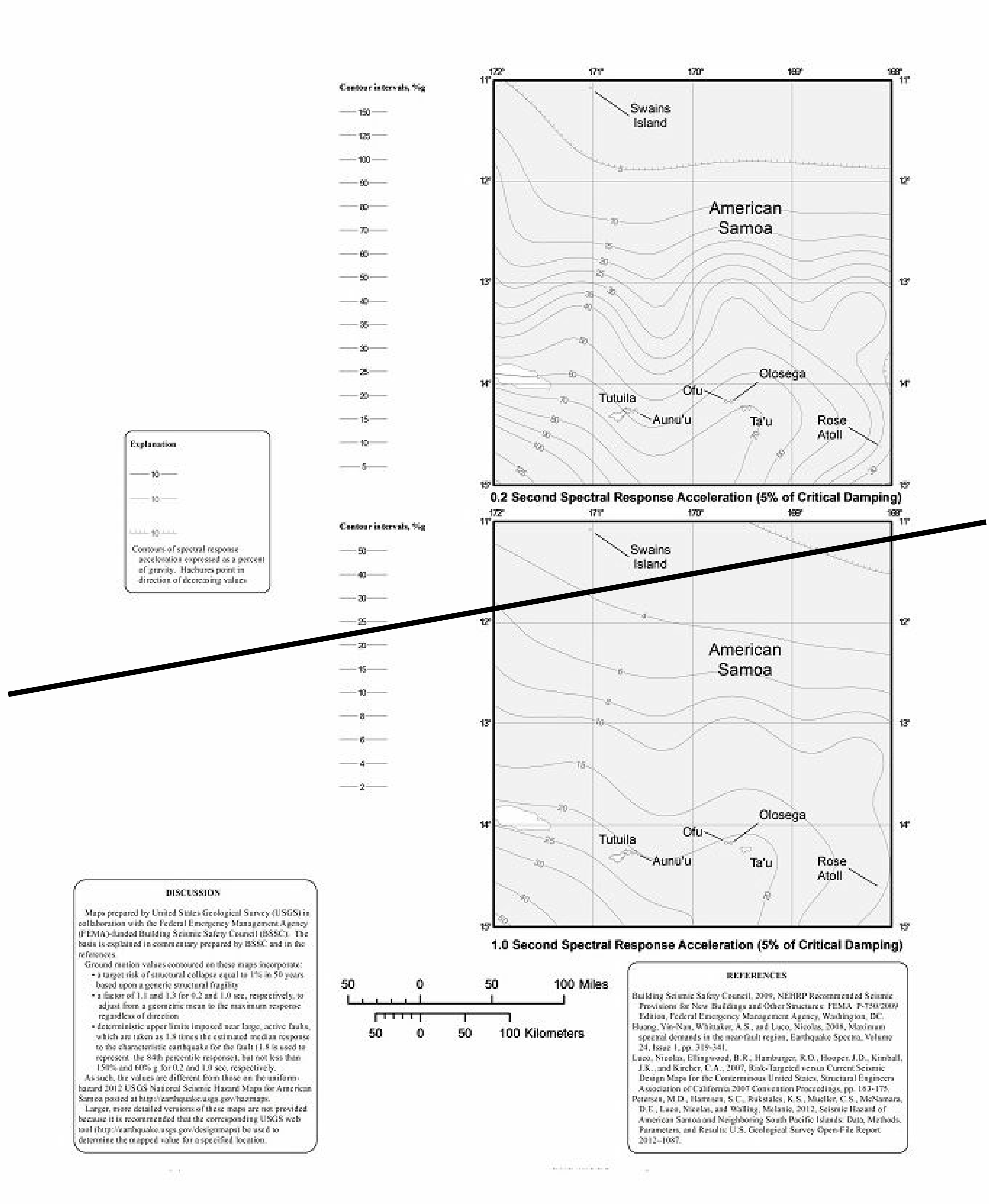
~~1613.3.1~~ (5)

~~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~~



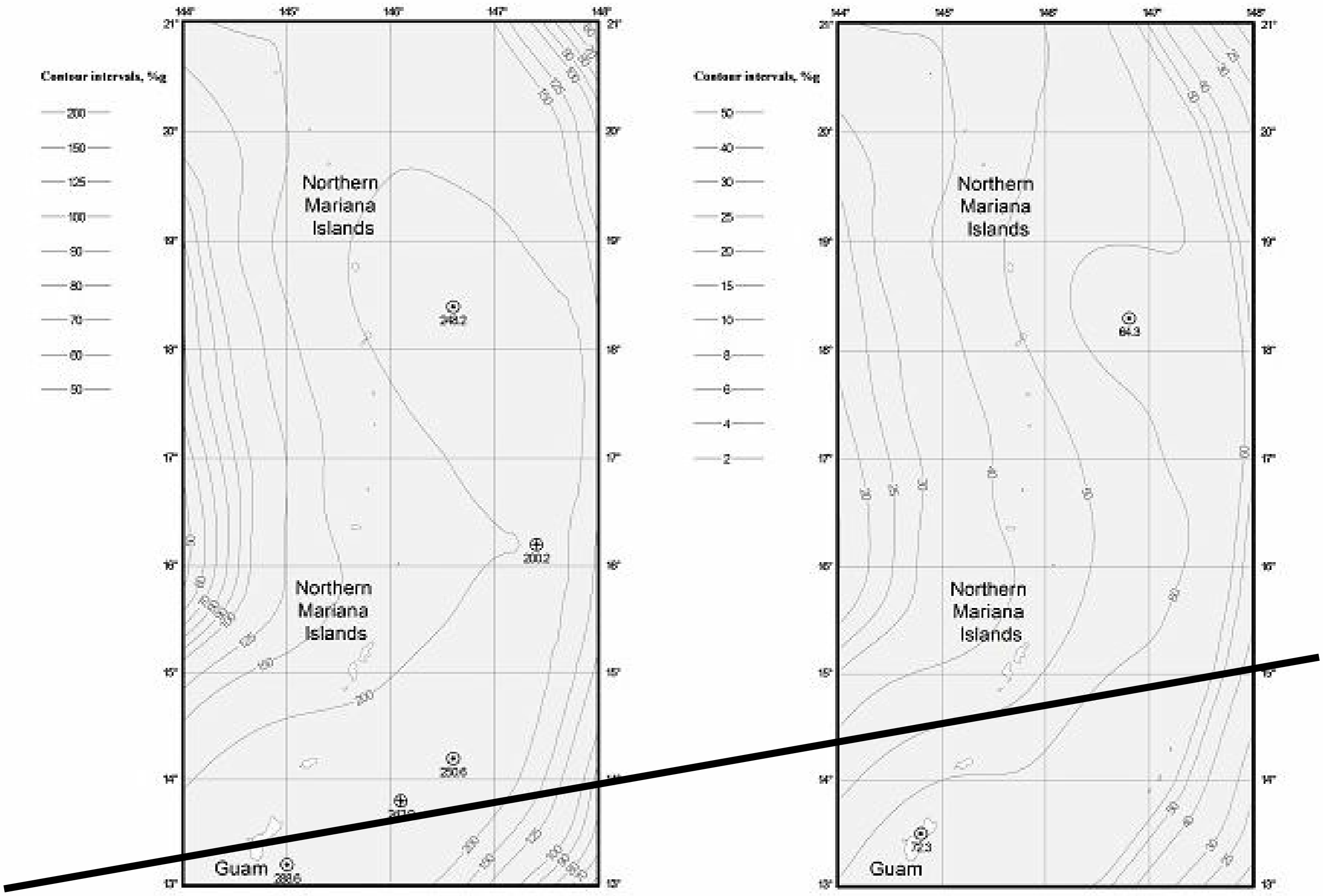
~~1613.3.1~~ (8)

~~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~~



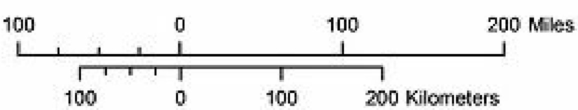
~~1613.3.1~~ (7)

~~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~~



0.2 Second Spectral Response Acceleration (5% of Critical Damping)

1.0 Second Spectral Response Acceleration (5% of Critical Damping)



Explanation

Contours of spectral response acceleration expressed as a percent of gravity.

Point values of spectral response acceleration expressed as a percent of gravity.

200.2 Local minimum

250.6 Local maximum

243.9 Saddle point

DISCUSSION

Maps prepared by United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency (FEMA)-funded Building Seismic Safety Council (BSSC). The basis is explained in commentary prepared by BSSC and in the references.

Ground motion values contoured on these maps incorporate:

- a target risk of structural collapse equal to 1% in 50 years based upon a generic structural fragility
- a factor of 1.1 and 1.3 for 0.2 and 1.0 sec, respectively, to adjust from a geometric mean to the maximum response regardless of direction
- deterministic upper limits imposed near large, active faults, which are taken as 1.8 times the estimated median response to the characteristic earthquake for the fault (1.8 is used to represent the 84th percentile response), but not less than 150% and 60% g for 0.2 and 1.0 sec, respectively.

As such, the values are different from those on the uniform-hazard 2012 USGS National Seismic Hazard Maps for Guam and the Northern Mariana Islands posted at http://earthquake.usgs.gov/hazmaps.

Larger, more detailed versions of these maps are not provided because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/designmaps) be used to determine the mapped value for a specified location.

REFERENCES

Building Seismic Safety Council, 2009, NEHRP Recommended Seismic Provisions for New Buildings and Other Structures: FEMA P-750/2009 Edition, Federal Emergency Management Agency, Washington, DC.

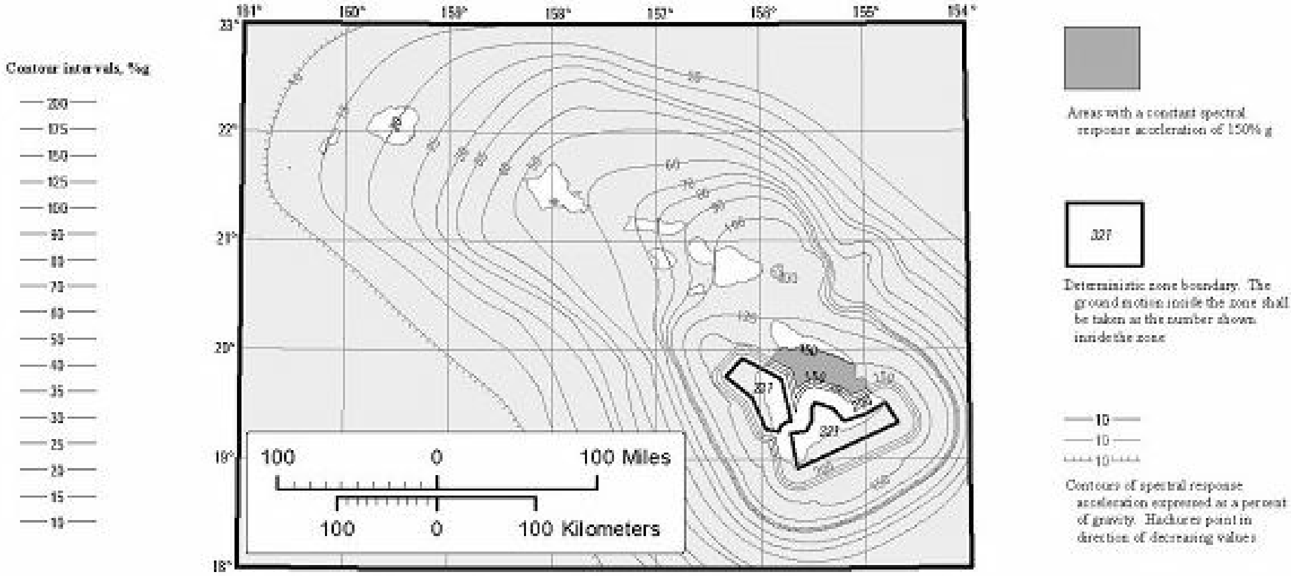
Huang, Yin-Nan, Whittaker, A.S., and Luco, Nicolas, 2008, Maximum spectral demands in the near-fault region, Earthquake Spectra, Volume 24, Issue 1, pp. 319-341.

Luco, Nicolas, Ellingwood, B.R., Hamburger, R.O., Hooper, J.D., Kimball, J.K., and Kircher, C.A., 2007, Risk-Targeted versus Current Seismic Design Maps for the Conterminous United States, Structural Engineers Association of California 2007 Convention Proceedings, pp. 163-175.

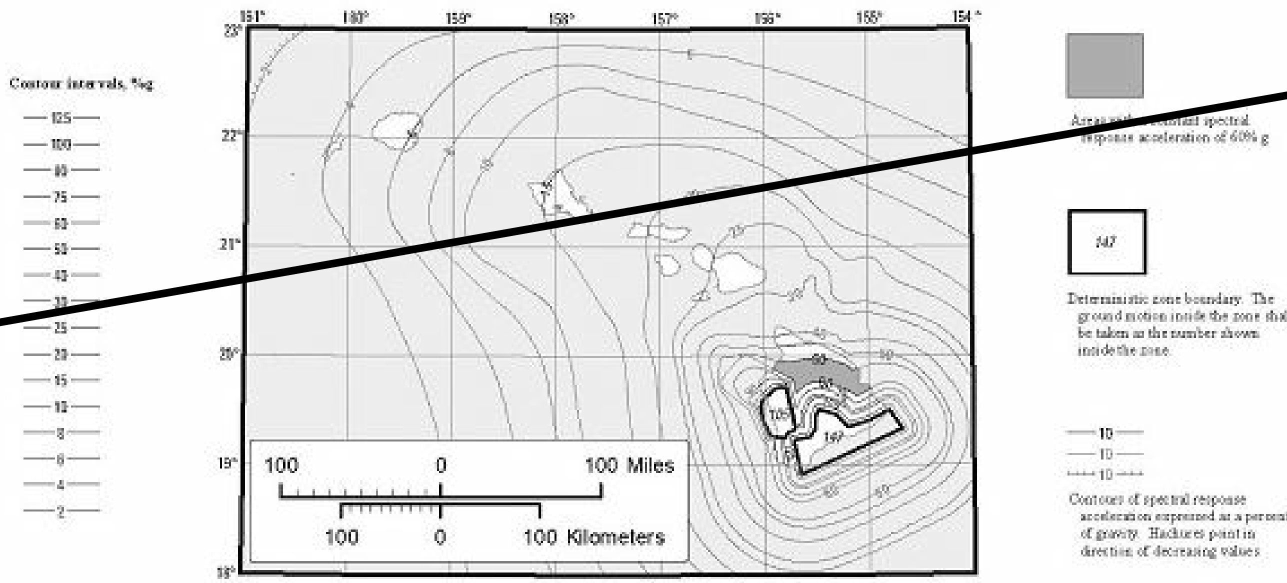
Mueller, C.S., Haller, K.M., Luco, Nicolas, Petersen, M.D., and Frankel, A.D., 2012, Seismic Hazard Assessment for Guam and the Northern Mariana Islands: U.S. Geological Survey Open-File Report 2012–1015.

~~1613.3.1~~ (3)

~~RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR HAWAII OF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B~~



0.2 Second Spectral Response Acceleration (5% of Critical Damping)



1.0 Second Spectral Response Acceleration (5% of Critical Damping)

DISCUSSION

Maps prepared by United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency (FEMA)-funded Building Seismic Safety Council (BSSC) and the American Society of Civil Engineers (ASCE). The basis is explained in commentaries prepared by BSSC and ASCE and in the references.

Ground motion values contoured on these maps incorporate:

- a target risk of structural collapse equal to 1% in 50 years based upon a generic structural fragility
- deterministic upper limits imposed near large, active faults, which are taken as 1.8 times the estimated median response to the characteristic earthquake for the fault (1.8 is used to represent the 84th percentile response), but not less than 150% and 60% g for 0.2 and 1.0 sec, respectively.

As such, the values are different from those on the uniform-hazard 1998 USGS National Seismic Hazard Maps for Hawaii posted at http://earthquake.usgs.gov/hazmaps.

Larger, more detailed versions of these maps are not provided because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/designmaps or http://content.seinstitute.org) be used to determine the mapped value for a specified location.

REFERENCES

Building Seismic Safety Council, 2009, NEHRP Recommended Seismic Provisions for New Buildings and Other Structures: FEMA P-750/2009 Edition, Federal Emergency Management Agency, Washington, DC.

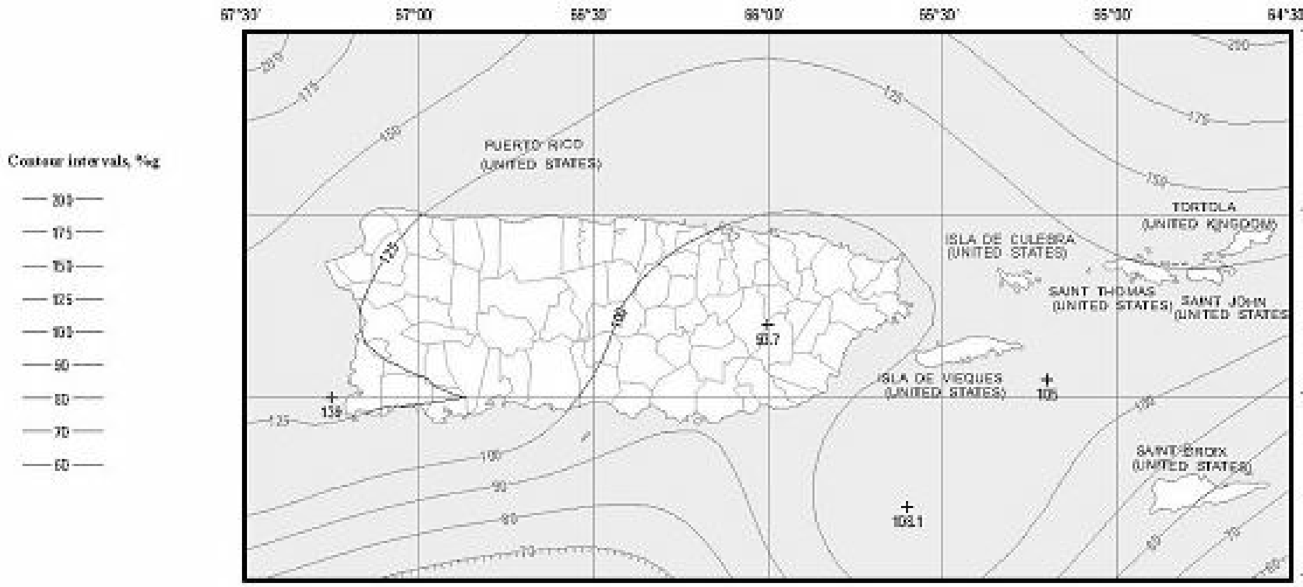
Huang, Yin-Nan, Whittaker, A.S., and Luco, Nicolas, 2008, Maximum spectral demands in the near-fault region, Earthquake Spectra, Volume 24, Issue 1, pp. 319-341.

Klein, F., Frankel, A.D., Mueller, C.S., Wesson, R.L., and Okubo, P., 2001, Seismic hazard in Hawaii: high rate of large earthquakes and probabilistic ground-motion maps, Bulletin of the Seismological Society of America, Volume 91, pp. 479-498.

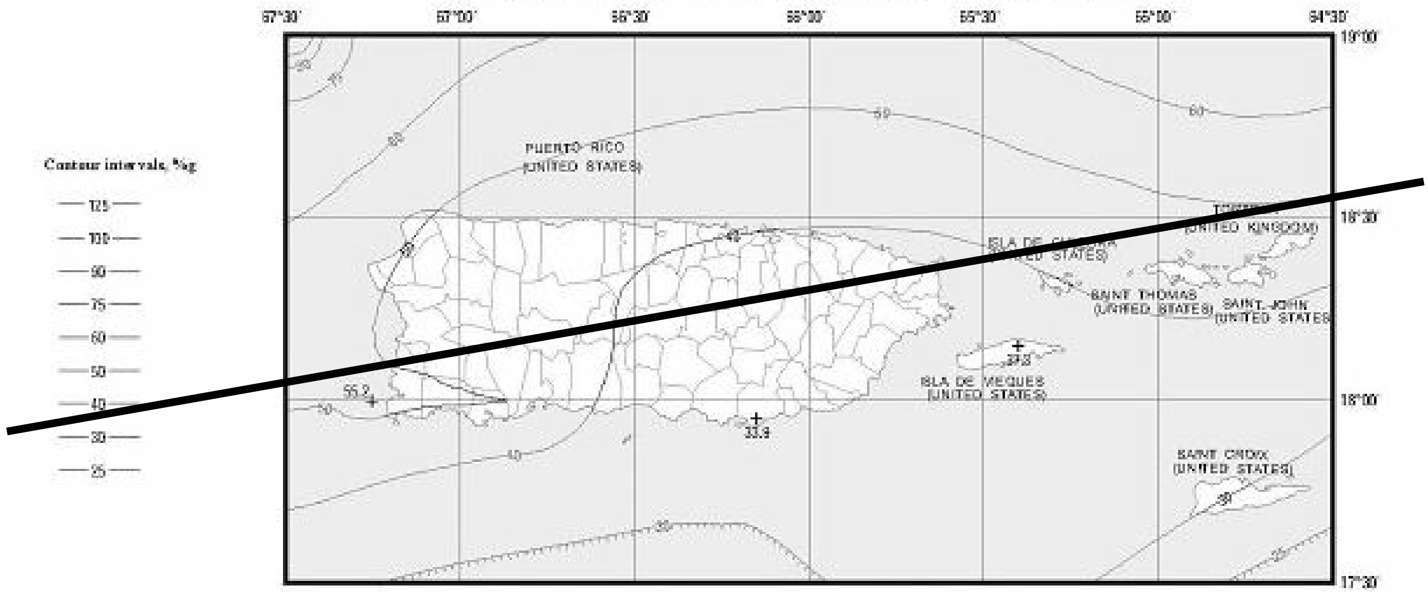
Luco, Nicolas, Ellingwood, B.R., Hamburger, R.O., Hooper, J.D., Kimball, J.K., and Kircher, C.A., 2007, Risk-Targeted versus Current Seismic Design Maps for the Conterminous United States, Structural Engineers Association of California 2007 Convention Proceedings, pp. 163-175.

~~1613.3.1~~ (6)

~~RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR PUERTO RICO AND THE UNITED STATES VIRGIN ISLANDSOF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B~~



0.2 Second Spectral Response Acceleration (5% of Critical Damping)



1.0 Second Spectral Response Acceleration (5% of Critical Damping)

Explanation

Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values

93.7

Point value of spectral response acceleration expressed as a percent of gravity

DISCUSSION

Maps prepared by United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency (FEMA)-funded Building Seismic Safety Council (BSSC) and the American Society of Civil Engineers (ASCE). The basis is explained in commentaries prepared by BSSC and ASCE and in the references.

Ground motion values contoured on these maps incorporate:

- a target risk of structural collapse equal to 1% in 50 years based upon a generic structural fragility
- a factor of 1.1 and 1.3 for 0.2 and 1.0 sec, respectively, to adjust from a geometric mean to the maximum response regardless of direction
- deterministic upper limits imposed near large, active faults, which are taken as 1.8 times the estimated median response to the characteristic earthquake for the fault (1.8 is used to represent the 84th percentile response), but not less than 150% and 60% g for 0.2 and 1.0 sec, respectively.

As such, the values are different from those on the uniform-hazard 2003 USGS National Seismic Hazard Maps for Puerto Rico and the U.S. Virgin Islands posted at http://earthquake.usgs.gov/hazmaps.

Larger, more detailed versions of these maps are not provided because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/designmaps or http://content.seinstitute.org) be used to determine the mapped value for a specified location.

50 0 50 Miles

50 0 50 Kilometers

REFERENCES

Building Seismic Safety Council, 2009, NEHRP Recommended Seismic Provisions for New Buildings and Other Structures: FEMA P-750/2009 Edition, Federal Emergency Management Agency, Washington, DC.

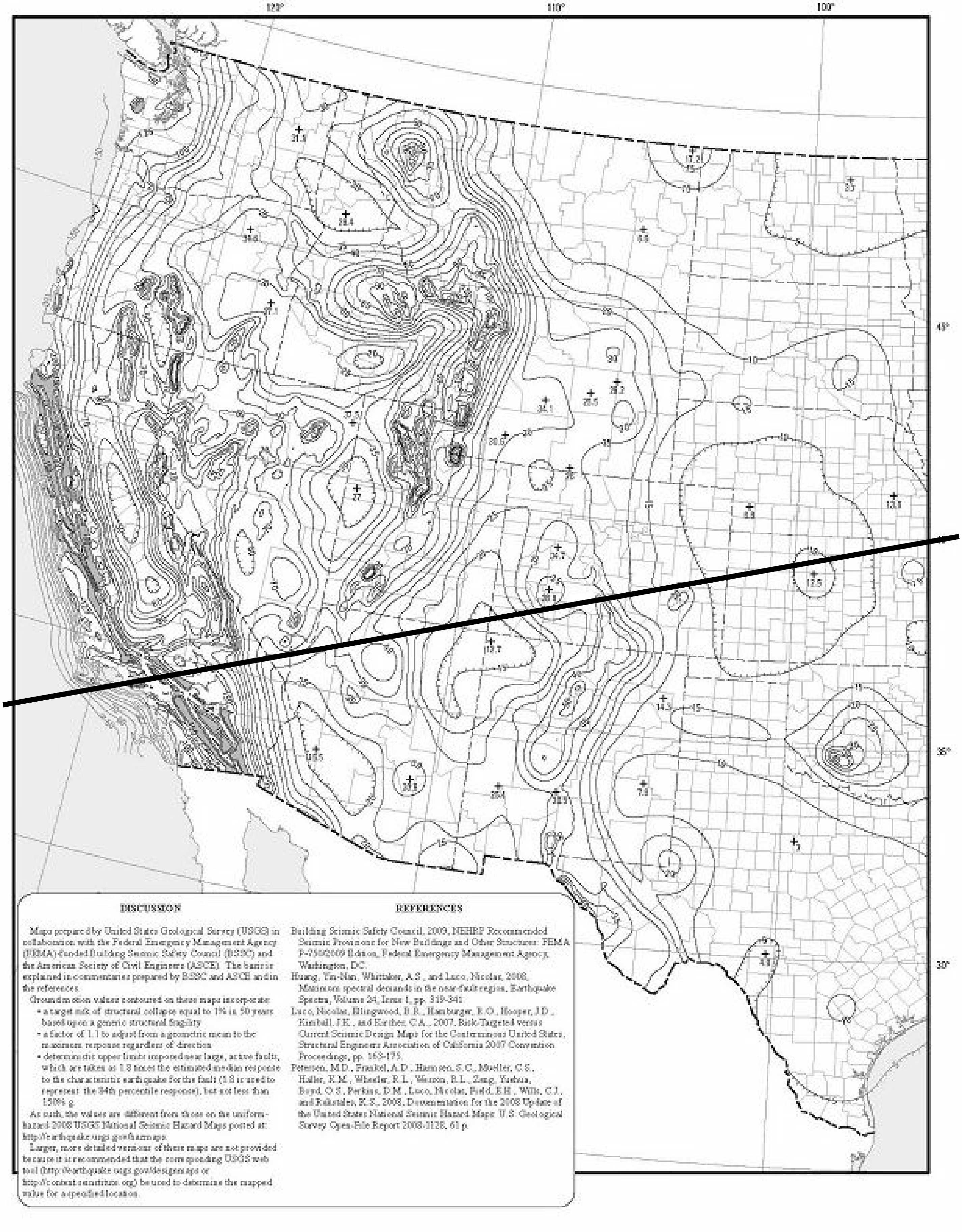
Huang, Yin-Nan, Whittaker, A.S., and Luco, Nicolas, 2008, Maximum spectral demands in the near-fault region, Earthquake Spectra, Volume 24, Issue 1, pp. 319-341.

Luco, Nicolas, Ellingwood, B.R., Hamburger, R.O., Hooper, J.D., Kimball, J.K., and Kircher, C.A., 2007, Risk-Targeted versus Current Seismic Design Maps for the Conterminous United States, Structural Engineers Association of California 2007 Convention Proceedings, pp. 163-175.

Mueller, C.S., Frankel, A.D., Petersen, M.D., and Leyendecker, E.V., 2003, Documentation for the 2003 USGS Seismic Hazard Maps for Puerto Rico and the U.S. Virgin Islands: U.S. Geological Survey Open-File Report 03-379.

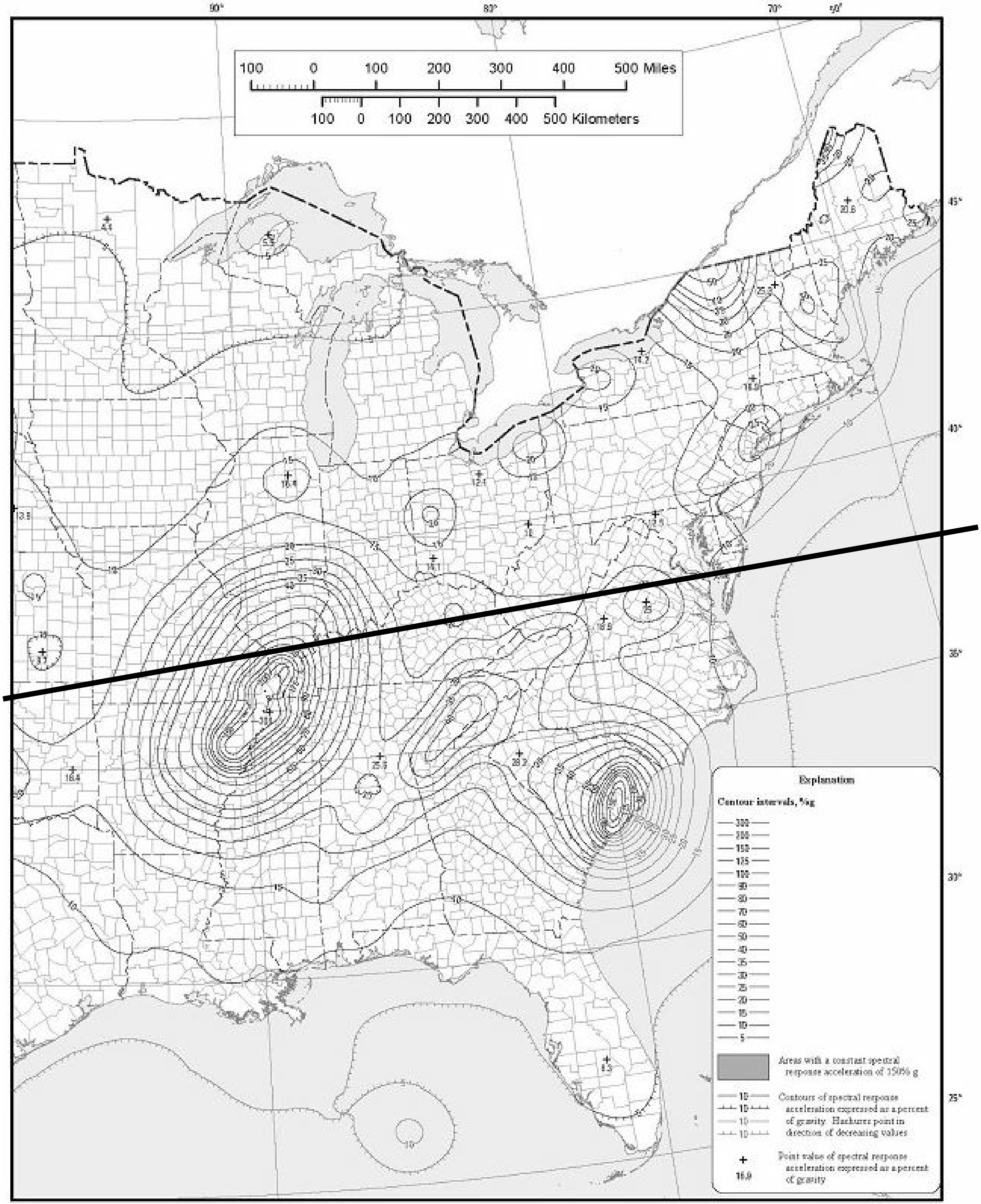
~~1613.3.1~~ (1)

~~RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B~~


120°
110°
100°
45°
35°
30°
DISCUSSION
Maps prepared by United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency (FEMA)-funded Building Seismic Safety Council (BSSC) and the American Society of Civil Engineers (ASCE). The basis is explained in commentaries prepared by BSSC and ASCE and in the references.
Ground motion values contoured on these maps incorporate:
• a target risk of structural collapse equal to 1% in 50 years based upon a generic structural fragility
• a factor of 1.1 to adjust from a geometric mean to the maximum response regardless of direction
• deterministic upper limits imposed near large, active faults, which are taken as 1.8 times the estimated median response to the characteristic earthquake for the fault (1.8 is used to represent the 84th percentile response), but not less than 150% g.
As such, the values are different from those on the uniform-hazard 2008 USGS National Seismic Hazard Maps posted at: http://earthquake.usgs.gov/hazmaps.
Larger, more detailed versions of these maps are not provided because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/designmaps or http://content.seinstitute.org) be used to determine the mapped value for a specified location.
REFERENCES
Building Seismic Safety Council, 2009, NEHRP Recommended Seismic Provisions for New Buildings and Other Structures: FEMA P-750/2009 Edition, Federal Emergency Management Agency, Washington, DC.
Huang, Yin-Nan, Whittaker, A.S., and Luco, Nicolas, 2008, Maximum spectral demands in the near-fault region, Earthquake Spectra, Volume 24, Issue 1, pp. 319-341.
Luco, Nicolas, Ellingwood, B.R., Hamburger, R.O., Hooper, J.D., Kimball, J.K., and Kircher, C.A., 2007, Risk-Targeted versus Current Seismic Design Maps for the Conterminous United States, Structural Engineers Association of California 2007 Convention Proceedings, pp. 163-175.
Petersen, M.D., Frankel, A.D., Harmsen, S.C., Mueller, C.S., Haller, K.M., Wheeler, R.L., Wesson, R.L., Zeng, Yuehua, Boyd, O.S., Perkins, D.M., Luco, Nicolas, Field, E.H., Wills, C.J., and Rukstales, K.S., 2008, Documentation for the 2008 Update of the United States National Seismic Hazard Maps: U.S. Geological Survey Open-File Report 2008-1128, 61 p.

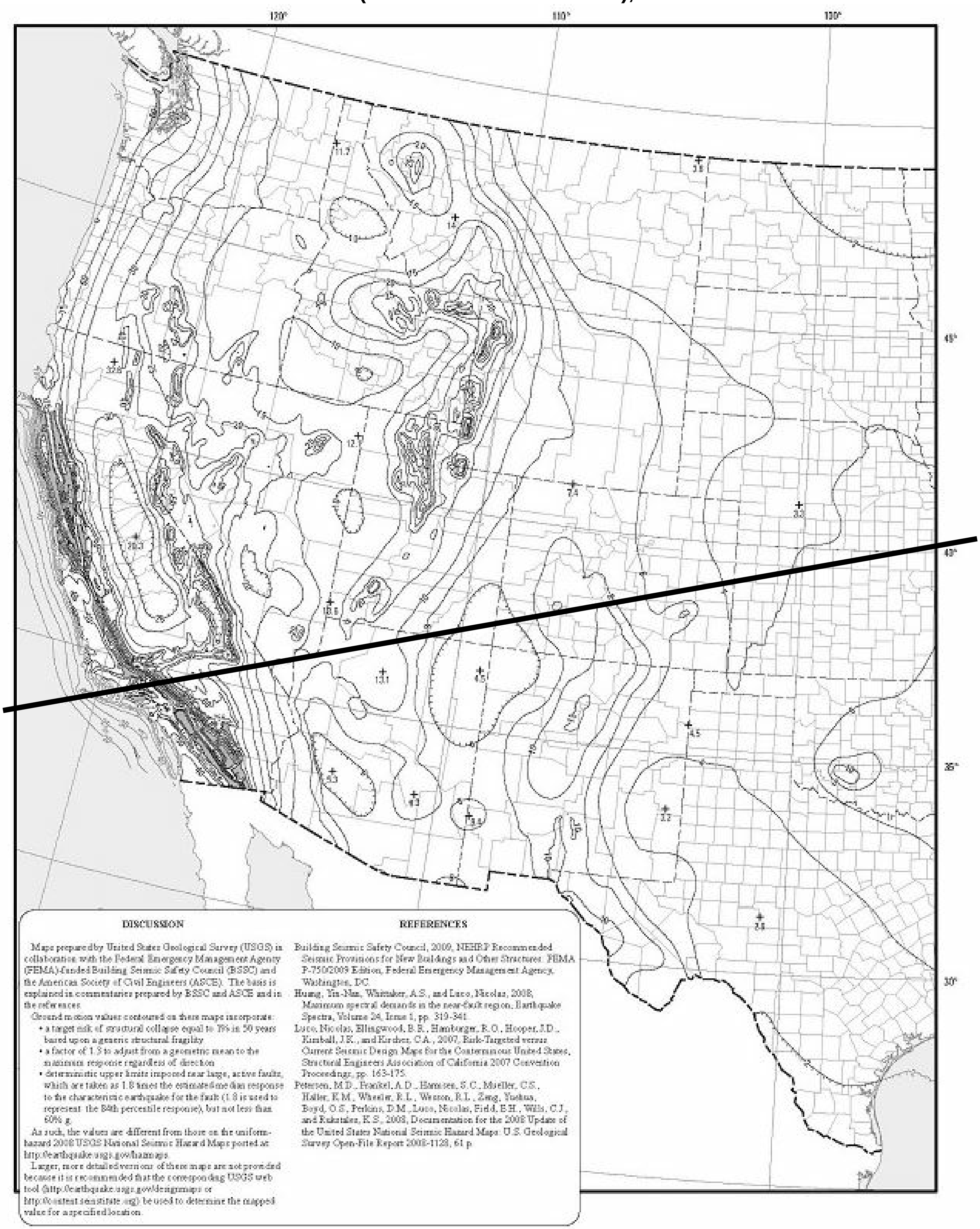
~~1613.3.1~~ (1)

~~RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B~~



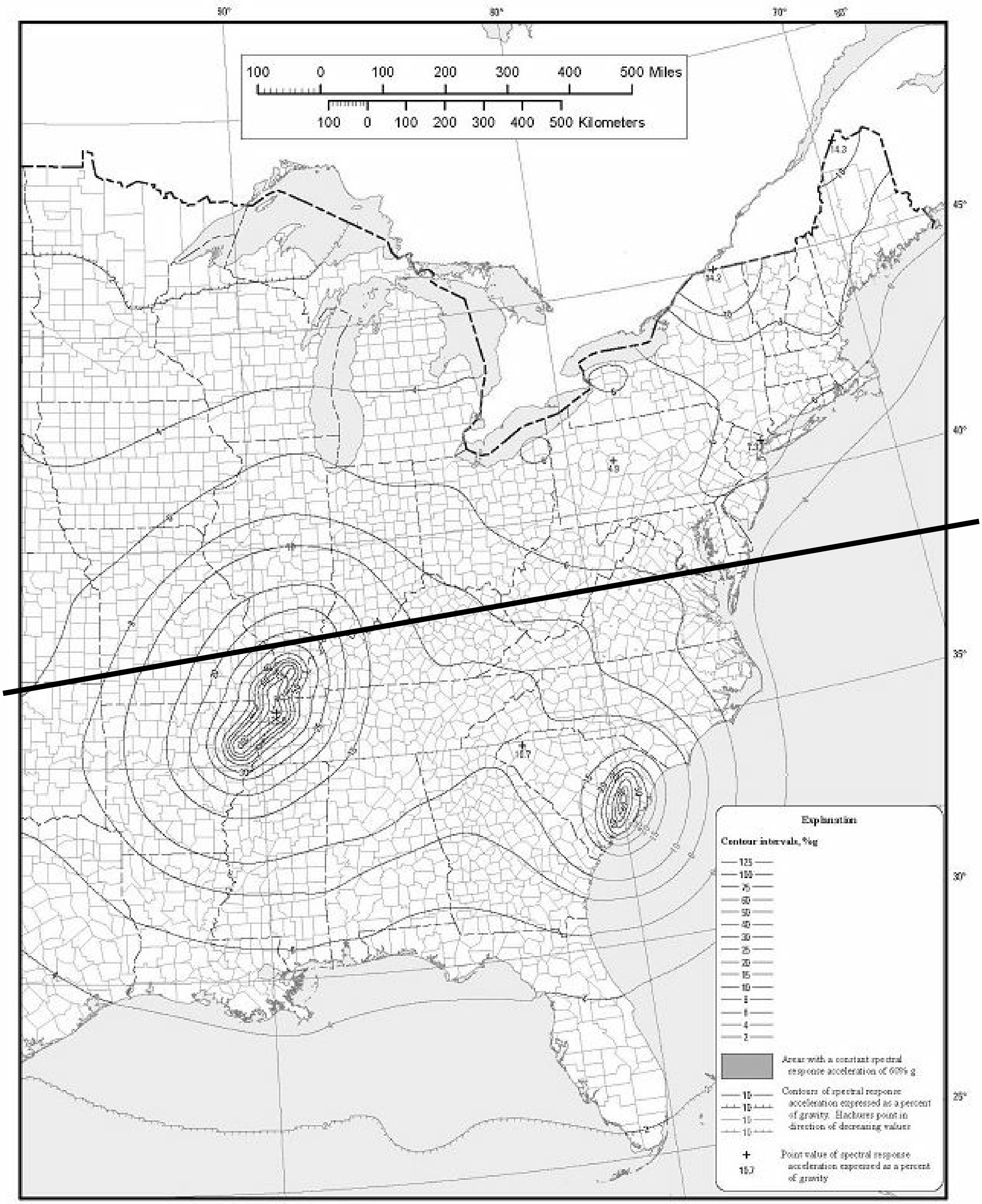
~~1613.3.1~~ (2)

~~RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE$_R$) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B~~



~~1613.3.1~~ (2)

~~RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B~~



~~**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 shall be used unless the building official or geotechnical data determines Site Class E or F soils are present at the site.~~

~~**1613.3.3 Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters.** The maximum considered earthquake spectral response acceleration for short periods,~~ $\cancel{S_{MS}}$~~, and at 1-second period,~~ $\cancel{S_{M1}}$~~, adjusted for *site class* effects shall be determined by Equations 16-37 and 16-38, respectively:~~

$\cancel{S_{MS} = F_a S_S}$ ~~**(Equation 16-37)**~~

$\cancel{S_{M1} = F_v S_1}$ ~~**(Equation 16-38)**~~

~~where:~~

$\cancel{F_a}$	~~=~~	~~Site coefficient defined in Table 1613.3.3(1).~~
$\cancel{F_v}$	~~=~~	~~Site coefficient defined in Table 1613.3.3(2).~~
$\cancel{S_S}$	~~=~~	~~The mapped spectral accelerations for short periods as determined in Section 1613.3.1.~~
$\cancel{S_1}$	~~=~~	~~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_V~~[a]

~~SITE CLASS~~	~~MAPPED SPECTRAL RESPONSE ACCELERATION AT 1-SECOND PERIOD~~				
	$\cancel{S_1 \le 0.1}$	$\cancel{S_1 = 0.2}$	$\cancel{S_1 = 0.3}$	$\cancel{S_1 = 0.4}$	$\cancel{S_1 \ge 0.5}$
~~A~~	~~0.8~~	~~0.8~~	~~0.8~~	~~0.8~~	~~0.8~~
~~B~~	~~1.0~~	~~1.0~~	~~1.0~~	~~1.0~~	~~1.0~~
~~C~~	~~1.7~~	~~1.6~~	~~1.5~~	~~1.4~~	~~1.3~~
~~D~~	~~2.4~~	~~2.0~~	~~1.8~~	~~1.6~~	~~1.5~~
~~E~~	~~3.5~~	~~3.2~~	~~2.8~~	~~2.4~~	~~2.4~~
~~F~~	~~Note b~~	~~Note b~~	~~Note b~~	~~Note b~~	~~Note b~~

~~a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period,~~ $\cancel{S_1}$~~.~~

~~b. 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]~~

	~~MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIOD~~

~~SITE CLASS~~					
	~~$S_S \le 0.25$~~	~~$S_S = 0.50$~~	~~$S_S = 0.75$~~	~~$S_S = 1.00$~~	~~$S_S \ge 1.25$~~
~~A~~	~~0.8~~	~~0.8~~	~~0.8~~	~~0.8~~	~~0.8~~
~~B~~	~~1.0~~	~~1.0~~	~~1.0~~	~~1.0~~	~~1.0~~
~~C~~	~~1.2~~	~~1.2~~	~~1.1~~	~~1.0~~	~~1.0~~
~~D~~	~~1.6~~	~~1.4~~	~~1.2~~	~~1.1~~	~~1.0~~
~~E~~	~~2.5~~	~~1.7~~	~~1.2~~	~~0.9~~	~~0.9~~
~~F~~	~~Note b~~	~~Note b~~	~~Note b~~	~~Note b~~	~~Note b~~

~~a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, S_S.~~

~~b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.~~

~~**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:~~

$$\cancel{V_{asd} = V_{ult}\sqrt{0.6}}$$ ~~**(Equation 16-39)**~~

$$\cancel{S_{DS} = \frac{2}{3}S_{MS}}$$ ~~**(Equation 16-40)**~~

~~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.~~

Revise as follows:

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_{DS} and S_{D1}, determined~~ in accordance with ~~Section 1613.3.4~~ ASCE 7 Chapter 11 or ~~the site-specific procedures~~ Section 12.14 as applicable.
Exception: Where a value of ~~ASCE 7. Each~~ seismic desgin category has been establised by the *building* ~~*and structure*~~ *official*, that value 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*~~ used.

TABLE 1613.3.5 (2)
~~SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION~~

~~VALUE OF S_{D1}~~	~~RISK CATEGORY~~		
	~~I or II~~	~~III~~	~~IV~~
~~S_{D1}~~	~~A~~	~~A~~	~~A~~
~~0.067g ≤ S_{D1}~~	~~B~~	~~B~~	~~C~~
~~0.133g ≤ S_{D1}~~	~~C~~	~~C~~	~~D~~
~~0.20g ≤ S_{D1}~~	~~D~~	~~D~~	~~D~~

TABLE 1613.3.5 (1)
~~SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD (0.2 second) RESPONSE ACCELERATION~~

~~VALUE OF S_{DS}~~	~~RISK CATEGORY~~		
	~~I or II~~	~~III~~	~~IV~~
~~S_{DS}~~	~~A~~	~~A~~	~~A~~
~~0.167g ≤ S_{DS}~~	~~B~~	~~B~~	~~C~~
~~0.33g ≤ S_{DS}~~	~~C~~	~~C~~	~~D~~
~~0.50g ≤S_{DS}~~	~~D~~	~~D~~	~~D~~

Delete without substitution:

~~**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_a, 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.~~

~~**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 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.~~

Reason: Many years ago, legacy building codes specified seismic loading using simple procedures through reference to a seismic zone map; assignment of each structure to one of five possible types; computation of a base shear coefficient using a single formula and criteria for distribution of the base shear up the structures height. Starting approximately in 1990, these design procedures became increasing more complex, w ith zonation maps abandoned in favor of spectral acceleration contour maps; five basic structures systems replaced w ith nearly 100 different types, each distinguished by different detailing requirements; and simple static analysis using a base shear equation replaced w ith a variety of linear and nonlinear dynamic methods. Early in this process, the system-specific detailing requirements became too voluminous to place w ith thin the body of the building code and instead the code referred to industry standards including ACI 318, AISC 341, TMS 502, and others for the needed requirements. How ever, the seismic maps, analysis procedures and classifications of seismic design category remained w ithin the body of the code. In 2006, most of this material w as removed from the code, leaving only the maps, and a portion of the procedures used to determine Seismic Design Category in the code. Other requirements necessary to compute seismic loading or even to determine Seismic Design Category w ere reference ed to ASCE 7, w here they have resided and have been maintained since. The maps that have continued to reside in the building code have not been useful because the scale is too small, and in regions of the country w here seismic design is significant, the contours are too closely spaced to allow determination of values from the maps. In fact, nearly all projects determine seismic ground motion values using w eb applications maintained by the USGS and w ithout direct reference to the maps. Furthermore, the materials pertaining to Seismic Design Category are incomplete and cannot by themselves be used w ithout reference to ASCE 7.
This proposal removed the now vestigial maps from the code as they are not useful in their present form. Additionally, the current maps are now obsolete because the USGS has developed a new series of maps that are referenced by ASCE 7-16. This proposal also removed the portions of the procedure used to determine Seismic Design Category because it is not possible ot use this portion of the procedure w ithout cross reference to ASCE 7, w here the information already resides and is maintained and updated as appropriate.

Complementing the publication of the 2016 edition of *ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, the seismic maps and seismic Design Category assignment tables w ill be made available separately from purchasing the full standard so that building officials and others w ill still have access to this specific data.

Cost Impact: Will not increase the cost of construction
The proposed changes w ill not impact the cost of construction. This proposal is a re-organization of the pointers in the IBC to the earthquake loads in the referenced loading standard ASCE 7. *ASCE 7 Minimum Design Loads and Associated Criteria for Buildings and Other Structures* w ill 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).

S115-16 : 1613.1-HAMBURGER12131

S116-16

IBC: 1613.3.

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

2015 International Building Code

Revise as follows:

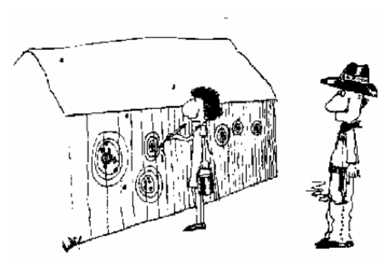
1613.3 ~~Seismic ground motion~~ Earthquake base shear coefficient values. ~~Seismic ground motion~~ Earthquake base shear coefficient values shall be determined in accordance with this section.

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 dow ngrading the *lateral design strength* or connection detailing requirements based upon a hazard model, w hich is largely a *numerical creation* heavily dependent on the perceived likelihoods of earthquake occurrences, as a con trolling 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 w ith a given magnitude M occurs, it causes a specific ground shaking that certainly does not take into account w hether 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, w hen considering tw o sites prone to earthquakes w ith the same magnitude M, given that all the remaining conditions are the same, the site w here large earthquakes are more sporadic appears naturally preferable for new settlements. Nevertheless parameters in seismic design must be equal at the tw o 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, w e favor building to the same standard to promote public safety [and community resilience], since there are no evidences that w hat happened in the past cannot repeat in the near future, or that it w ill occur only after a very long time."

- Peresan and Panza (2012)



"The Texas sharpshooter fallacy takes its name from a gunman w ho shoots at a side of a barn, only later to draw targets around a cluster of points that w ere 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://w w w .investopedia.com/terms/t/texas-sharpshooter-fallacy.asp

Bibliography:

FORUM: Improving Earthquake Hazard Assessments in Italy: An Alternative to "Texas Sharpshooting."
Peresan, A. and G.F. Panza. (2012) FORUM: Improving Earthquake Hazard Assessments in Italy: An Alternative to "Texas Sharpshooting."
Eos Vol. 93, No. 51 18 December 2012
http://onlinelibrary.wiley.com/doi/10.1029/2012EO510009/epdf

"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 Eartrhquake
http://earthquake.usgs.gov/earthquakes/eqinthenews/2011/se082311a/

The Mw 5.8 Virginia Earthquake of August 23, 2011
http://www.eqclearinghouse.org/2011-08-23-virginia/files/2011/12/EERI-GEER-DRC-Virginia-eq-report.pdf

Residential and Building Damage near epicenter: M 5.8 Mineral, Virginia EQ MMI VII – VIII
http://www.eqclearinghouse.org/2011-08-23-virginia/2011/09/02/residential-business-damage/

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 2008https://www.google.com/search?q=wells+nevada+earthquake+2008&biw=1280&bih=899&tbm=isch&tbo=u&source=univ&sa=X&ved=0ahUKEwjO7KmN

https://disastersafety.org/ibhs-risks-earthquake/

Cost Breakdown of Nonstructural Building Elements
Taghavi, S. and Miranda, E. (2003). Cost Breakdown of Nonstructural Building Elements, PEER Report 2003/05, Pacific 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 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://www.earthquakespectra.org/doi/abs/10.1193/1.4000032

Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Holliday, L. and Kang, T. (2014). "Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions." J. Perform. Constr. Facil., 10.1061/(ASCE)CF.1943-5509.0000630, 04014141.

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
California Seismic Safety Commission, 2005 Edition, 49 p.
http://www.disclosuresource.com/downloads/earthquake.pdf

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
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
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
Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee, 2013, 249 p. 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}}

S116-16 : 1613.3-BELA11853

S117-16

IBC: 1613.3.1.

Proponent : Zhenming Wang, Kentucky Geological Survey, University of Kentucky, Lexington, KY 40506, representing Kentucky Geological Survey (zmwang@uky.edu)

2015 International Building Code

1613.3.1 Mapped acceleration parameters. The parameters S_S and S_1 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). Where S_1 is less than or equal to 0.04 and S_S is less than or equal to 0.15, the structure is permitted to be assigned *Seismic Design Category* A.

Delete and substitute as follows:

FIGURE 1613.3.1 (1)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

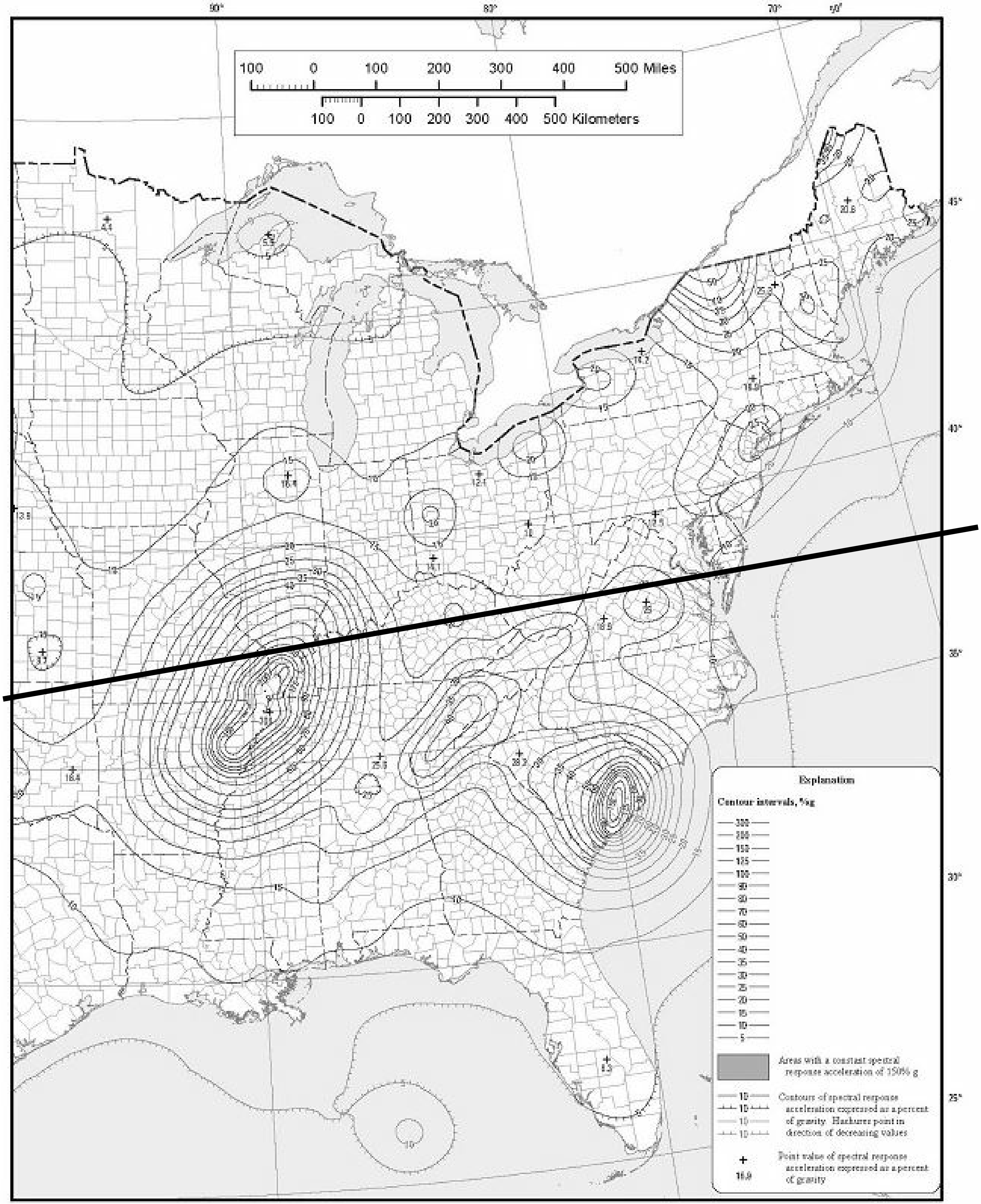


FIGURE 1613.3.1 (1)

RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

(Existing code change figure not shown for clarity)

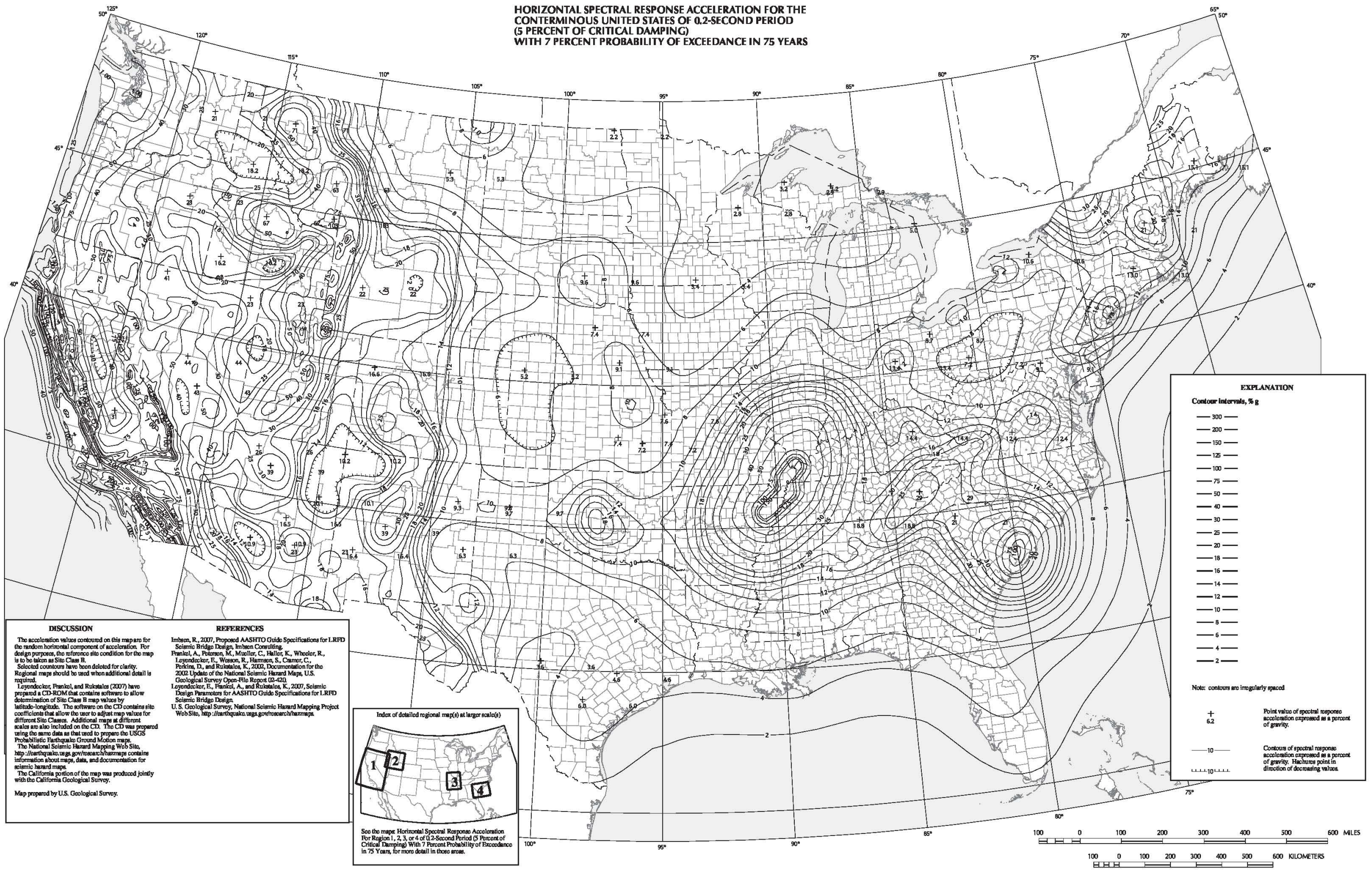


FIGURE 1613.3.1 (2)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

(Existing code change figure not shown for clarity)

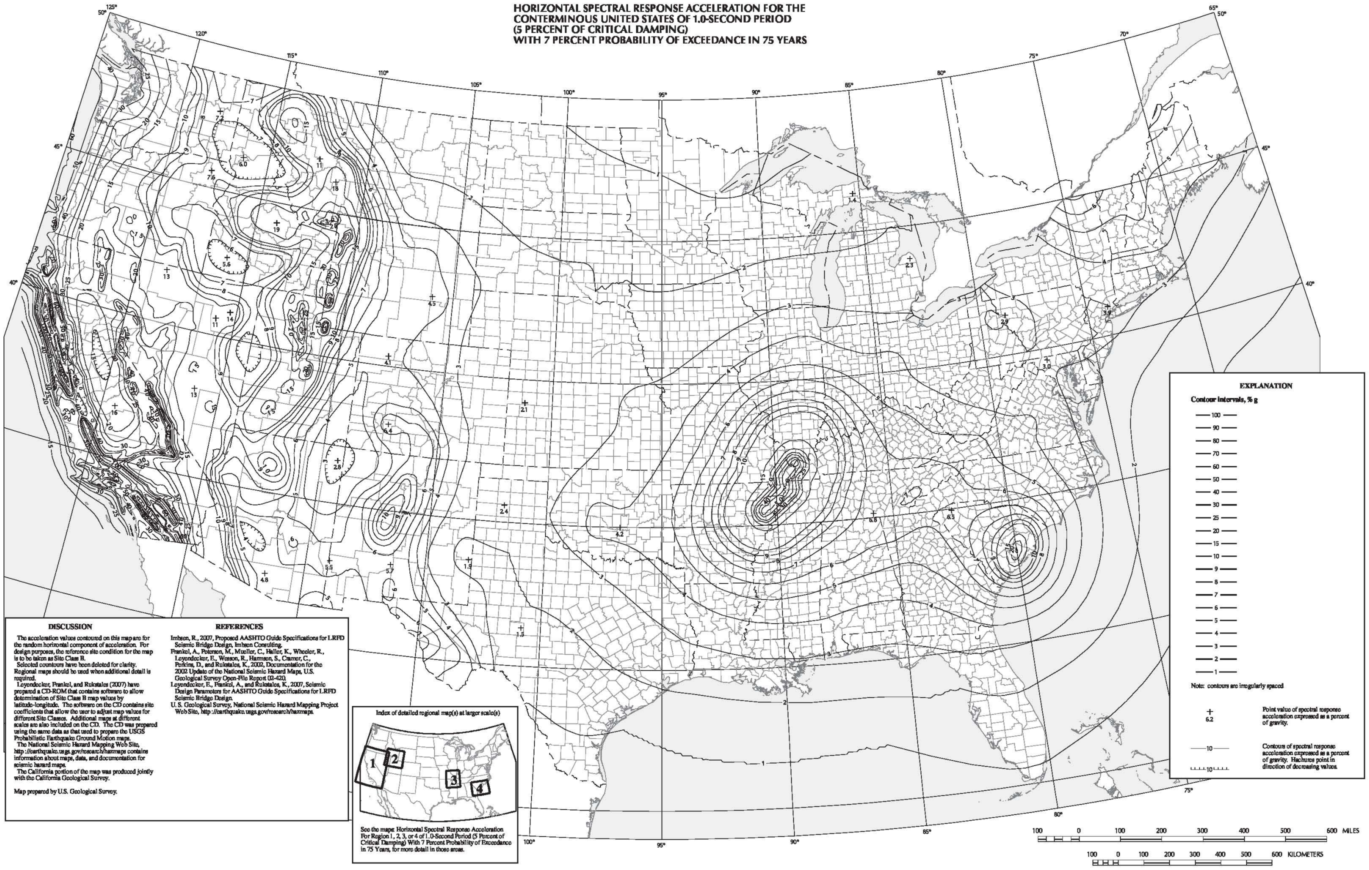
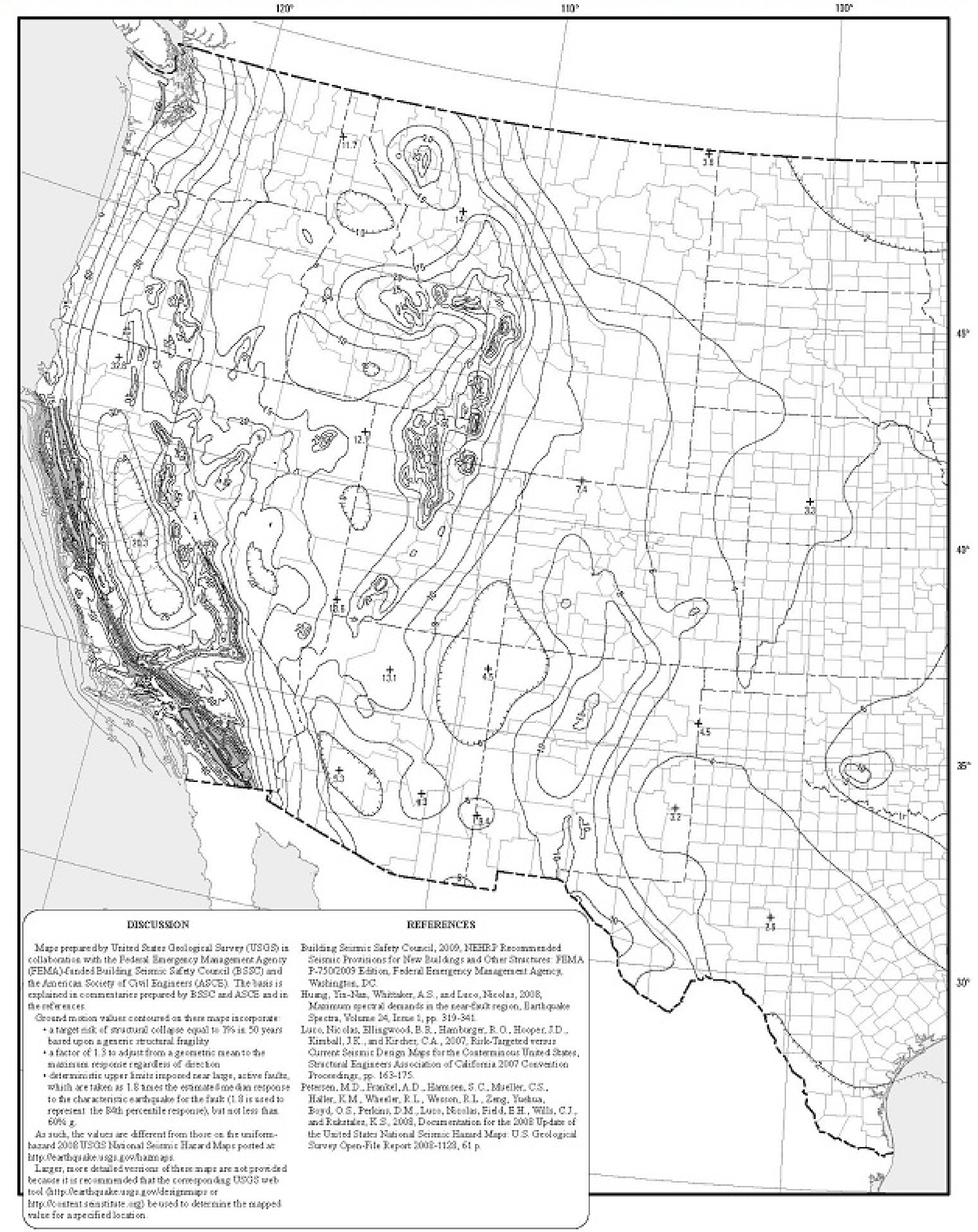


FIGURE 1613.3.1 (2)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

(Existing code change figure not shown for clarity)



Reason: the Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Response Accelerations of 0.2s and 1.0s in the central US are much higher than those in California; the highest 0.2s and 1.0s response

accelerations are 3.06g and 1.25g, respectively, in the central US, w hereas the highest response accelerations are 2.00g and 1.00g, respectively, in California. In other w ords, the seismic risk (i.e., the chance that the structure w ill experience partial or total collapse as a result of the most intense earthquake ground motion [MCE_R]) is higher the central US than anyw here in California. This higher seismic risk in the central US is not consistent w ith sciences (e.g., Wang and Cobb, 2012) and has caused intensive debate and discussion among scientists and engineers (e.g., AHERP, 2011; IEPNMSZEH, 2011; Beavers and Uddin, 2014; Wang, 2014).
Replacing Figures 613.3.1(1) and 613.3.1(2) w ith Figures 3.4.1-3 and 3.4.1-4 of AASHTO-2009.

Bibliography: Advisory Committee on Earthquake Hazards Reduction (ACEHR), 2011, National Earthquake Hazards Reduction Program (NEHRP) bicentennial statement, February 11, 2011.
Beavers J, Uddin N (eds) (2014) Seismic hazard design issues in the central United States. In: ASCE Council on Disaster Risk Management Monograph 7, chapter 9

Independent Expert Panel on New Madrid Seismic Zone Earthquake Hazards, 2011, Report: 26 p.

Wang Z (2014) Seismic design in w estern Kentucky: issues and alternatives. In: Beavers J, Uddin N (eds) Seismic hazard design issues in the central United States. In: ASCE Council on Disaster Risk Management Monograph 7, Chapter 9, pp 111–141

Wang Z, Cobb JC (2012) A critique of probabilistic versus deterministic seismic hazard analysis w ith special reference to the New Madrid Seismic Zone. Geological Society of America Special Paper 493: 259-275, doi:10.1130/2012.2493(13)

Cost Impact: Will not increase the cost of construction
The changes w ill reduce the cost of construction and other related costs in the central United States.

S117-16 : 1613.3-WANG13096

S118-16

IBC: 1613.3.1, 1613.3.1(1) (New), 1613.3.1(2) (New).

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

2015 International Building Code

~~**FIGURE 1613.3.1 (1)**~~
~~**RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_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 (1)**~~
~~**RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_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_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 (2)**~~
~~**RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_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_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_R) GROUND MOTION RESPONSE ACCELERATIONS FOR ALASKA OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B**~~

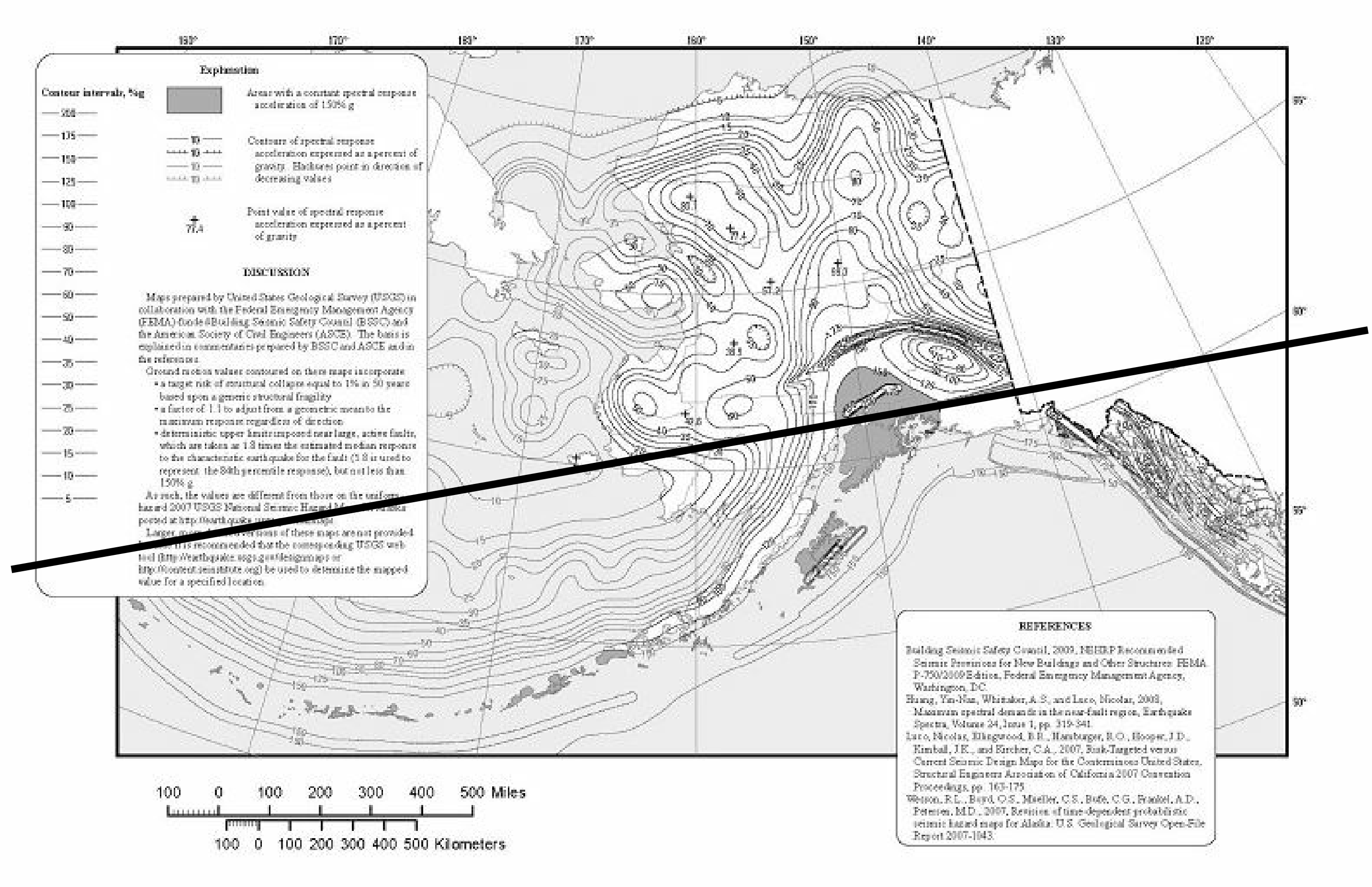
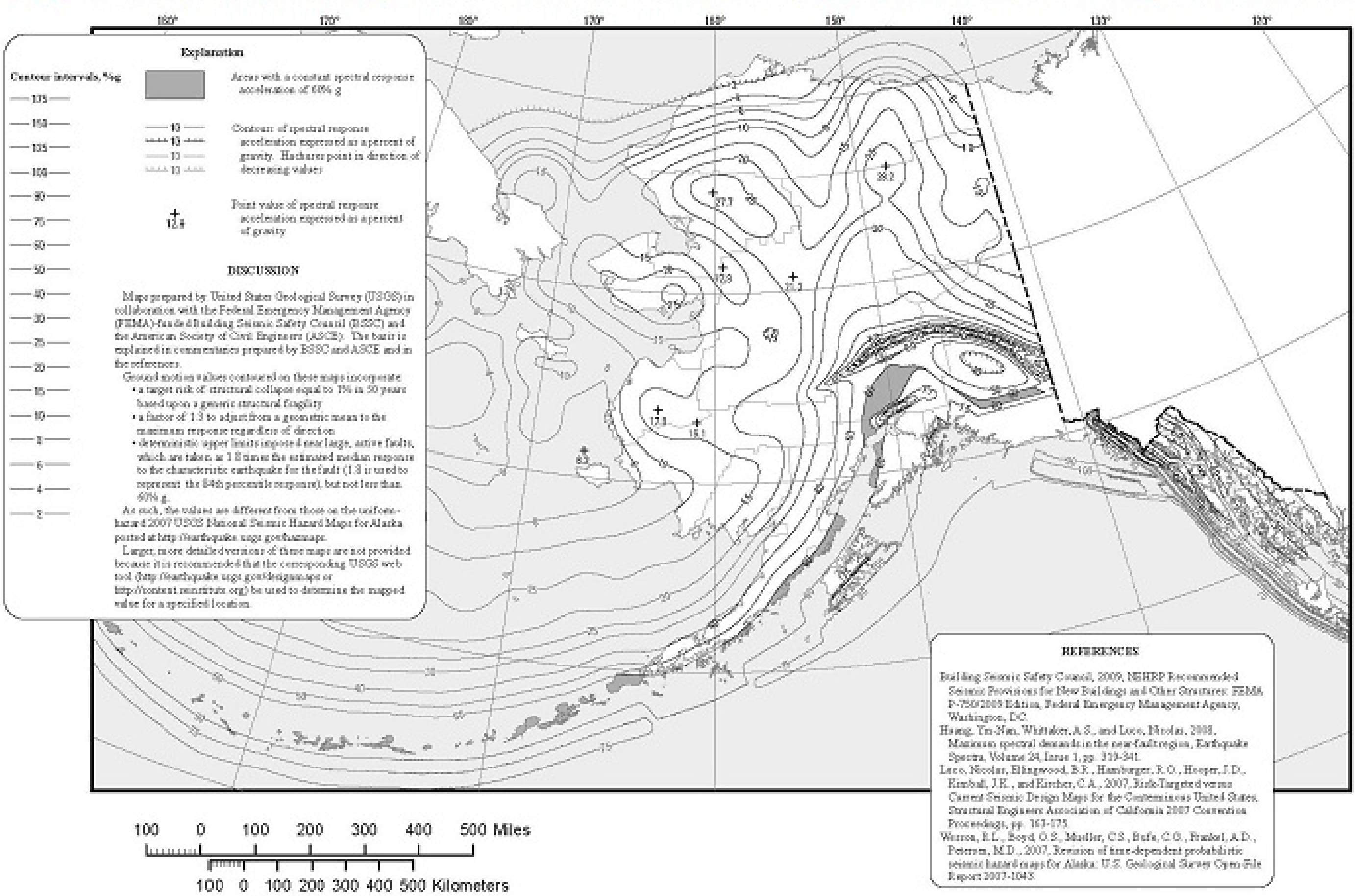


FIGURE 1613.3.1 (5)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR ALASKA OF 1.0-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

(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 ISLANDSOF 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~~

FIGURE 1613.3.1(1)
LATERAL DESIGN STRENGTH (BASE SHEAR) COEFFICIENT EXPRESSED AS SEISMIC ZONES 0-4 1994/1997 UBC

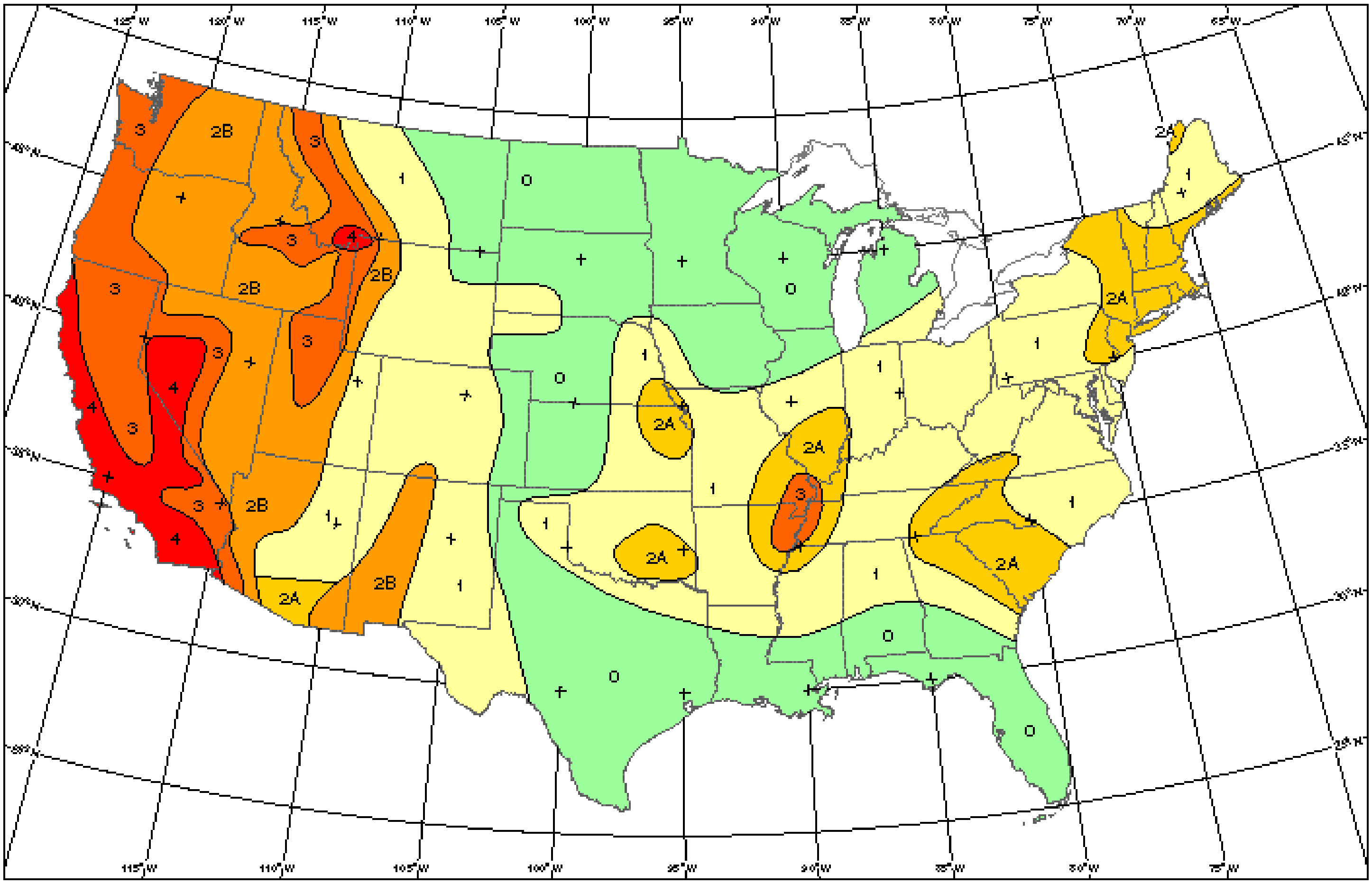
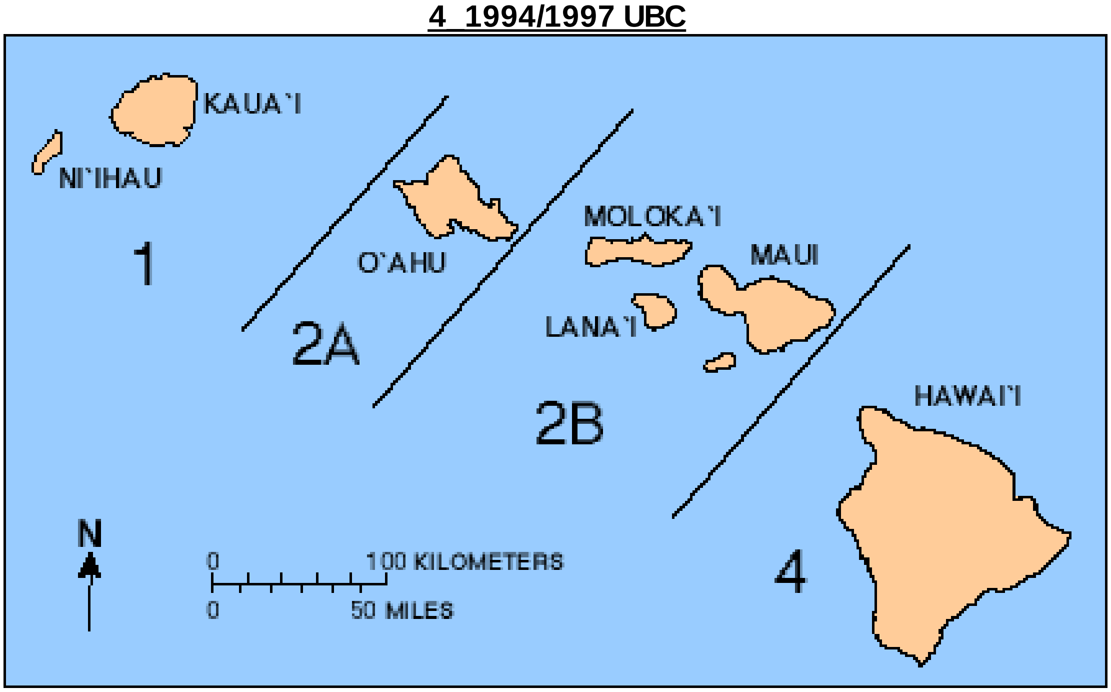


Figure A7. 1994 Uniform Building Code zone map. Zones are identified by the numbers from 0 to 4. Seismic zone factors are assigned to each zone; Zone 0 = 0, Zone1 = 0.075, Zone 2A = 0.15, Zone 2B = 0.20, Zone 3 = 0.3, and Zone 4 = 0.4. Each zone also has specific structural detailing requirements. After ICBO, 1994 (This map was redrawn from the original source, if differences occur, the original source should be used).

FIGURE <u>1613.3.1(2)</u>
<u>LATERAL DESIGN STRENGTH (BASE SHEAR) COEFFICIENT EXPRESSED AS SEISMIC ZONES 0-4 1994/1997 UBC</u>



Reason: "Any problem, no matter how complicated, can be made *simple*; if looked at in the right w ay."

- 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 w ay!"

- Prof. George W. Housner
Caltech, Earthquake Engineering – 1992

https://w w w .eeri.org/site/images/projects/oralhistory/housner.pdf

"I NEVER USE W0RDS IN A STORY, THAT I DON'T KNOW WHAT THEY MEAN!"

- Lou Costello

"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."*

NIST GCR-14-917-26 Cost Analysis and Benefit Studies for Earthquake-Resistant Construction in Memp[his, GTennessee, Dec. 2013 http://nehrp.gov/pdf/NIST%20GCR%2014-917-26_CostAnalysesandBenefitStudiesforEarthquake-ResistantConstructioninMemphisTennessee.pdf
http://nehrp.gov/pdf/NIST%20GCR%2014-917-26_CostAnalysesandBenefitStudiesforEarthquake-ResistantConstructioninMemphisTennessee.pdf

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 w hat to do about it - http://w w w .sciencedirect.com/science/article/pii/S0040195112003721
http://w w w .sciencedirect.com/science/article/pii/S0040195112003721
http://w eb.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, w e examine w hat w ent w rong 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*, w hose values are *chosen based on the mapmakers' preconceptions*. When these are incorrect, maps do poorly. This situation w ill improve at best slow ly, ow ing to our limited understanding of earthquake processes. How ever, because hazard mapping has become w idely accepted and used to make major decisions, w e suggest tw o changes to improve current practices. *First, the uncertainties in hazard map predictions should be assessed and clearly communicated to potential users*. Recognizing the uncertainties w ould 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, w hich is common and useful in similar fields, w ill show how w ell maps actually w ork and hopefully help produce measurable improvements. There are likely, how ever, limits on how w ell hazard maps can ever be made because of the intrinsic variability of earthquake processes." (Stein et. al. 2012)

PSHA: Is it Science?

PSHA: isit science?

Probabilistic seismic hazard analysis (PSHA) is beginning to be seen as unreliable. The problem w ith PSHA is that its data are inadequate and its logic is defective. Much more reliable, and more scientific, are deterministic procedures, especially w hen coupled w ith 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?

Why are the Standard Probabilistic Methods of Estimating Seismic Hazard and Risks Too Often Wrong?

Earthquake Hazard, Risk, and Disasters, Elsevier Inc., 2014, pp. 309-357. DOI: 10.1016/B978-0-12-394848-9.00012-2

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."] w ithin individual seismic zones. How ever, globally over the last decade such events keep occurring w here 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" w ithin individual seismic zones. How ever, globally over the last decade such events keep occurring w here PSHA predicted seismic hazard to be low . Systematic comparison of the observed ground shaking w ith the expected one reported by the Global Seismic Hazard Assessment Program (GSHAP) maps discloses gross underestimation w orldw ide. Several inconsistencies w ith 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 tw o to three orders of magnitude. The total death toll in 2000–2011 (w hich exceeds 700,000 people, including tsunami victims) calls for a critical reappraisal of GSHAP results, as w ell as of the underlying methods.

In this chapter, w e discuss the limits in the formulation and use of PSHA, addressing some theoretical and practical issues of seismic hazard assessment, w hich range from the overly simplified assumption that one could reduce the tensor problem of seismic-w ave 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, w hich 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 w ith some general considerations and constructive ideas tow ard improved seismic hazard and effective risk assessment. Specifically, w e 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

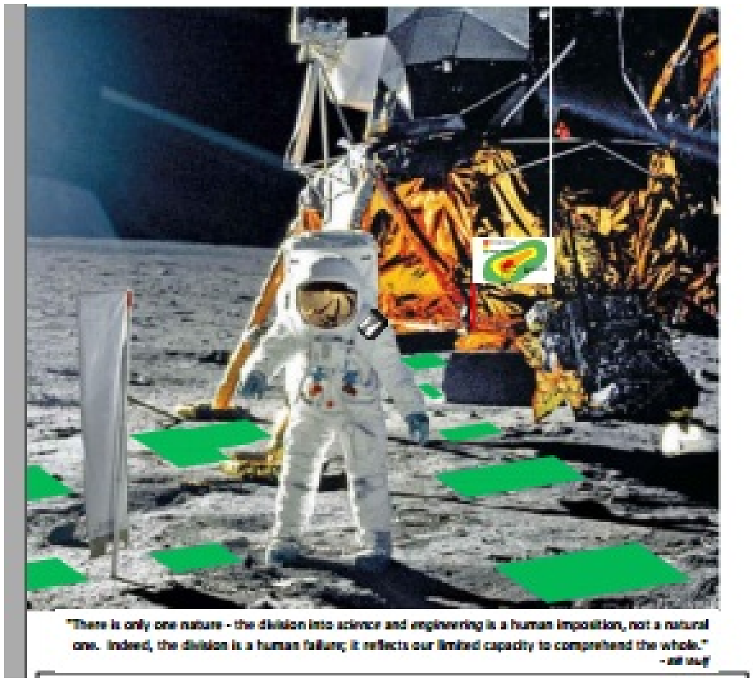
"The 'map' is w hat w e think resembles reality, and w e should use it as a guide in our thinking and actions. *One is well advised, when traveling to a new territory, to take a good map and then to heck the map with the actual territory during the journey*. This map must be subject to new objective scientific insights w ith due consideration of the potential imminence of the global changes. Our actions should reflect this view point."

- G.J. Wasserburg

LETTERS – Comment of "AGU Statement: Investigation of Scientists and OfficialsL'Aquila, Italy, Is Unfounded." Eos Vol. 91, No. 42, 19 Oct. 2010, p. 384.

http://onlinelibrary.w iley.com/enhanced/doi/10.1002/2013EF000225/

https:agu.confex.com/agu/spc2014/w ebprogram/Paper1558.html



"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 w hole,"

- Bill Wulf

Bela, J. (2014), Too generous to a fault? Is reliable earthquake safety a lost art? Errors in expected human losses due to incorrect seismic hazard estimates. Earth's Future, 2: 569-578, doi:10.1002/2013EF000225

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 incorrecdt eimoic haszard estimates - http://onlinelibrary.w iley.com/enhanced/doi/10.1002/2013EF000225/
http://onlinelibrary.w iley.com/enhanced/doi/10.1002/2013EF000225/

Errors in Seismic Hazard Assessment are Creating Huge Human Losses
https:agu.confex.com/agu/spc2014/w ebprogram/Paper1558.html

"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," w hich w as then also assumed to be the "risk to w hich the populations w ere exposed." But time-after-time-after-time the actual earthquakes that occurred w ere not only "surprises" (many times larger than those implied on the maps), but they w ere 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), w e need to ask not only: "Is reliable earthquake safety a lost art?" but also: "Who and w hat w ere the **'Raiders of the Lost Art?'** "

The Problem with Probabilistic Methods
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 w ill 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)."

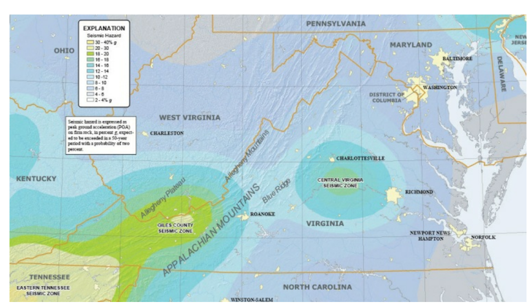
Seismic Hazard Assessment: Issues and Alternatives

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.

"FEMA here. No, we haven't had any substantiated reports of earthquakes."

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.
[http://www.nibs.org/client/assets/files/bssc/P749/P-749_Chapter5.pdf]
Earthquake-Resistant Design Concepts: An Introduction to the NEHRP Recommended Seismic Provisions for New Buildings and Other Structures - http://www.fema.gov/media-library-data/20130726-1759-25045-5477/fema_p_749.pdf
http://www.fema.gov/media-library-data/20130726-1759-25045-5477/fema_p_749.pdf Chp. 5 - Seismic Design Categories

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.

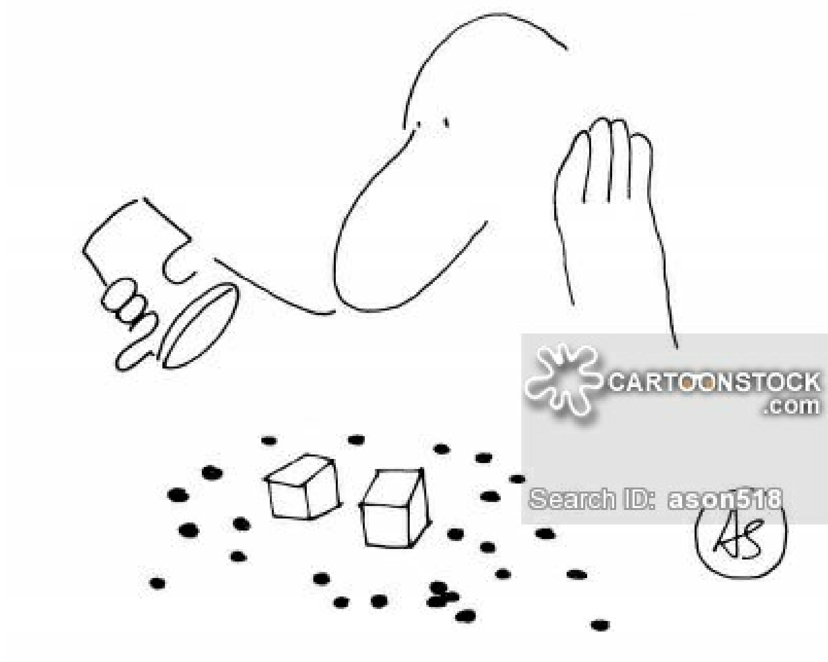
These numbers translate to about a 30% decline in design strength (from a low number to an even lower number) in just the last decade! (for the SDS "Design Earthquake Spectral Response Acceleration Parameter"). A 33% increase in design strength used to be the difference between Seismic Zone 3 and Seismic 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=oN86d0CdgHQ

"We have nothing to fear but veneer itself!"



WHAT IS THE CHANCE OF AN EARTHQUAKE?

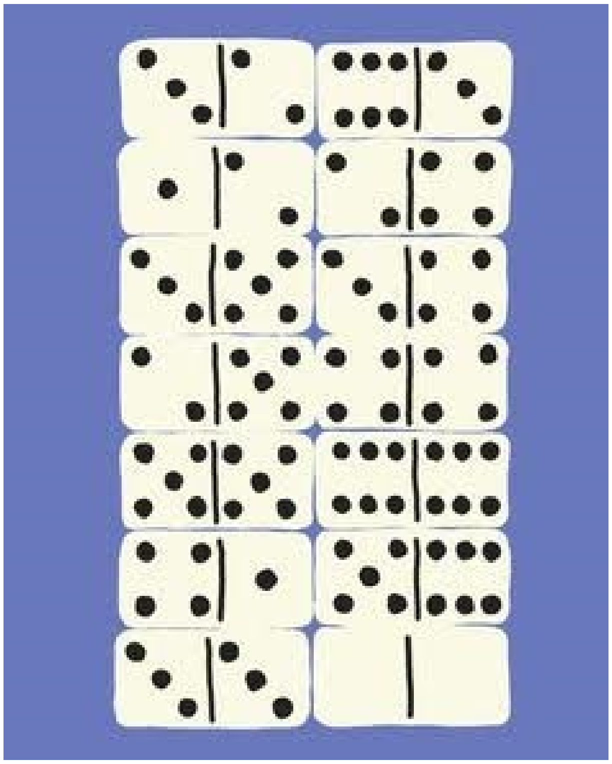
WHAT IS THE CHANCE OF AN EARTHQUAKE? - http://www.stat.berkeley.edu/~stark/Preprints/611.pdf

http://w w w .stat.berkeley.edu/~stark/Preprints/611.pdf

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 w ell as securing w ater heaters, bookcases, and other heavy objects. They should keep first aid supplies, w ater, and food on hand. They should largely ignore the USGS probability forecast.

ROBUST

bust!



DSHA and

NDSHA *psha*

Maps to the Future

During development of the **[1] 2015 NEHRP Provisions** the **[2] BSSC Provisions Update Committee (PUC)** considered **[3]** a proposal to adopt new maps developed by **[4] USGS**. **USGS** had produced the new maps using the **[5]** basic rules established previously by the **[6] Project 97** and **[7] Project 07** efforts, but incorporating updated databases on source activity rates and segmentation, and updated **[8] [GMPEs]** ground motion prediction equations. As would be anticipated, mapped values in some locations increased [*Bummer!]* and in others decreased [*Yeah!*] , with the amplitude of change generally falling under 20%, but sometimes reversing directional trends **[yo-yo-ing effect]** observed in recent prior map revisions [See "Take Me Home Seismic Loads" - below].

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 [09-28-2015], 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 eathquake hazard mapping is obviously as unsatsifactory as it is defective - in addressing public safety concerns from real earthquakes!*

Called Project 17 - Developing Next Generation Seismic Design Values Maps , 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 it 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."* Dissapointingly, 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.

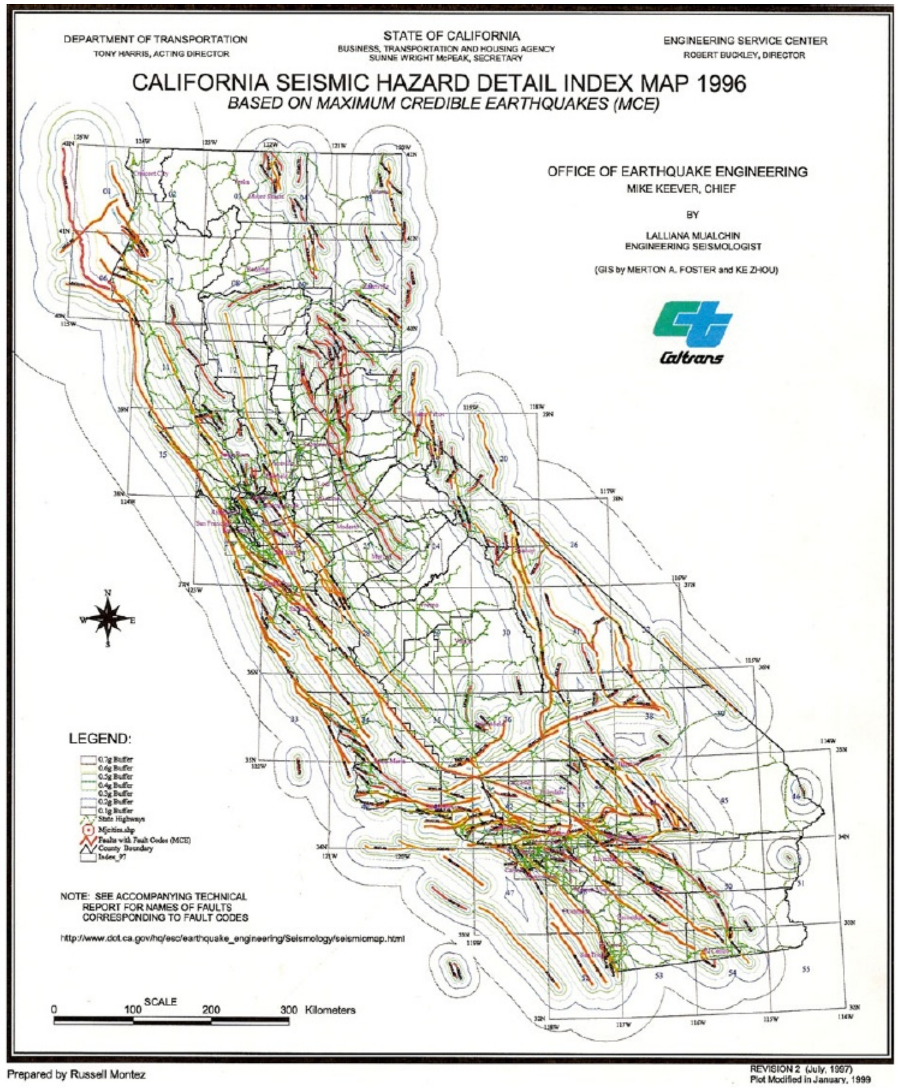
http://www.ce.memphis.edu/7137/PDFs/Cornell/1583.pdf

So this code change proposal is duly needed to restore the code format to a **Step 1** which 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.

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."



<u>History of Modern Earthquake Hazard Mapping and Assessment in California Using a Deterministic or Scenario Approach</u>

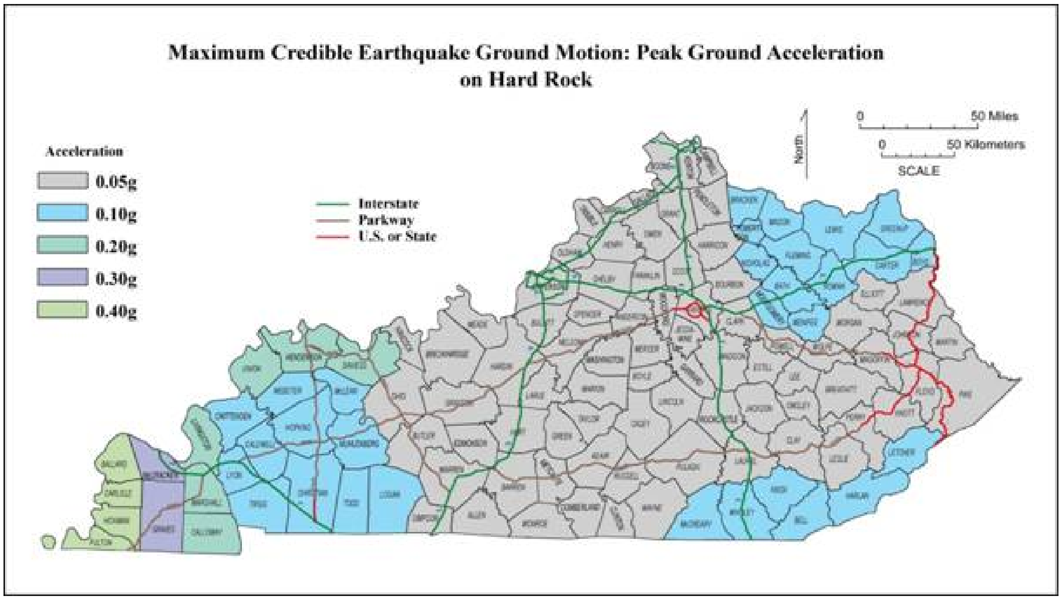
[History of Moderrn Earthquake Hazard Mapping and Assesment in California Using a Deterministic or Scenario Approach - http://www.academia.edu/7849458/History_of_Modern_Earthquake_Hazard_Mapping_and_Assessment_in](http://www.academia.edu/7849458/History_of_Modern_Earthquake_Hazard_Mapping_and_Assessment_in)

http://www.academia.edu/7849458/History_of_Modern_Earthquake_Hazard_Mapping_and_Assessment_in_California
https://www.researchgate.net/publication/226495072_History_of_Modern_Earthquake_Hazard_Mapping_and_Assess

<u>History of Modern Earthquake Hazard Mapping and Assessment in California Using Deterministic or Scenario Approach - presentation</u>

[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)

http://indico.ictp.it/event/a09145/session/51/contribution/37/material/0/0.pdf



Kentucky Geological Survey Challenges the USGS Seismic Hazard Map
https://w w w .youtube.com/w atch?v=zlHM9tUFT8g

Seismic Hazard Assessment for Kentucky
Seismic Hazard Assessment for Kentucky - http://w w w .uky.edu/KGS/geologichazards/research_assessment.htm
http://w w w .uky.edu/KGS/geologichazards/research_assessment.htm

"The future's *whatever* you make it to be . . . *make it a good one!*"

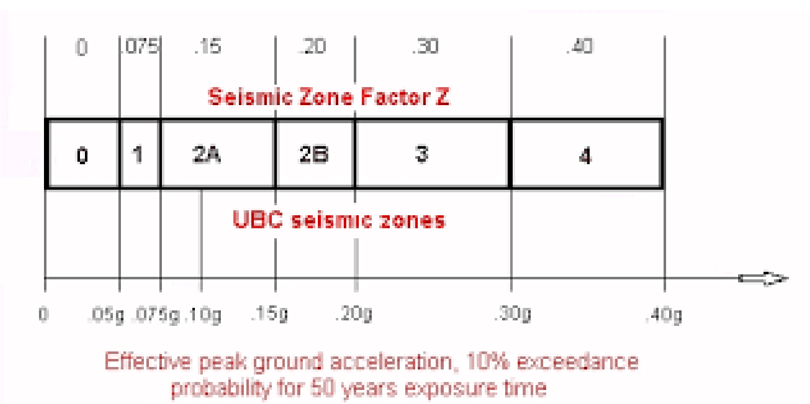
- Doc Emmet Brow n
Back to the Future

III

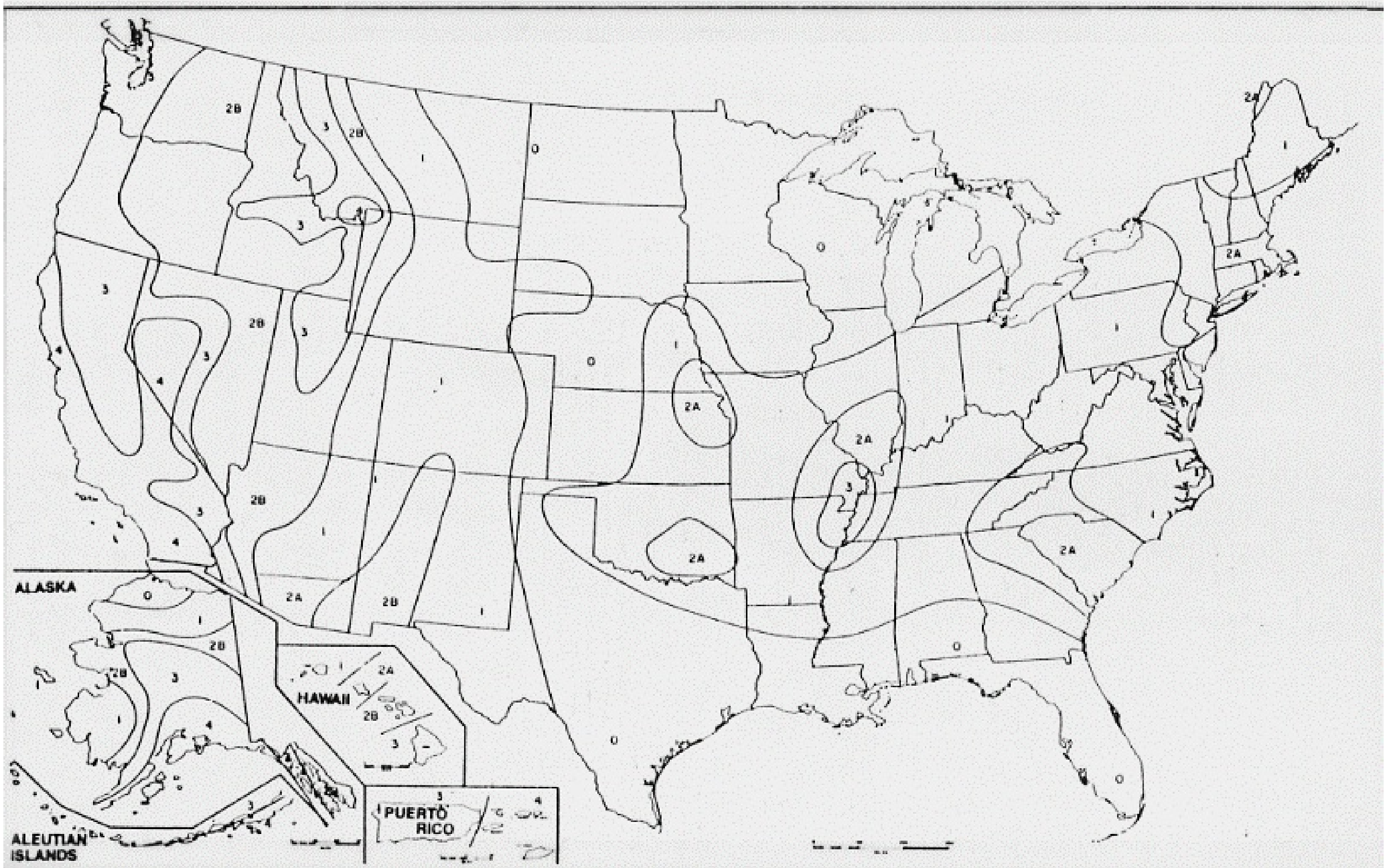
Or . . . *more of this?*

"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."

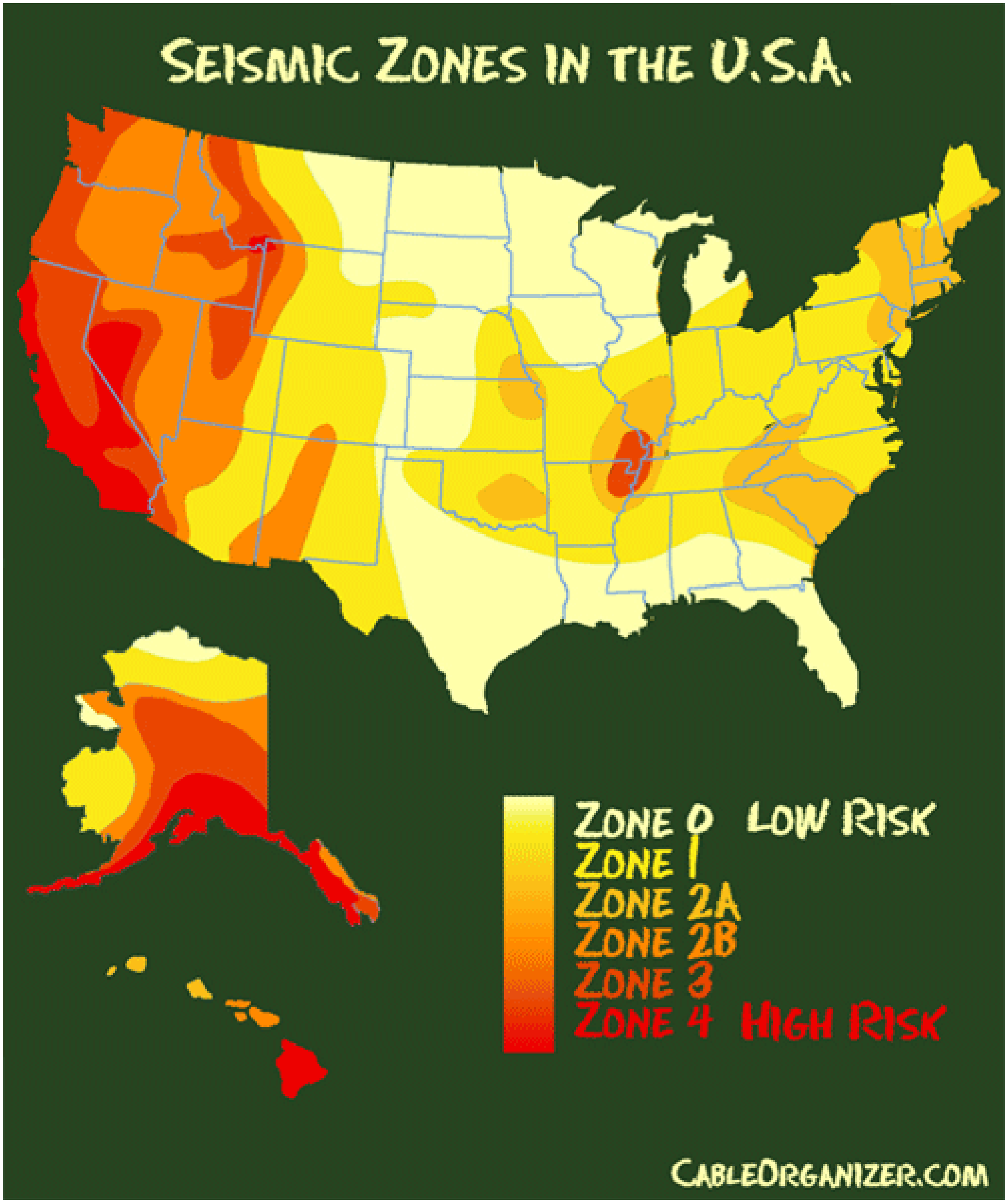
Lateral Design Strength (Base Shear) Coefficient expressed as Seismic Zones 0 - 4_Color



Relationship betw een *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 expresses as Seismic Zones 0-4_B/W



Lateral Design Strength (Base Shear) Coefficient expressed as Seismic Zones 0-4_Color

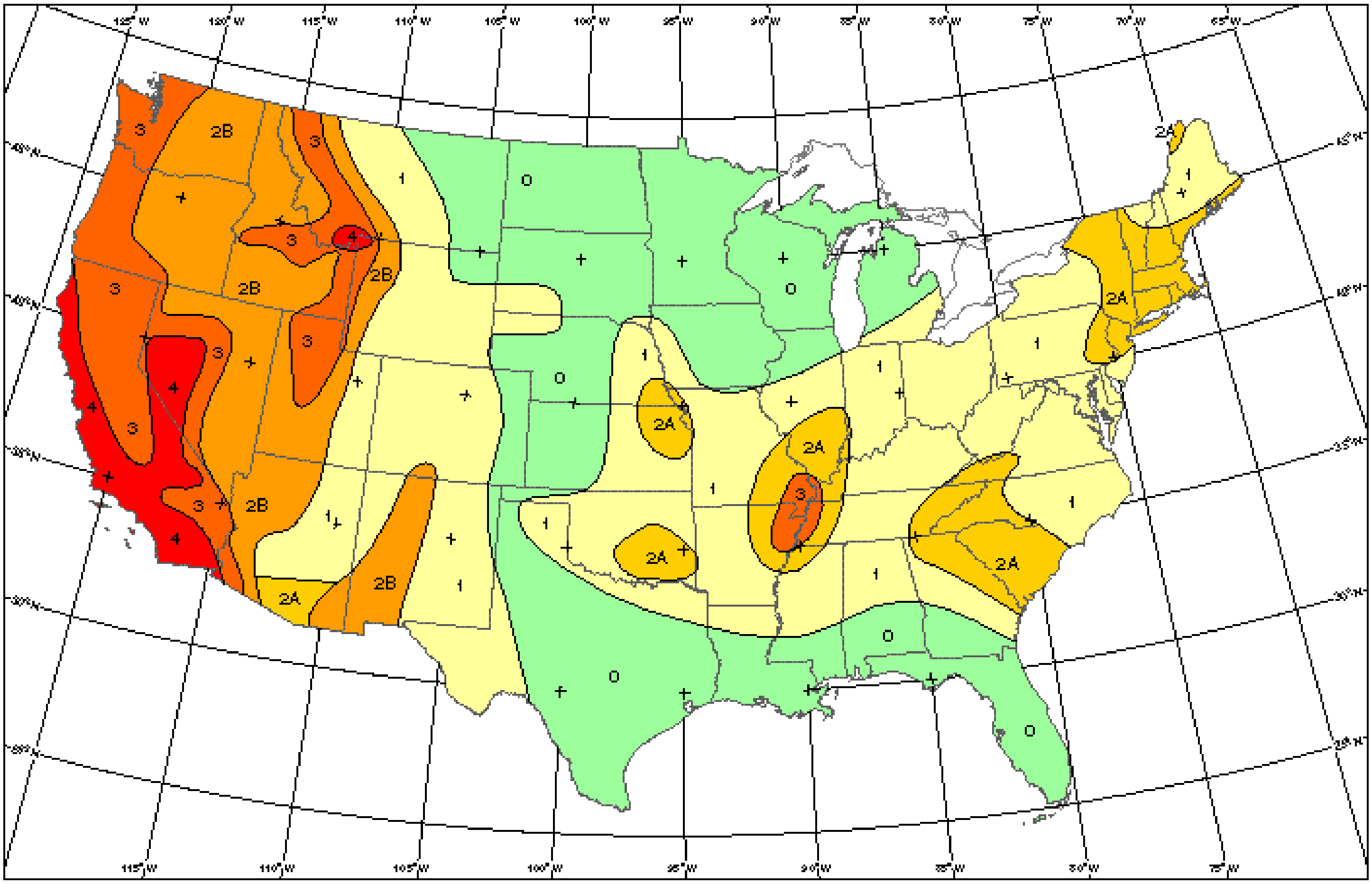
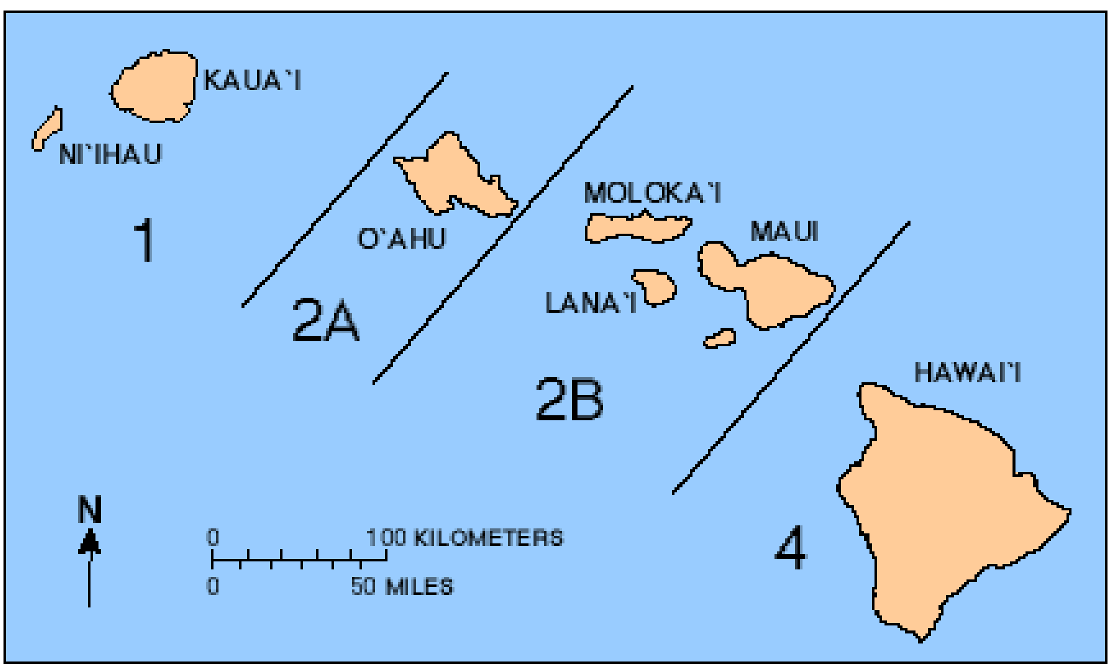
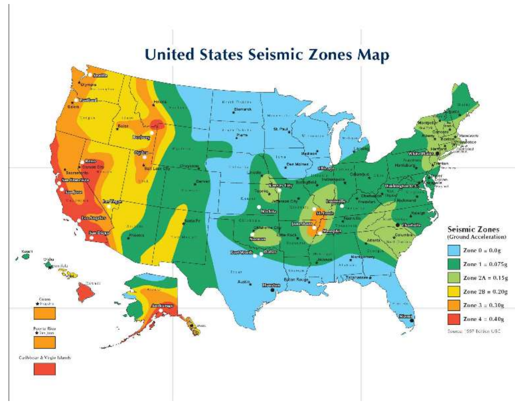


Figure A7. 1994 Uniform Building Code zone map. Zones are identified by the numbers from 0 to 4. Seismic zone factors are assigned to each zone; Zone 0 = 0, Zone1 = 0.075, Zone 2A = 0.15, Zone 2B = 0.20, Zone 3 = 0.3, and Zone 4 = 0.4. Each zone also has specific structural detailing requirements. After ICBO, 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_Color



Bibliography: <u>**S110-12 (2012 IBC) - deleting MCE_R Ground Motion Maps - restoring previous MCE Ground Motion Maps**</u>

[S110-12 (2012 IBC) - deleting MCEr Ground Motion Maps - restoring previous MCE Ground Motion Maps - http://www.bcd.oregon.gov/boards/bcsb/board_pack/2012/20120926/minute_posting_handout.pdf]
http://www.bcd.oregon.gov/boards/bcsb/board_pack/2012/20120926/minute_posting_handout.pdf

<u>**Earthquake-Resistant Design According to 1997 UBC**</u>

[Earthquake-Resistant Design According to 1997 UBC - http://site.iugaza.edu.ps/sshihada/files/2012/02/Handout-11-12.pdf]
http://site.iugaza.edu.ps/sshihada/files/2012/02/Handout-11-12.pdf

<u>**PSHA: is it science?**</u>
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https://www.researchgate.net/publication/238378491_PSHA_is_it_science ;
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[George W. Housner, EERI Oral History 1997]
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?

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"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 w ithstand 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."

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Wells, D.L., and K.J. Coppersmith (1994). New empirical relationships among magnitude, rupture length, rupture width, rupture area, and surface displacement, Bull. Seismol. Soc. Am. 84, 974–1002.

Wiggins, J.H. (1975). Procedure for Determining Acceptable Risk Ground Motion Design Criteria, J. H. Wiggins Company, Redondo Beach, California Technical Report No. 75-1229.

Wilder, Billy: (1906 - 2002). "If you're going to tell people the truth, be funny or they will kill you."

Wyss, M., A. Nekrasova, and V. Kossobokov (2012). Errors in expected human losses due to incorrect seismic hazard estimates, Nat Hazards, 62, 927-935. DOI: 10.1007/s11069-012-0125-5

Zacher, E.G. (1990). U. S. Earthquake Zoning Map--History of the Structural Engineers Association of California's (SEAOC's) involvement in the development of the U. S. earthquake zoning map. Proceedings of the 50th Regional Conference, Feb. 25–26, 1988, Los Angeles, California. Council on Tall Buildings and Urban Habitat Council Report 903.377, Lehigh University, Bethlehem, PA, 17–23.

"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 coeficients," 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 determinisitic 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 structuralo 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
Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee, 2013, 249 p. NEHRP Consultants Joint Venture A partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering.

In general, w here costs might be increased, cost premiums above requirements for w ind tend to fall w ithin a range of +1-3%. For cases w here seismic requirements w ould be now additional to w hat previous codes either applied/neglected/failed to enforce, estimates probably w ould fall w ithin the range of 0.25 - 1%.

Analysis: See RB18-16 for IRC coordination proposal

S118-16 : FIGURE 1613.3.1-BELA11922

S119-16

IBC: 1613.3.1.

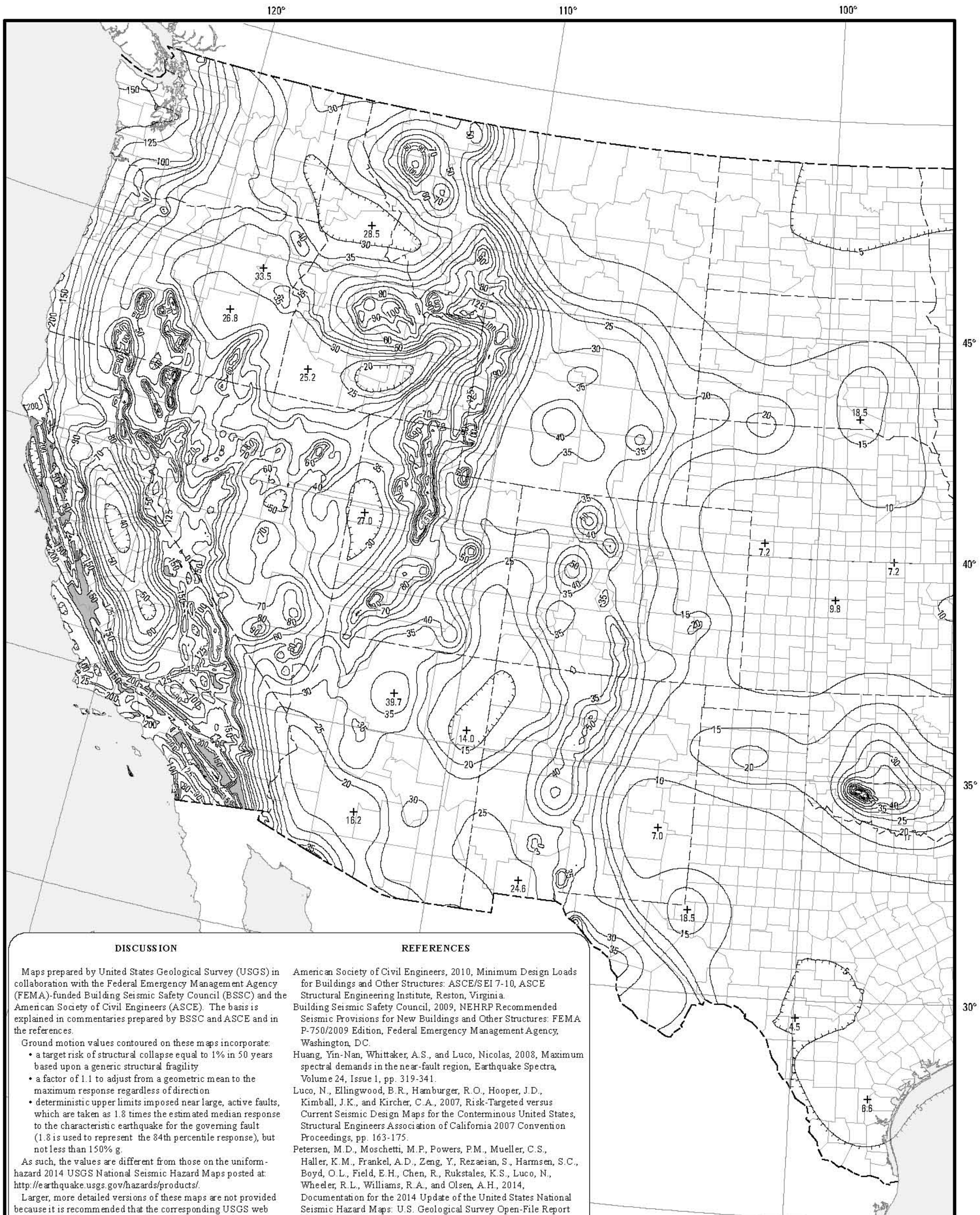
Proponent : Kelly Cobeen, Wiss Janney Elstner Associates, Inc., representing Federal Emergency Management Agency and National Institute of Building Sciences Building Seismic Safety Council's Code Resource Support Committee (kcobeen@wje.com)

2015 International Building Code

Revise as follows:

FIGURE ~~1613.3.1~~ 1613.3.1(1)-1 (1)
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

(Existing code change figure not shown for clarity)



DISCUSSION

Maps prepared by United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency (FEMA)-funded Building Seismic Safety Council (BSSC) and the American Society of Civil Engineers (ASCE). The basis is explained in commentaries prepared by BSSC and ASCE and in the references.

Ground motion values contoured on these maps incorporate:

- a target risk of structural collapse equal to 1% in 50 years based upon a generic structural fragility
- a factor of 1.1 to adjust from a geometric mean to the maximum response regardless of direction
- deterministic upper limits imposed near large, active faults, which are taken as 1.8 times the estimated median response to the characteristic earthquake for the governing fault (1.8 is used to represent the 84th percentile response), but not less than 150% g.

As such, the values are different from those on the uniform-hazard 2014 USGS National Seismic Hazard Maps posted at: http://earthquake.usgs.gov/hazards/products/.

Larger, more detailed versions of these maps are not provided because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/hazards/designmaps/ or http://content.seinstitute.org) be used to determine the mapped value for a specified location.

REFERENCES

American Society of Civil Engineers, 2010, Minimum Design Loads for Buildings and Other Structures: ASCE/SEI 7-10, ASCE Structural Engineering Institute, Reston, Virginia.

Building Seismic Safety Council, 2009, NEHRP Recommended Seismic Provisions for New Buildings and Other Structures: FEMA P-750/2009 Edition, Federal Emergency Management Agency, Washington, DC.

Huang, Yin-Nan, Whittaker, A.S., and Luco, Nicolas, 2008, Maximum spectral demands in the near-fault region, Earthquake Spectra, Volume 24, Issue 1, pp. 319-341.

Luco, N., Ellingwood, B.R., Hamburger, R.O., Hooper, J.D., Kimball, J.K., and Kircher, C.A., 2007, Risk-Targeted versus Current Seismic Design Maps for the Conterminous United States, Structural Engineers Association of California 2007 Convention Proceedings, pp. 163-175.

Petersen, M.D., Moschetti, M.P., Powers, P.M., Mueller, C.S., Haller, K.M., Frankel, A.D., Zeng, Y., Rezaeian, S., Harmsen, S.C., Boyd, O.L., Field, E.H., Chen, R., Rukstales, K.S., Luco, N., Wheeler, R.L., Williams, R.A., and Olsen, A.H., 2014, Documentation for the 2014 Update of the United States National Seismic Hazard Maps: U.S. Geological Survey Open-File Report 2014-1091, 224 p.

Figure 1613.3.1(1) Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Response Accelerations for the Conterminous United States of 0.2-Second Spectral Response Acceleration (5% of Critical Damping), Site Class B *(continued)*

FIGURE ~~1613.3.1~~ 1613.3.1(1)-2 (1)

RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

(Existing code change figure not shown for clarity)

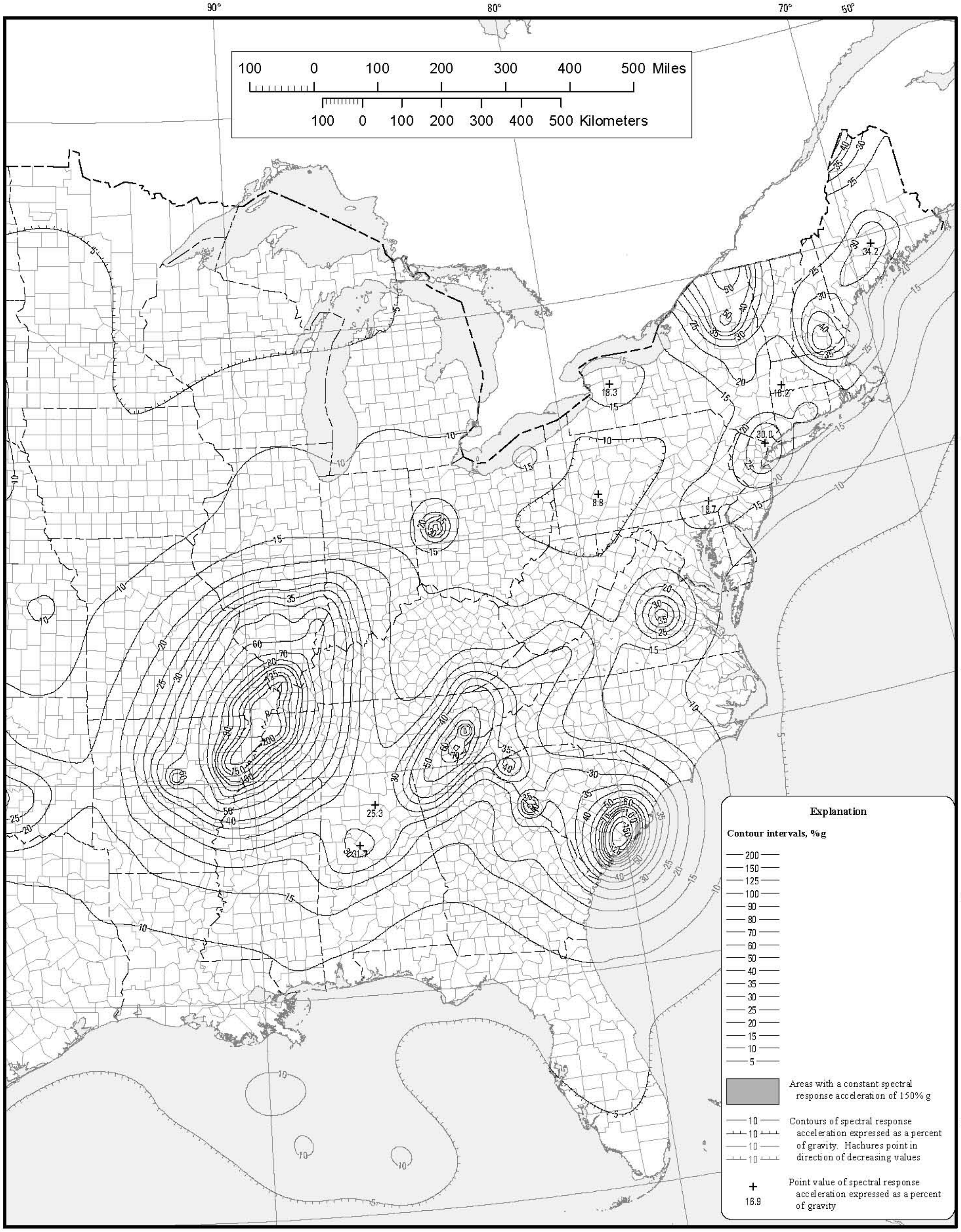
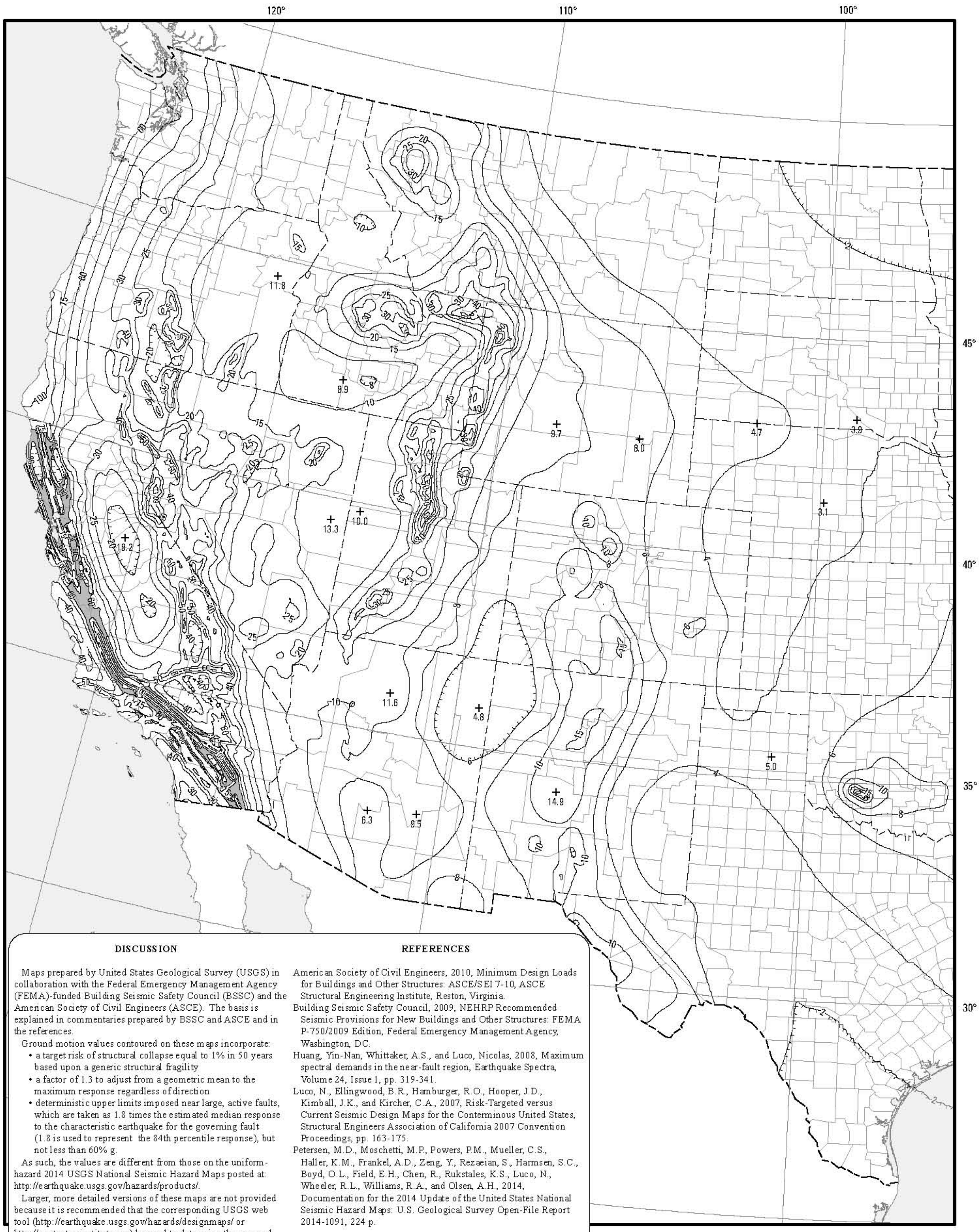


Figure 1613.3.1(1)-continued Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Response Accelerations for the Conterminous United States of 0.2-Second Spectral Response Acceleration (5% of Critical Damping), Site Class B

FIGURE ~~1613.3.1~~ 1613.3.1(2)-1 (2)

RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

(Existing code change figure not shown for clarity)



DISCUSSION

Maps prepared by United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency (FEMA)-funded Building Seismic Safety Council (BSSC) and the American Society of Civil Engineers (ASCE). The basis is explained in commentaries prepared by BSSC and ASCE and in the references.

Ground motion values contoured on these maps incorporate:

- a target risk of structural collapse equal to 1% in 50 years based upon a generic structural fragility
- a factor of 1.3 to adjust from a geometric mean to the maximum response regardless of direction
- deterministic upper limits imposed near large, active faults, which are taken as 1.8 times the estimated median response to the characteristic earthquake for the governing fault (1.8 is used to represent the 84th percentile response), but not less than 60% g.

As such, the values are different from those on the uniform-hazard 2014 USGS National Seismic Hazard Maps posted at: http://earthquake.usgs.gov/hazards/products/.

Larger, more detailed versions of these maps are not provided because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/hazards/designmaps/ or http://content.seinstitute.org) be used to determine the mapped value for a specified location.

REFERENCES

American Society of Civil Engineers, 2010, Minimum Design Loads for Buildings and Other Structures: ASCE/SEI 7-10, ASCE Structural Engineering Institute, Reston, Virginia.

Building Seismic Safety Council, 2009, NEHRP Recommended Seismic Provisions for New Buildings and Other Structures: FEMA P-750/2009 Edition, Federal Emergency Management Agency, Washington, DC.

Huang, Yin-Nan, Whittaker, A.S., and Luco, Nicolas, 2008, Maximum spectral demands in the near-fault region, Earthquake Spectra, Volume 24, Issue 1, pp. 319-341.

Luco, N., Ellingwood, B.R., Hamburger, R.O., Hooper, J.D., Kimball, J.K., and Kircher, C.A., 2007, Risk-Targeted versus Current Seismic Design Maps for the Conterminous United States, Structural Engineers Association of California 2007 Convention Proceedings, pp. 163-175.

Petersen, M.D., Moschetti, M.P., Powers, P.M., Mueller, C.S., Haller, K.M., Frankel, A.D., Zeng, Y., Rezaeian, S., Harmsen, S.C., Boyd, O.L., Field, E.H., Chen, R., Rukstales, K.S., Luco, N., Wheeler, R.L., Williams, R.A., and Olsen, A.H., 2014, Documentation for the 2014 Update of the United States National Seismic Hazard Maps: U.S. Geological Survey Open-File Report 2014-1091, 224 p.

Figure 1613.3.1(2) Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Response Accelerations for the Conterminous United States of 1-Second Spectral Response Acceleration (5% of Critical Damping), Site Class B *(continued)*

FIGURE ~~1613.3.1~~ 1613.3.1(2)-22 (2)

RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

(Existing code change figure not shown for clarity)

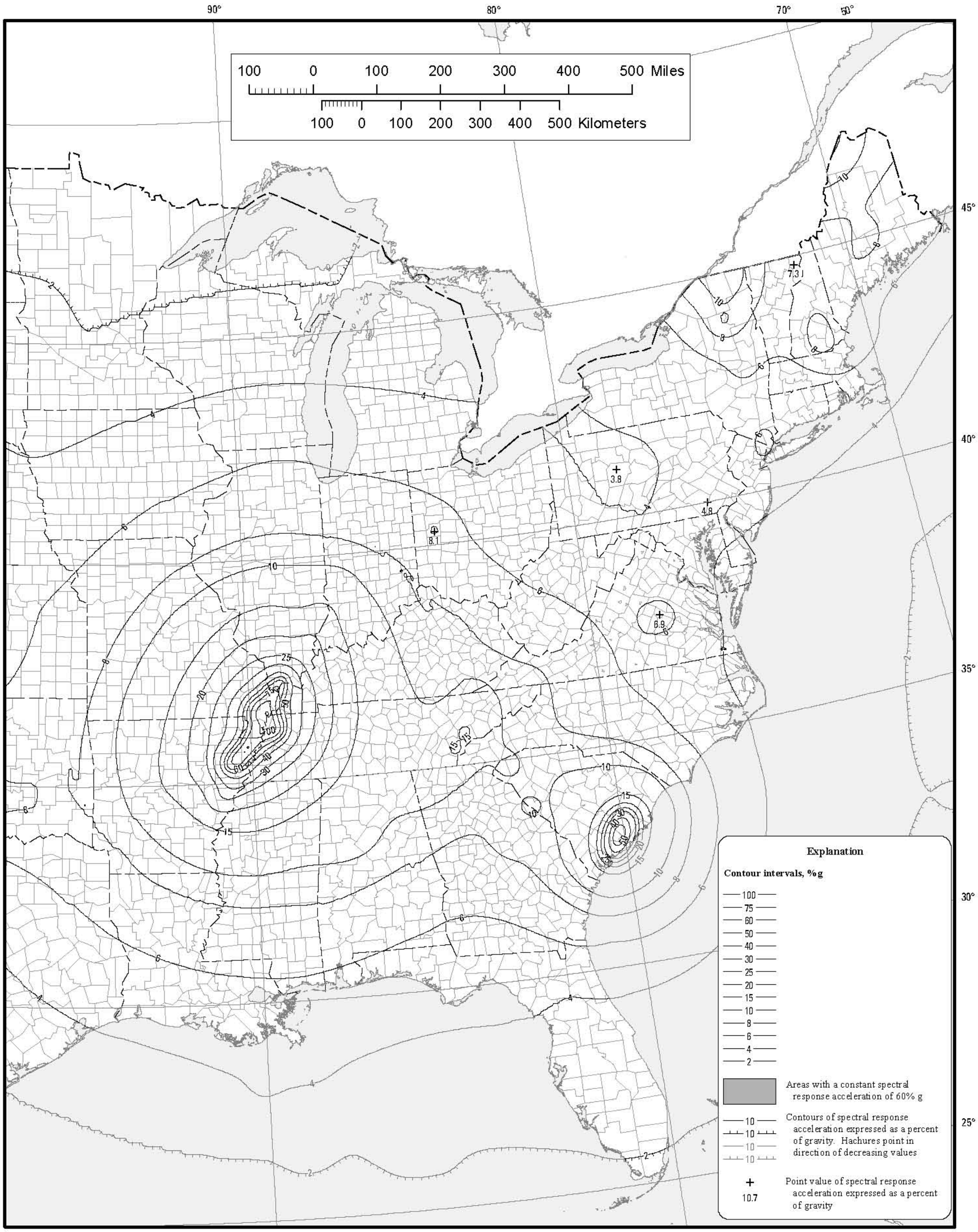
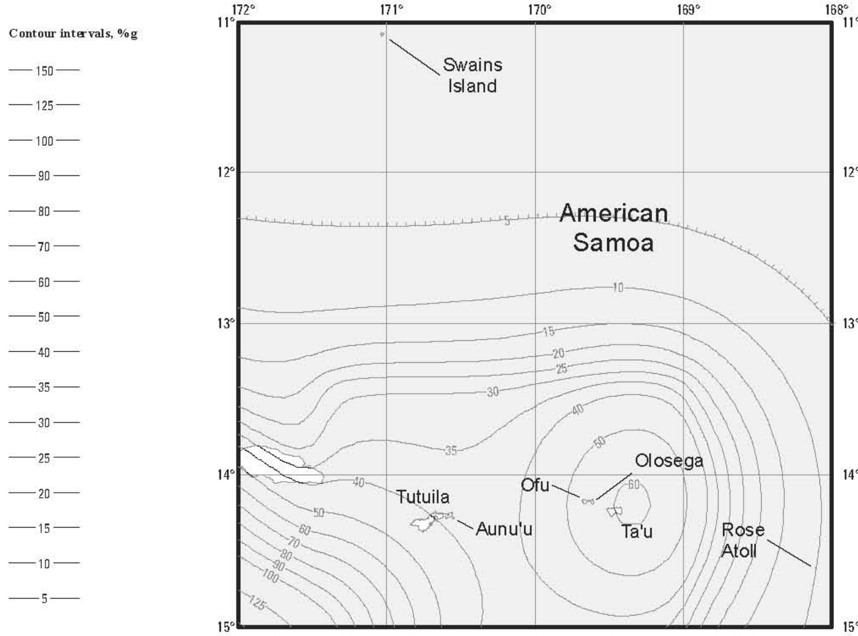


Figure 1613.3.1(2)-continued Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Response Accelerations for the Conterminous United States of 1-Second Spectral Response Acceleration (5% of Critical Damping), Site Class B

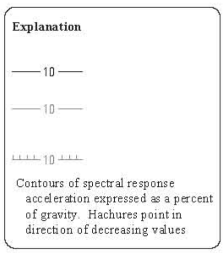
FIGURE ~~1613.3.1~~ 1613.3.1(8) (8)

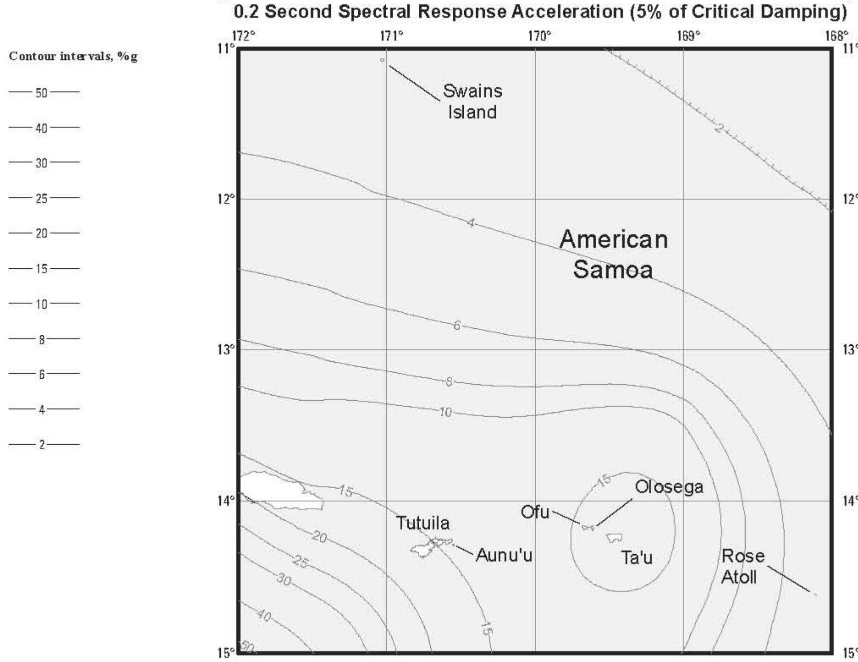
RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR AMERICAN SAMOA OF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

(Existing code change figure not shown for clarity)



0.2 Second Spectral Response Acceleration (5% of Critical Damping)





1.0 Second Spectral Response Acceleration (5% of Critical Damping)

DISCUSSION

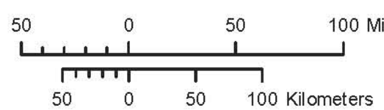
Maps prepared by United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency (FEMA)-funded Building Seismic Safety Council (BSSC). The basis is explained in commentary prepared by BSSC and in the references.

Ground motion values contoured on these maps incorporate:

- a target risk of structural collapse equal to 1% in 50 years based upon a generic structural fragility
- a factor of 1.1 and 1.3 for 0.2 and 1.0 sec, respectively, to adjust from a geometric mean to the maximum response regardless of direction
- deterministic upper limits imposed near large, active faults, which are taken as 1.8 times the estimated median response to the characteristic earthquake for the fault (1.8 is used to represent the 84th percentile response), but not less than 150% and 60% g for 0.2 and 1.0 sec, respectively.

As such, the values are different from those on the uniform-hazard 2012 USGS National Seismic Hazard Maps for American Samoa posted at http://earthquake.usgs.gov/hazmaps.

Larger, more detailed versions of these maps are not provided because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/designmaps) be used to determine the mapped value for a specified location.



REFERENCES

Building Seismic Safety Council, 2009, NEHRP Recommended Seismic Provisions for New Buildings and Other Structures: FEMA P-750/2009 Edition, Federal Emergency Management Agency, Washington, DC.

Huang, Yin-Nan, Whittaker, A.S., and Luco, Nicolas, 2008, Maximum spectral demands in the near-fault region, Earthquake Spectra, Volume 24, Issue 1, pp. 319-341.

Luco, Nicolas, Ellingwood, B.R., Hamburger, R.O., Hooper, J.D., Kimball, J.K., and Kircher, C.A., 2007, Risk-Targeted versus Current Seismic Design Maps for the Conterminous United States, Structural Engineers Association of California 2007 Convention Proceedings, pp. 163-175.

Petersen, M.D., Harmsen, S.C., Rukstales, K.S., Mueller, C.S., McNamara, D.E., Luco, Nicolas, and Walling, Melanie, 2012, Seismic Hazard of American Samoa and Neighboring South Pacific Islands: Data, Methods, Parameters, and Results: U.S. Geological Survey Open-File Report 2012–1087.

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

Reason: This proposal incorporates the most recent seismic design maps prepared by the U.S. Geological Survey

(USGS) in collaboration with the Federal Emergency Management Agency (FEMA) and the Building Seismic Safety Council (BSSC). The proposed maps are consistent with those incorporated into the NEHRP Provisions (FEMA P-1050-1, 2015) and ASCE 7-16. The proposed maps incorporate significant new information on earthquake faults and ground motion attenuation, and are more consistent with the site-specific ground motion procedures of ASCE 7-16 Chapter 21. Technical reasons behind the changes are documented in FEMA P-1050-1, Section C22. Further documentation is provided in Luco et al. (2015) and companion papers in a special issue of *Earthquake Spectra* (2015).

Bibliography: [NEHRP Recommended Seismic Provisions for New Buildings and Other Structures] [FEMA P-1050-1] [Building Seismic Safety Council] [2015] [483-495]
[https://www.fema.gov/media-library/assets/documents/107646]
[Earthquake Spectra] [Updates to Building-Code Maps for the 2015 NEHRP Recommended Seismic Provisions] [Luco, N, Bachman, R.e., Crouse, C.B., Harris, J.R., Hooper, J.D., Kircher, C.A., Caldwell, P.J., and Rukstales, K.S.] [2015] [Volume 31, pages S245-S271]

Cost Impact: Will increase the cost of construction
Use of the proposed maps can result in modest increases OR decreases in overall construction cost depending on the grographic location. Because the cost of structural systems is generally a small portion of the overall construction cost, the overall impact, whether increase or decrease, is thought to be quite modest.

Analysis: Coordinated code change proposal for the IRC is RB17-16.

S119-16 : FIGURE 1613.3.1-COBEEN12073

S120-16

IBC: 1613.3.3.

Proponent : James Bela, representing Oregon Earthquakie 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.~~ lateral design strength coefficients ~~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:~~

~~$S_{MS} = F_a S_s$ **(Equation 16-37)**~~

~~$S_{M1} = F_v S_1$ **(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_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.~~

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 Coeficient, 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 *Fa* and *Fv* (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 Respon se Modification, or **R Factors**.

1988 Uniform Building Code
http://shop.iccsafe.org/1988-uniform-building-code-download-1.html
http://www.amazon.com/Uniform-Building-Code-1988-Edition/dp/B003EB5BB0

1990 SEAOC BLUE BOOK
http://seaoc.org/bookstore/overview-perspective-1988-ubc-and-blue-book-applicable-1990-edition

1997 Uniform Building Code
http://www.iamcivilengineer.com/2014/08/download-uniform-building-code-ubc-97.html

Robert E. Bachman and David R. Bonneville (2000)
The Seismic Provisions of the 1997 Uniform Building Code. Earthquake Spectra: February 2000, Vol. 16, No. 1, pp. 85-100.
http://www.earthquakespectra.org/doi/abs/10.1193/1.1586084?journalCode=eqsa

Bibliography:

See also BIBLIOGRAPHY in Proposal: Figure 1613.3.1 RISK-TARGETED MCER

Cost Breakdown of Nonstructural Building Elements
Taghavi, S. and Miranda, E. (2003). Cost Breakdown of Nonstructural Building Elements, PEER Report 2003/05, Pacific 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 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://www.earthquakespectra.org/doi/abs/10.1193/1.4000032

Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Holliday, L. and Kang, T. (2014). "Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions." J. Perform. Constr. Facil., 10.1061/(ASCE)CF.1943-5509.0000630, 04014141.

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
California Seismic Safety Commission, 2005 Edition, 49 p.
http://www.disclosuresource.com/downloads/earthquake.pdf

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
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

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 flaw ed 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 alw ays 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, 2013, 249 p. NEHRP Consultants Joint Venture A partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering.

In general, w here costs might be increased, cost premiums above requirements for w ind tend to fall w ithin a range of +1-3%. For cases w here seismic requirements w ould be now additional to w hat previous codes either applied/neglected/failed to enforce, estimates probably w ould fall w ithin the range of 0.25 - 1%.

{{1143}}

Analysis: A review of the standard(s) proposed for inclusion in the code, 1997 UBC, w ith regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) w ill be posted on the ICC w ebsite on or before April 1, 2016.

S120-16 : 1613.3.3-BELA12903

S121-16

IBC: 1613.3.4.

Proponent : James Bela, representing Oregon Earthquakie 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:~~

$$V_{asd} = V_{ult}\sqrt{0.6}$$ ~~(Equation 16-39)~~

$$S_{DS} = \frac{2}{3}S_{MS}$$ ~~(Equation 16-40)~~

~~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 unw arranted, 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 anyw ay) is rapidly lost in the shuffle in comparison to other considerations, such as the Respon se Modification, or R Factors.

Furthermore, taking 2/3rds of a number that is *not stable* betw een successive code editions is not only problematical, but also very unw ise w ith regard to both public safety and community resilience!

1988 Uniform Building Code
http://shop.iccsafe.org/1988-uniform-building-code-dow nload-1.html
http://w w w .amazon.com/Uniform-Building-Code-1988-Edition/dp/B003EB5BB0

1990 SEAOC BLUE BOOK
http://seaoc.org/bookstore/overview -perspective-1988-ubc-and-blue-book-applicable-1990-edition

1997 Uniform Building Code
http://w w w .iamcivilengineer.com/2014/08/dow nload-uniform-building-code-ubc-97.html

Robert E. Bachman and David R. Bonneville (2000)
The Seismic Provisions of the 1997 Uniform Building Code. Earthquake Spectra: February 2000, Vol. 16, No. 1, pp. 85-100.
http://w w w .earthquakespectra.org/doi/abs/10.1193/1.1586084?journalCode=eqsa

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doi:http://dx.doi.org/10.1193/1.4000032

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Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions
Holliday, L. and Kang, T. (2014). "Low-Cost Earthquake Solutions for Nonengineered Residential Construction in Developing Regions." J. Perform. Constr. Facil., 10.1061/(ASCE)CF.1943-5509.0000630, 04014141.

Permalink: http://dx.doi.org/10.1061/(ASCE)CF.1943-5509.0000630
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http://www.disclosuresource.com/downloads/earthquake.pdf

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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
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 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
Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee, 2013, 249 p. 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}}

S121-16 : 1613.3.4-BELA12975

S122-16

IBC: 1613.3.5, 1613.3.5.1, 1613.3.5.2.

Proponent : James Bela, representing Oregon Earthquakie 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_{DS} and S_{D1}, 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_a, 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_S 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*.

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http://shop.iccsafe.org/1988-uniform-building-code-download-1.html
http://www.amazon.com/Uniform-Building-Code-1988-Edition/dp/B003EB5BB0

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http://seaoc.org/bookstore/overview-perspective-1988-ubc-and-blue-book-applicable-1990-edition

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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, w hen one looks at the true "life-cycle" costs of a large population of buildings, w hen 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-dow n betw een map editions, this could effect cost accordingly, but I estimate that this w ould be minor.

This proposal may or may not affect the cost of construction. This is (1) because detached one- and tw o-family dw ellings must be already built to w ithstand the lateral forces due to w ind; 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 tw o-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 flaw ed 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 alw ays 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."*

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Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee, 2013, 249 p. NEHRP Consultants Joint Venture A partnership of the Applied Technology Council and the Consortium of Universities for Research in Earthquake Engineering.

In general, w here costs might be increased, cost premiums above requirements for w ind tend to fall w ithin a range of +1-3%. For cases w here seismic requirements w ould be now additional to w hat previous codes either applied/neglected/failed to enforce, estimates probably w ould fall w ithin the range of 0.25 - 1%.

{{1143}}

S122-16 : 1613.3.5-BELA13016

S123-16

IBC: 1613.5 (New), 1613.5.1 (New), 1613.5.1.1 (New), 1613.5.1.2 (New), [A] 104.11.

Proponent : Mark Moore, ZFA Structural Engineers, representing Structural Engineers Association of Northern California and National Council of Structural Engineers Association

2015 International Building Code

Add new text as follows:

1613.5 Additional requirements for alternative materials, design and methods of construction. The provisions of Section 1613.5 shall be in addition to the requirements of Section 104.11 and shall apply to buildings and structures that are assigned to Seismic Design Category C, D, E or F.

1613.5.1 Evaluation report and review. The *building official* shall require either an evaluation report or a project-specific peer review for structural components or systems in the earthquake load path. An evaluation report or a peer review report shall be submitted to the *building official*. The alternative material, design or method of construction shall be *approved* where the *building official* finds that the proposed design meets this section and Section 104.11.

1613.5.1.1 Evaluation report. The evaluation report shall be prepared by an *approved agency* regularly engaged in evaluating structural components or systems in accordance with the building code and referenced standards. The agency shall:

1. Use a written acceptance criteria or evaluation criteria which is an agreed upon set of conditions or characteristics that must be present in order for an alternative material, design or method to be evaluated.
2. Engage an independent peer review panel to review the acceptance criteria and evaluation report. The independent peer review panel shall consist of at least two subject-matter experts for the product or system for which they are providing a review. The independent peer review panel shall provide their findings and recommendations to the evaluation report provider.
3. Use a public and transparent majority approval process to develop acceptance criteria with a minimum of a 30 calendar day public comment period. Responses to the public comments shall be made public.
4. Maintain data confidentiality and impartiality during the evaluation report development process.
5. Make publically available all acceptance or evaluation criteria.
6. Engage registered design professionals for the evaluation process.
7. Maintain on file all supporting data, reports and comments used for evaluation and comparison to the acceptance criteria.
8. Ensure results are comparable and reproducible.
9. Establish protocols for a minimum of annual component or system manufacture surveillance, resolving complaints and addressing non-conformity

1613.5.1.2 Project-specific peer review. A peer reviewer(s) shall be a subject-matter expert approved by the *building official.* The peer review report shall include findings and a recommendation to the *building official*.

Revise as follows:

[A] 104.11 Alternative materials, design and methods of construction and equipment. The provisions of this code are not intended to prevent the installation of any material or to prohibit any design or method of construction not specifically prescribed by this code, provided that any such alternative has been *approved*. An alternative material, design or method of construction shall be *approved* where the *building official* finds that the proposed design is satisfactory and complies with the intent of the provisions of this code, and that the material, method or work offered is, for the purpose intended, not less than the equivalent of that prescribed in this code in quality, strength, effectiveness, *fire resistance*, durability and safety. Where the alternative material, design or method of construction is not *approved*, the *building official* shall respond in writing, stating the reasons why the alternative was not *approved*. <u>The additional requirements of Section 1613.5 shall apply to buildings and structures assigned to Seismic Design Category C, D, E or F.</u>

Reason: Compromise to Life Safety and/or Property Preservation under earthquake loading: Individual opinion and relaxed design and inspection requirements can and have occurred through application of current 104.11 provisions for components and systems in the earthquake load path.
Great codes, standards, and designs are ineffective if we do not have consistent compliance. Structural systems are a significant factor in damage due to seismic events as well as for public safety. Codes are developed through a rigorous governmental consensus process and standards are developed through a rigorous consensus process. Within the structural provisions of the I-codes and referenced standards, there are more robust component and system-specific design and inspection requirements for areas of higher seismic risk in recognition of the higher risk these areas pose to the public. The structural provisions, which are publically developed and consensus based, lead to a targeted level of reliability. The rigor and intent of these provisions are not captured in the current 104.11 provisions unless the Building Official knows of, and is conversant with, their development.

The Building Official is now obligated to describe code provision-specific reasons why they find a component or system non-compliant. In the 2015 IBC/IRC a substantive revision occurred where the last sentence was added stating, "*Where the alternative material, design or method of construction is not approved, the building official shall respond in writing, stating the reasons why the alternative was not approved.*" This was the first step in creating a transparent, consistent reasoning process for adhering to 104.11, though it does not go far enough to realistically do so as it only covers what the building official is required to do. More onus on the proponent and peer reviewers is required.

The current 104.11 provisions can and have been abused allowing premanufactured components or systems to be approved as the primary seismic force-resisting system whereas such components or systems using a ASCE-venue for evaluation would likely be rejected or significantly augmented with prescriptive design or inspection requirements.

Currently, component or system approval criteria is developed and used without a majority approval or consensus process, thus allowing unvetted criteria to be approved and used in structural designs. To remedy this, it is strongly recommended that similar public protection be provided by requiring a clearly defined and robust review for alternative seismic force-resisting systems in areas of greater seismic risk. This will improve uniformity of requirements across jurisdictions and better ensure the safety of our built environment.

SEAOC and NCSEA therefore support the additional provisions to Sections 104.11 and 1613.5 for components and systems in the earthquake load path so that a more robust, codified majority approval process is used.

For higher seismic areas, this code change seeks to provide clarity and add rigor to the alternative process requirements to improve consistency and better protect public safety and property. Further discussion points in support of this code addition are as follows:

- Some advocates of proprietary materials or assemblies understand the requirements in each code cycle and navigate the approval process to properly meet code intent and build industry confidence in their product or system. Other advocates of these alternatives, however, have limited understanding of code requirements and assumptions, often seeking to waive or reduce code minimums without fully exploring the impact of their proposals on the behavior of the structure as a whole. Lastly, some advocates use their extensive knowledge of code to utilize vagaries through 104.11 to put components and systems in a stronger market position by either not meeting or exceeding code intent, and setting precedence. An effective peer review process can help to normalize the understanding of the advocates and bring consistency to evaluation reports.

- While the building official retains sole authority for all approvals, a more rigorous process to be followed by evaluation report providers should help to better inform and guide the building official when considering alternatives in higher seismic regions. This should help to relax the pressure on building officials when they exercise the powers provided to them under the code. Additionally, information flow from the advocate to the building official will be more reliable given the better substantiated and consistent requirements.
- Subject-matter experts (SME) shall be available and routinely called upon to provide input on products, systems or design methodology evaluations. SME qualification is subject to the Building Official. While requirements for the classification of an SME are difficult to codify, they are necessary to ensure a proper review is achieved or least to provide code provisions from which the building official can identify reasons for nonconformance. The SME are expected to ensure that tests do not result in an increase in the conditional probability of collapse given MCE_R ground shaking. The SME will review effects of differences in hysteretic characteristics of both the code-prescribed and alternative systems as demonstrated by laboratory testing; the type of failure of the alternative system; uncertainties associated with laboratory test data for the code prescribed and alterative system; the repeatability of laboratory test results in production of alternative systems; and the statistical adequacy of the available laboratory testing database. This will likely create a talent resource pool which can be relied upon by building officials as an additional resource.
- The requirements provide a minimum set of standards for those third party evaluation report providers who wish to provide alternative product, system or design methodology evaluation services and will result in a clear, consistent and transparent process enforceable by law which shall provide the proper balance of innovation and public safety in high seismic regions.

Lastly, the one-off (project-specific) application of 104.11 process is retained so that introducing technologies can still be performed by a competent professional engineer with a peer review.

Bibliography: SEAOC Blue Book article "An Evaluation of Current practices Related to the Development and Implementation of Acceptance Criteria and Product Evaluation Reports," October 2011

Cost Impact: Will not increase the cost of construction
There is a modest change in process to the larger agencies that provide evaluation reports to be in compliance with the proposed provisions. Those costs are insignificant compared to the material and fabrication construction cost. There will be higher initial cost to demonstrate a component or system is code compliant for entities that do not have in place the proposed requirements, but this represents a small but growing section of evaluation report providers. Indeed, providing a consistent set of evaluation report development process requirements may create efficiency in the process and diminish lengthy project-by-project reviews, which would reduce construction cost.

S123-16 : 1613.5 (NEW)-MOORE12847

S124-16

IBC: 1613.6.

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

2015 International Building Code

Revise as follows:

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~~*approved* analysis or shake-table testing, using input motions consistent with ASCE 7 lateral and vertical seismic forces for nonstructural components on roofs.

Reason: When preparing structural analysis to justify a ballasted, nonpenetrating photovoltaic panel system, nonlinear response-history analysis is just one option. For example, the method developed by the Structural Engineers Association of California is a simplified displacement method. This method is incorporated in the draft of ASCE 7-16, w hich is under development. The revision to allow approved analysis is consistent w ith the 2013 California Building Code, as approved by the California Building Standards Commission.

Bibliography: California Building Code, Part 2, CBSC, 2013, Page #47
http://codes.iccsafe.org/app/book/content/PDF/2013/2013%20California/13Building/PDFs/Chapter%2016%20-%20Structural%20Design.pdf

Cost Impact: Will not increase the cost of construction
This proposal w ill allow more flexibility in choosing a method of analysis for justifying ballasted, unattached rooftop PV systems. Engineering cost w ill be the same or less. Cost of construction w ill be the same or less.

S124-16 : 1613.6-CAIN10999

S125-16

IBC: 1615.1.

Proponent : James Bela, representing Oregon Earthquakie 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.~~
<u>*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.</u>

Reason: High rise buildings (> 75 ft) are an important factor in determining community resilience, and therefore they are w orthy of additional design effort and care.

<u>**Cities, Earthquakes, and Time**</u>
Cities, Earthquakes, and Time - https://w w w .youtube.com/w atch?v=EdKJna-MYY4
https://w w w .youtube.com/w atch?v=EdKJna-MYY4

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

S125-16 : 1615.1-BELA13461

S126-16

IBC: 1615.1.

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

2015 International Building Code

Revise 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~~ Section 1615.3 if they are frame structures ~~shall comply with the requirements of~~ , or Section ~~1615.3. Bearing~~ 1615.4 if they are bearing wall structures ~~shall comply with the requirements of Section 1615.4~~.

Reason: As currently written, one could mistakenly interpret that all of section 1615.1 applies to all *High-rise buildings* that are assigned to *Risk Category* III or IV. That is incorrect. Only section 1615.3 or 1615.4 apply based on the type of structure for such buildings. In addition, as currently written, one could mistakenly interpret that Section 1615.3 applies to all frame structures and that Section 1615.4 applies to all bearing wall structures. That also is incorrect, and when these provisions were added into the IBC, the requirements of 1615.3 were to apply to frame structures that were *High-rise buildings* and that were assigned to *Risk Category* III or IV. Likewise the requirements of 1615.4 were to apply to bearing wall structures that were *High-rise buildings* and that were assigned to *Risk Category* III or IV. This proposed change corrects the section to be consistent with the intent and clarifies the provisions. A designer or owner still has the option to apply these provisions to other buildings in other risk categories, but it would not be required.

Cost Impact: Will not increase the cost of construction
The cost of construction will not increase by clarifying the provisions of 1615.1, and the clarification could decrease cost due to misinterpretation.

S126-16 : 1615.1-HUSTON13284

S127-16

IBC: 1616 (New), 1616.1 (New), 1616.2 (New), 1616.3 (New), 1616.4 (New), 1616.5 (New), 1616.5.1 (New), 1616.6 (New), 1616.6.1 (New), 1616.7 (New), 1616.8 (New), 1616.8.1 (New), 1616.8.2 (New).

Proponent : Karl Rubenacker, Gilsanz Murray Steficek, representing Codes & Standards Committee, Structural Engineer's Association of New York (karl.rubenacker@gmsllp.com)

2015 International Building Code

Add new text as follows:

SECTION 1616 STRUCTURAL PEER REVIEW

1616.1 General The provisions of this section specify where structural peer review is required, how and by whom it is to be performed.

1616.2 Where required A structural peer review of the primary structure shall be performed and a report provided for the following buildings:

1. Buildings classified as Risk Category IV and more than 50,000 square feet (4645 m2) of framed area.
2. Buildings with 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 using nonlinear time history analysis or with special seismic energy dissipation systems.
6. Buildings designed for areas with 3,000 or more occupants in one area in close proximity, including fixed seating and grandstand areas.
7. Buildings where a structural peer review is requested by the *building official*.

1616.3 Structural peer review. It shall be verified that the structural design is in general conformance with the requirements of this code.

1616.4 Structural peer reviewer The structural peer review shall be performed by a qualified independent structural engineer who has been retained by or on behalf of the owner. A structural peer reviewer shall meet the qualification requirements of the *building official*.

1616.5 Extent of the structural peer review. The structural peer reviewer shall review the plans and specifications submitted with the permit application for general compliance with the structural and foundation design provisions of this code. The reviewing engineer shall perform the following tasks at a minimum:

1. Confirm that the design loads conform to this code.
2. Confirm that other structural design criteria and design assumptions conform to this code and are in accordance with generally accepted engineering practice.
3. Review geotechnical and other engineering investigations that are related to the foundation and structural design and confirm that the design properly incorporates the results and recommendations of the investigations.
4. Review the structural frame and the load supporting parts of floors, roofs, walls and

foundations. Cladding, cladding framing, stairs, equipment supports, ceiling supports, non-loadbearing partitions, railings and guards, and other secondary structural items shall be excluded.

5. Confirm that the structure has a complete load path.
6. Perform independent calculations for a representative fraction of systems, members, and details to check their adequacy. The number of representative systems, members, and details verified shall be sufficient to form a basis for the reviewer's conclusions.
7. Verify that performance-specified structural components (such as certain precast concrete elements) have been appropriately specified and coordinated with the primary building structure.
8. Verify that the design engineer of record complied with the structural integrity provisions of the code.
9. Review the structural and architectural plans for the building. Confirm that the structural plans are in general conformance with the architectural plans regarding loads and other conditions that may affect the structural design.
10. Confirm that major mechanical items are accommodated in the structural plans.
11. Attest to the general completeness of the structural plans and specifications.

1616.5.1 Structural design criteria. When the design criteria and design assumptions are not shown on the drawings or in the computations, the structural engineer of record shall provide a statement of these criteria and assumptions for the reviewer. In addition, the design engineer shall provide information and/or calculations, if requested by the peer reviewer.

1616.6 Structural peer review report. The reviewing engineer shall submit a report to the *building official* stating whether or not the structural design shown on the plans and specifications generally conforms to the structural and foundation requirements of this code.

1616.6.1 Contents. The report shall demonstrate, at a minimum, compliance with Items 1 through 11 of Section 1616.5. In addition, the report shall also include the following:

1. The codes and standards used in the structural design of the project.
2. The structural design criteria, including loads and performance requirements.
3. The basis for design criteria that are not specified directly in applicable codes and standards. This should include reports by specialty consultants such as wind tunnel study reports and geotechnical reports. Generally, the report should confirm that existing conditions at the site have been investigated as appropriate and that the design of the proposed structure is in general conformance with these conditions.

1616.7 Phased submission. If an application is submitted for a permit for the construction of foundations or any other part of a building before the construction documents for the whole building have been submitted, then the structural peer review and report shall be phased. The structural peer reviewer shall be provided with sufficient information on which to make a structural peer review of the phased submission.

1616.8 Responsibility. Responsibilities of the structural engineer of record and the structural peer reviewer shall be in accordance with this section.

1616.8.1 Structural engineer of record. The structural engineer of record shall retain sole responsibility for the structural design. The activities and reports of the structural peer reviewer shall not relieve the structural engineer of record of this responsibility.

1616.8.2 Structural peer reviewer. The structural peer reviewer's report shall state his or her

opinion regarding the design by the engineer of record. The standard of care to which the structural peer reviewer shall be held in the performance of the structural peer review and report is that the level of skill and care are consistent with structural peer review services performed by professional engineers licensed in the state in which the project is located for similar types of projects.

Reason: It is vitally important to ensure public safety of significant structures.
The structures meeting this criteria are more complex than a typical structure, have critical services, or are of enormous size, with significant consequences of failure.

The peer review process improves the engineering profession.

Cost Impact: Will not increase the cost of construction
Will not increase the cost of construction, peer review occurs during design. The affected structures are a very small subset of the building population.

S127-16 : 1616 (NEW)-RUBENACKER13478

S128-16

IBC: 1704.6, 1704.6.1, 1704.6.2.

Proponent : Gerald Gunny, City of Henderson Department of Building and Fire, representing Southern Nevada Chapter of the International Code Council

2015 International Building Code

Revise as follows:

1704.6 Structural observations. Where required by the provisions of Section 1704.6.1 or 1704.6.2 <u>or when specifically required by the building official</u>, the owner or the owner's authorized agent shall employ a *registered design professional* to perform structural observations. Structural observation does not include or waive the responsibility for the inspections in Section 110 or the *special inspections* in Section 1705 or other sections of this code.

Prior to the commencement of observations, the structural observer shall submit to the *building official* a written statement identifying the frequency and extent of structural observations.

At the conclusion of the work included in the permit, the structural observer shall submit to the *building official* a written statement that the site visits have been made and identify any reported deficiencies that, to the best of the structural observer's knowledge, have not been resolved.

1704.6.1 Structural observations for seismic resistance. Structural observations shall be provided for those structures assigned to *Seismic Design Category* D, E or F where one or more of the following conditions exist:

1. The structure is classified as *Risk Category* III or IV.
2. The height of the structure is greater than 75 feet (22 860 mm) above the base as defined in ASCE 7.
3. The structure is assigned to *Seismic Design Category* E, is classified as *Risk Category* I or II, and is greater than two *stories abovegrade plane*.
4. When so designated by the *registered design professional* responsible for the structural design.
5. ~~When such observation is specifically required by the *building official*.~~

1704.6.2 Structural observations for wind requirements. Structural observations shall be provided for those structures sited where V_{asd} as determined in accordance with Section 1609.3.1 exceeds 110 mph (49 m/sec), where one or more of the following conditions exist:

1. The structure is classified as *Risk Category* III or IV.
2. The *building height* is greater than 75 feet (22 860 mm).
3. When so designated by the *registered design professional* responsible for the structural design.
4. ~~When such observation is specifically required by the *building official*.~~

Reason: The intent of this revision is to allow the building official to require structural observations for structurally complex or special occupancy buildings that are not otherwise required by Sections 1704.6.1 or 1704.6.2.

Cost Impact: Will increase the cost of construction
Increase in project fees associated with a registered design professional providing the structural observation services when otherwise not required by Sections 1704.6.1 or 1704.6.2.

S128-16 : 1704-GUNNY11595

S129-16

IBC: 1704.2, 1705.11.2, 1705.12.3, 1705.2.2, 1705.2.2 (New), 1705.2.2.2 (New), 1705.2.2.3 (New), 1705.2.2.3.1 (New), 1705.2.4.

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 *specialinspections* 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 ~~2211.7~~ 2211.1.2 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~~ 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~~ is installed in accordance with the *approved* ~~truss submittal package~~ design as specified in Section 2211.1.3.

Delete and substitute as follows:

1705.11.2 Cold-formed steel light-frame construction. ~~*Periodicspecial 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.~~

> ~~**Exception:** *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 (ie. 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

S130-16

IBC: 1704.2.3.

Proponent : Mark Gilligan, representing Self

2015 International Building Code

Revise as follows:

1704.2.3 Statement of special inspections. The applicant shall submit a statement of *special inspections* in accordance with Section 107.1 as a condition for permit issuance. This statement shall be in accordance with Section 1704.3.

> ~~**Exception:** A statement of *special inspections* is not required for portions of structures designed and constructed in accordance with the cold-formed steel light-frame construction provisions of Section 2211.7 or the conventional light-frame construction provisions of Section 2308.~~

Reason: Delete exception since Exception #3 to Section 1704.2 already excepts the same w ork from special inspections..

Cost Impact: Will not increase the cost of construction
Since this change eliminates a redundancy there w ill be no impact on cost.

S130-16 : 1704.2.3-GILLIGAN11827

S131-16

IBC: 1704.2.5, 1704.2.5.1, 1704.5.

Proponent : Bonnie Manley, AISI, representing American Institute of Steel Construction (bmanley@steel.org)

2015 International Building Code

Revise as follows:

1704.2.5 Special inspection of fabricated items. Where fabrication of structural, load-bearing or lateral load-resisting members or assemblies is being conducted on the premises of a fabricator's shop, *special inspections* of the *fabricated items* shall be performed during fabrication, except where the fabricator has been approved by the *building official* to perform work without *special inspections* in accordance with 1704.2.5.1.

~~**Exceptions:**~~

1. ~~*Special inspections* during fabrication are not required where the fabricator maintains *approved* detailed fabrication and quality control procedures that provide a basis for control of the workmanship and the fabricator's ability to conform to *approved construction documents* and this code. Approval shall be based upon review of fabrication and quality control procedures and periodic inspection of fabrication practices by the building official.~~
2. ~~Special inspections are not required where the fabricator is registered and *approved* in accordance with Section 1704.2.5.1.~~

1704.2.5.1 Fabricator approval. *Special inspections* during fabrication are not required where the work is done on the premises of a fabricator ~~registered and~~ approved to perform such work without *special inspection*. Approval shall be based upon review of the fabricator's written ~~procedural~~ fabrication procedures and quality control manuals that provide a basis for control of materials and workmanship, with periodic auditing of fabrication and quality control practices by an *approved agency* or by the ~~*approved agency*~~ *building official*. At completion of fabrication, the *approved* fabricator shall submit a *certificate of compliance* to the owner or the owner's authorized agent for submittal to the *building official* as specified in Section 1704.5 stating that the work was performed in accordance with the *approved construction documents*.

1704.5 Submittals to the building official. In addition to the submittal of reports of *special inspections* and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner's authorized agent to the *building official* for each of the following:

1. *Certificates of compliance* for the fabrication of structural, load-bearing or lateral load-resisting members or assemblies on the premises of ~~a registered and~~ an *approved fabricator* in accordance with Section 1704.2.5.1.
2. *Certificates of compliance* for the seismic qualification of nonstructural components, supports and attachments in accordance with Section 1705.13.2.
3. *Certificates of compliance* for *designated seismic systems* in accordance with Section 1705.13.3.
4. Reports of preconstruction tests for shotcrete in accordance with Section 1908.5.
5. *Certificates of compliance* for open web steel joists and joist girders in accordance with Section 2207.5.

6. Reports of material properties verifying compliance with the requirements of AWS D1.4 for weldability as specified in Section 26.6.4 of ACI 318 for reinforcing bars in concrete complying with a standard other than ASTM A 706 that are to be welded; and
7. Reports of mill tests in accordance with Section 20.2.2.5 of ACI 318 for reinforcing bars complying with ASTM A 615 and used to resist earthquake-induced flexural or axial forces in the special moment frames, special structural walls or coupling beams connecting special structural walls of *seismic force-resisting systems* in *structures* assigned to *Seismic Design Category* B, C, D, E or F.

Reason: The primary purpose of this proposal is to streamline and focus the provisions for fabricator approval. Section 1704.2.5, Exception 1 and Section 1704.2.5.1 provide almost identical paths for obtaining fabricator approval, with only slight differences in presentation. Trying to discern the subtle differences can often prove frustrating to the user. So, the proposed modification consolidates Section 1704.2.5 Exception 1 with 1704.2.5.1. Now extraneous, Section 1704.2.5 Exception 2 is recommended for deletion.
Additional modifications to the section include deleting the term "registered" from the applicable sections. It is not a defined term in the IBC for fabricators, so its true intent is open to interpretation. The proposal also adds a reference to material control, which typically refers to the general oversight of the materials by the fabricator and involves procedures for storage, release and movement of materials throughout the fabrication processes. Finally, the proposal adds requirements for periodic auditing of quality control practices in addition to the auditing of fabrication practices. It seems reasonable to add auditing of the quality control practices, since initial approval is based upon review of both the written fabrication procedures and quality control manuals.

Cost Impact: Will not increase the cost of construction
This modification is primarily editorial and is not intended to have a significant impact on current costs.

S131-16 : 1704.2.5-MANLEY12811

S132-16

IBC: , 1704.2.6 (New), 1704.2.6.1 (New), 1704.5, 202 (New).

Proponent : Bonnie Manley, AISI, representing American Institute of Steel Construction (bmanley@steel.org)

2015 International Building Code

Add new definition as follows:

SECTION 202 DEFINITIONS

APPROVED ERECTOR An established and qualified person, firm or corporation *approved* by the building official pursuant to Chapter 17 of this code.

SECTION 202 DEFINITIONS

ERECTED ITEM Fabricated structural, load-bearing or lateral load-resisting members or assemblies, and structural materials produced in accordance with standards referenced by this code, installed in a building or structure.

Add new text as follows:

1704.2.6 Special inspection of erected items. *Special inspections* of the *erected items* shall be performed during erection of a building or structure.

Exceptions:

1. *Special inspections* during erection are not required where the erector maintains *approved* detailed erection quality control procedures that provide a basis for control of the workmanship and the erector's ability to conform to *approved construction documents* and this code. Approval shall be based upon review of the erector's quality control procedures and periodic inspection of the erector's practices by the *building official*.
2. Special inspections are not required where the erector is registered and *approved* in accordance with Section 1704.2.6.1.

1704.2.6.1 Erector approval. *Special inspections* during erection are not required where the work is done during installation in a building or structure by an erector registered and *approved* to perform such work without *special inspection*. Approval shall be based upon review of the erector's written procedural and quality control manuals and periodic auditing of the erector's practices by an *approved agency*. At completion of erection, the *approvederector* shall submit a *certificate of compliance* to the owner or the owner's authorized agent for submittal to the *building official* as specified in Section 1704.5 stating that the materials supplied and work performed by the erector are in accordance with the *approved construction documents*.

Revise as follows:

1704.5 Submittals to the building official. In addition to the submittal of reports of *special inspections* and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner's authorized agent to the *building official* for each of the following:

1. *Certificates of compliance* for the fabrication of structural, load-bearing or lateral load-resisting members or assemblies on the premises of a registered and *approved*

fabricator in accordance with Section 1704.2.5.1.

2. *Certificates of compliance* for the erection of structural, load-bearing or lateral load-resisting members or assemblies by an *approved erector* in accordance with Section 1704.2.6.1.
3. *Certificates of compliance* for the seismic qualification of nonstructural components, supports and attachments in accordance with Section 1705.13.2.
4. *Certificates of compliance* for *designated seismic systems* in accordance with Section 1705.13.3.
5. Reports of preconstruction tests for shotcrete in accordance with Section 1908.5.
6. *Certificates of compliance* for open web steel joists and joist girders in accordance with Section 2207.5.
7. Reports of material properties verifying compliance with the requirements of AWS D1.4 for weldability as specified in Section 26.6.4 of ACI 318 for reinforcing bars in concrete complying with a standard other than ASTM A 706 that are to be welded; and
8. Reports of mill tests in accordance with Section 20.2.2.5 of ACI 318 for reinforcing bars complying with ASTM A 615 and used to resist earthquake-induced flexural or axial forces in the special moment frames, special structural walls or coupling beams connecting special structural walls of *seismic force-resisting systems* in *structures* assigned to *Seismic Design Category* B, C, D, E or F.

Reason: The overall quality of a structure is improved w hen and w here quality procedures are in place and a defined quality process is applied during construction. On the other hand, inspection, w hile useful to detect occasional errors, does not necessarily catch all errors nor does it lead to improved overall quality w hen implemented solely on its ow n.
Since its inception, the IBC has permitted an approved fabricator exemption from special inspections w hen the fabricator meets the requirements specified in Section 1704.2.5. General consensus is that implementation of the approved fabricator exemption has resulted in improved quality of construction. This proposal seeks to extend this concept and its resulting improvements in quality to erectors.

The 2010 edition of AISC 360, Chapter N, *Quality Control and Quality Assurance*, first introduced the concept of an approved erector for structural steel construction. Chapter N requires quality control inspections to be performed by the erector, quality assurance inspections to be performed by the special inspector, and non-destructive testing (NDT) to be performed by the special inspector. Section N7 allow s quality assurance inspections to be performed by an erector approved by the authority having jurisdiction. How ever, the required NDT is not permitted to be performed by an erector, even an approved erector. For the w aiver of third-party inspection, at completion of erection, the approved erector is required to submit a certificate of compliance to the authority having jurisdiction stating that the materials supplied and the w ork performed by the erector are in accordance w ith the approved construction documents.

An example of how an erector may be approved by the authority having jurisdiction is through the use of the AISC Certification program, w hich is detailed under "Certifications" at w w w .aisc.org. Steel erectors may be designated as an AISC Certified Erector or AISC Advanced Certified Steel Erector. Requirements for approved erectors can be found in AISC 206-13, *AISC Certification Program for Structural Steel Erectors*. The audits performed under this program confirm that the company has the personnel, know ledge, organization, equipment, experience, capability, procedures and commitment to produce the required quality of w ork for a given certification category.

The definitions proposed for Section 202 are based upon similar definitions for both approved fabricator and fabricated item. How ever, for erected item, the definition has been expanded to consider that erectors can w ork w ith both fabricated items and non-fabricated items.

The language proposed for Section 1704.2.6 parallels the language currently in place for the approved fabricator in Section 1704.2.5. Ultimately, any approved changes in the requirements for approved fabricators should be evaluated for applicability to the approved erectors language.

Cost Impact: Will not increase the cost of construction
The current approach to approved fabricators in the IBC has a neutral impact on cost and may even reduce costs over the longterm. The owner who does not choose to use an approved fabricator must pay for third-party inspection. The owner who does use an approved fabricator is not required to hire a separate third-party inspector; the quality procedures and inspections performed by the fabricator are included in the cost of fabrication directly. Since the cost of doing it right the first time is thought to be less than the cost of after-the-fact inspection and subsequent repair, the approved fabricator approach likely costs less than the non-approved fabricator on average. Adding parallel provisions for an approved erector in the IBC is expected to have a similar minimal impact on cost.

S132-16 : 1704.2.6 (NEW)-MANLEY11462

S133-16

IBC: 1704.6, 1704.6.1, 1704.6.1 (New), 1704.6.2.

Proponent : Gregory Robinson, representing National Council of Structural Engineers Associations (grobinson@lbyd.com)

2015 International Building Code

Revise as follows:

1704.6 Structural observations. Where required by the provisions of Section 1704.6.1, 1704.6.2 or ~~1704.6.2~~ 1704.6.3, the owner or the owner's authorized agent shall employ a *registered design professional* to perform structural observations. Structural observation does not include or waive the responsibility for the inspections in Section 110 or the *special inspections* in Section 1705 or other sections of this code.

Prior to the commencement of observations, the structural observer shall submit to the *building official* a written statement identifying the frequency and extent of structural observations.

At the conclusion of the work included in the permit, the structural observer shall submit to the *building official* a written statement that the site visits have been made and identify any reported deficiencies that, to the best of the structural observer's knowledge, have not been resolved.

Add new text as follows:

1704.6.1 Structural observations for structures. Structural observations shall be provided for those structures where one or more of the following conditions exist:

1. The structure is classified as *Risk Category IV*.
2. The structure is a *high-rise building*.
3. The structure has an occupant load of more than 1000.
4. When so designated by the *registered design professional* responsible for the structural design.
5. When such observation is specifically required by the *building official*.

Revise as follows:

~~1704.6.1~~ 1704.6.2 Structural observations for seismic resistance. Structural observations shall be provided for those structures assigned to *Seismic Design Category* D, E or F where one or more of the following conditions exist:

1. The structure is classified as *Risk Category* III or IV.
2. The height of the structure is greater than 75 feet (22 860 mm) above the base as defined in ASCE 7.
3. The structure is assigned to *Seismic Design Category* E, is classified as *Risk Category* I or II, and is greater than two ~~*stories above*~~stories above the*grade plane*.
4. ~~When so designated by the *registered design professional* responsible for the structural design.~~
5. ~~When such observation is specifically required by the *building official*.~~

~~1704.6.2~~ 1704.6.3 Structural observations for wind ~~requirements~~ resistance. ~~Structural observations shall be provided for those structures sited where~~ *Structural observations shall be provided for those structures sited where* $V_{\cancel{asd}}$ $_{ult}$ ~~as determined in accordance with Section 1609.3.1 exceeds 110 mph~~ is 130mph (~~49~~ 58 m/sec) or greater, where one or more of the following conditions exist:

1. The structure is classified as *Risk Category* III or IV.
2. The~~*building height*~~ height of the structure is greater than 75 feet (22 860 mm) above the *grade plane*.
3. ~~When so designated by the *registered design professional* responsible for the structural design.~~
4. ~~When such observation is specifically required by the *building official*.~~

Reason: Currently the code requires structural observation only in the limited situations of tall buildings or higher risk category structures located in high seismic and w ind areas. It is the opinion of the National Council of Structural Engineers Associations that structural observation should be required for all large, or important, buildings anyw here in the country. It is w ell established that the quality of construction is increased w hen the engineer w ho designed the structure can verify that key construction conditions are in conformance w ith the design intent. Structural observation is meant to augment the detailed inspection provided by the special inspectors. It should be required w herever the consequence of structural failure is greater by virtue of complexity, size, occupancy, or risk.

Currently, a 7 story office building in San Francisco w ould require structural observation but a 60 story highrise or a 40000 seat stadium in New York w ould not. This proposal is intended to increase public safety by requiring that all similar structures are afforded the benefit of structural observation, not just the ones at risk of earthquakes or hurricanes. Therefore, Section 1704.6.1 is added for large or high risk buildings anyw here in the US.

In addition, the last w ord of the Wind section is changed to "resistance" to match the Seismic Section. Also in the Wind section, "building height" is changed to "height of structure" to match the Seismic section and to allow it to apply to all structures, not just buildings the same as the Seismic section.

In the Wind section, the w ind speed trigger is changed to 130 MPH to match the current factored level of w ind forces.

Cost Impact: Will not increase the cost of construction
The code change proposal w ill not increase the cost of construction. The may be a small increase to design fees if engineers request fee to cover this added service. How ever, it is generally held by many structural engineers that requirements stipulated by the building code w ill view ed as w ithin the normal scope of services therefore it is not anticipated that there w ill be a general increase in engineering fees resulting from this proposal.

S133-16 : 1704.6-ROBINSON11538

S134-16

IBC: 1704.6.2.

Proponent : Don Scott, representing National Council of Structural Engineering Associations (dscott@pcs-structural.com)

2015 International Building Code

Revise as follows:

1704.6.2 Structural observations for wind requirements. Structural observations shall be provided for those structures sited where $V_{\cancel{asd}\ \underline{ult}}$ ~~as determined in accordance with Section 1609.3.1 exceeds 110~~ is 130 mph (~~49~~ 58 m/sec) or greater, where one or more of the following conditions exist:

1. The structure is classified as *Risk Category* III or IV.
2. The *building height* is greater than 75 feet (22 860 mm).
3. When so designated by the *registered design professional* responsible for the structural design.
4. When such observation is specifically required by the *building official*.

Reason: Prior to IBC 2012, the 110 mph wind speed indicated in this section approximately aligned with the definition of wind borne debris regions. In IBC 2012, the definition of wind borne debris regions was adjusted to Vult = 130, along with changes to the wind maps. Vult is approximately Vasd = 100 mph. NCSEA believes that the wind speed trigger for structural observations needs to be adjusted in order to maintain the same level of structural observation in high wind areas that existed prior to IBC 2012. Therefore a wind speed trigger of Vult = 130 mph is recommended.

Cost Impact: Will not increase the cost of construction
This proposal will NOT increase construction costs. This is only a correction to a trigger that was made in previous editions of the IBC.

S134-16 : 1704.6.2-SCOTT12726

S135-16

IBC: 1705.2.

Proponent : Bonnie Manley, AISI, representing American Institute of Steel Construction (bmanley@steel.org)

2015 International Building Code

Revise as follows:

1705.2 Steel construction. The *special inspections* and nondestructive testing of steel construction in buildings, structures, and portions thereof shall be in accordance with this section.

> **Exception:***Special inspections* of the steel fabrication process shall not be required where the ~~fabricator~~ fabrication process for the entire building or structure does not ~~perform~~ include any welding, thermal cutting or heating operation of any kind ~~as part of the fabrication process~~. In such cases, the fabricator shall be required to submit a detailed procedure for material control that demonstrates the fabricator's ability to maintain suitable records and procedures such that, at any time during the fabrication process, the material specification and grade for the main stress-carrying elements are capable of being determined. Mill test reports shall be identifiable to the main stress-carrying elements when required by the *approvedconstruction documents*.

Reason: Because of the adoption of comprehensive quality assurance requirements for steel construction, through reference to AISC 360 Chapter N for example, this exception needs to be tightened considerably. The proposed modification focuses the exception on a full "project" rather than on individual "structural steel elements" . For entire projects that do not require any thermal cutting (such as beam copes), heat cambering, or welding (essentially no base plates), but need just plain material or simple bolted construction, this exception continues to waive special inspection and submittal of certificates of compliance. This exception, as now amended, would have limited use for very simple, small projects.
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Cost Impact: Will increase the cost of construction
This may increase the cost of construction where the exception was improperly implemented previously. However, it is really intended to be editorial in nature.

S135-16 : 1705.2-MANLEY12953

S136-16

IBC: 1705.3.

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

TYPE	CONTINUOUS SPECIAL INSPECTION	PERIODIC SPECIAL INSPECTION	REFERENCED STANDARD [a]	IBC REFERENCE
1. Inspect reinforcement, including prestressing tendons, and verify placement.	—	X	ACI 318 Ch. 20, 25.2, 25.3, 26.6.1-26.6.3	1908.4
2. Reinforcing bar welding:				
a. Verify weldability of reinforcing barsother than ASTM A 706;	—	X	AWS D1.4	
b. ~~Inspect single pass fillet welds, maximum 5/16"; 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.	X	~~X~~	ACI 318: 26.6.4	—
c. Inspect welding of shear reinforcement.	X	—		
~~c~~d. Inspect all other welds.	~~X~~	X		
3. Inspect anchors cast in concrete.	—	X	ACI 318: 17.8.2	—
4.Inspect anchors post-installed in hardened concrete members. [b]				

a. Adhesive anchors installed in horizontally or upwardly inclined orientations to resist sustained tension loads.	X		ACI 318: 17.8.2.4	—
b. Mechanical anchors and adhesive anchors not defined in 4.a				
5. Verify use of required design mix.	—	X	ACI 318: Ch. 19, 26.4.3, 26.4.4	1904.1, 1904.2, 1908.2, 1908.3
6. Prior to concrete placement, fabricate specimens for strength tests, perform slump and air content tests, and determine the temperature of the concrete.	X	—	ASTM C 172 ASTM C 31 ACI 318: 26.4, 26.12	1908.10
7. Inspect concrete and shotcrete placement for proper application techniques.	X	—	ACI 318: 26.5	1908.6, 1908.7, 1908.8
8.Verify maintenance of specified curing temperature and techniques.	—	X	ACI 318: 26.5.3-26.5.5	1908.9
9. Inspect prestressed concrete for:				
a. Application of prestressing forces; and	X	—	ACI 318: 26.10	—
b. Grouting of bonded prestressing tendons.	X	—		
10. Inspect erection of precast concrete members.	—	X	ACI 318: Ch. 26.9	—
11. Verify in-situ concrete strength, prior to stressing of tendons in post-tensioned concrete and prior to removal of shores and forms from	—	X	ACI 318: 26.11.2	—

beams and structural slabs.				
12. Inspect formwork for shape, location and dimensions of the concrete member being formed.	—	X	ACI 318: 26.11.1.2(b)	—

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.

TABLE 1705.2.2
REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD [a]
1. Material verification of cold-formed steel deck:			
a. Identification markings to conform to ASTM standards specified in the approved construction documents.	—	X	Applicable ASTM material standards
b. Manufacturers' certified test reports.	—	X	
2. Inspection of welding:	—	—	
a. Cold-formed steel deck			
1. Floor and roof deck welds		X	AWS D1.3
b. Reinforcing ~~steel~~ bars:			
1. Verification of weldability of reinforcing ~~steel~~ bars other than ASTM A 706.	—	X	AWS D1.4 ACI 318 Section 3.5.2
~~2. Reinforcing steel resisting flexural and axial forces in intermediate and special moment frames, and boundary elements of special structural walls of concrete and shear reinforcement.~~	~~X~~	~~—~~	
~~3. Shear reinforcement.~~	~~X~~	~~—~~	
~~4. Other reinforcing steel.~~	~~—~~	~~X~~	
2. Single-pass fillet welds, maximum 5/16"		X	
3. All other welds	X		

For SI: 1 inch = 25.4 mm.

a. Where applicable, see also Section 1705.11, Special inspection for seismic resistance.

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, w here w elding of reinforcing bars is not uncommon, should decrease modestly through elimination of unnecessary continuous special inspection in many cases.

S136-16 : TABLE 1705.3-GHOSH12750

S137-16

IBC: 1705.11.1, 1705.12.2, 1705.5, 1705.5 (New).

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 1705.5
REQUIRED SPECIAL INSPECTION OF WOOD CONSTRUCTION

TYPE	CONTINUOUS SPECIAL INSPECTION	PERIODIC SPECIAL INSPECTION
1. Verify that grade stamp on framing lumber, plywood and OSB panels conform to the approved construction documents.	-	X
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.	-	X
3. Verify that details of wood framing including framing layout, member sizes, blocking, bridging and bearing lengths conform to the approved construction documents.	-	X
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.	-	X

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. ~~*Continuousspecial 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.~~*Periodicspecial 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~~ 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~~ 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 w ood construction has become more commonly used for load bearing applications of significant height and in regions w ith moderate and high seismic and w ind 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 w ith w ood construction that is not subject to the same requirements for Special Inspection as the same buildings constructed w ith 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 tw o family dw ellings, 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 w ind resistance, and 1705.12 Special inspection for seismic resistance.

The proposed revisions to 1705.11 and 1705.12 are to coordinate betw een the additional requirements for Special Inspections in high seismic and high w ind 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 w ith 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 w here 1705.11 (w ind) 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 tw elve (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, tw enty tw o (22) items to for w elding, tw elve (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 betw een the systems w ith regard to Special Inspections; tw elve vs. forty four vs. zero.

Cost Impact: Will not increase the cost of construction
There w ill be no increase in construction cost due to the increased Special Inspection that w ill take place. Currently structural engineers provide for these inspections in project specifications. How ever, individual requirements vary greatly and there is not a consistent level of requirements. Standardization of these requirements in the Code w ill reduce delays and added costs due to confusion created by varying specifications. The improved field quality assurance w ill 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.

S137-16 : 1705.5-ROBINSON11665

S138-16

IBC: 1705.5.2.

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 trusse~~s 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 w ii harmonize the special inspection requirements for w ood trusses so that they have the same requirements as open w eb 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 w ith their responsibilities, determines if and w here an individual member of a truss needs to be braced to prevent out of plane buckling w hen subjected to design loads. This is w hat is called permanent individual truss member restraint/bracing. The locations w here this restraint/bracing is needed has historically been show n on the truss design draw ings as a small rectangle w ith an "x" thru it, along w ith 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 show n 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 w ood 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 w ill ensure the bracing gets installed w here it is required by the Truss designer.

One such collapse w as of a fire station in Pinetop Arizona, w here the permanent bracing w as not installed

.

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 w alls

.

A 2008 inspection of a building in Gig Harbor, WA show ed "piggy-back" trusses w ith missing bracing and collectors

.

Cost Impact: Will increase the cost of construction
The cost of construction w ill increase slightly by the amount of the cost of the special inspector. This increased cost how ever is minimal and justified.

S138-16 : 1705.5.2-HUSTON13310

S139-16

IBC: 1705.6, 1705.7, 1705.8, 1705.9.

Proponent : Mark Gilligan, representing Self (mark@gilligan.name); Woodward Vogt, Paradigm Consultants, Inc., representing GeoCoalition (woody@paradigmconsultants.com)

2015 International Building Code

Revise as follows:

1705.6 Soils. *Special inspections* and tests of existing site soil conditions, ~~fill placement~~ excavations, and ~~load-bearing requirements~~ fill shall be performed in accordance with this section and Table 1705.6. The *approved* ~~geotechnical report and the~~ *construction documents* ~~prepared by the *registered design professionals*~~ shall be used to determine compliance. ~~During fill placement, the special inspector shall verify that proper materials and procedures are used in accordance with the provisions of the *approved* geotechnical report.~~

> **Exception:** Where Section 1803 ~~does~~ or *approved construction documents* do not require reporting of materials and procedures for fill placement, the special inspector shall verify that the in-place dry density of the compacted fill is not less than 90 percent of the maximum dry density ~~at~~ within 2 percent of optimum moisture content determined in accordance with ASTM D 1557.

TABLE 1705.6
REQUIRED SPECIAL INSPECTIONS AND TESTS OF SOILS

TYPE	CONTINUOUS SPECIAL INSPECTION	PERIODIC SPECIAL INSPECTION
1. Verify materials below shallow foundations comply with the specified criteria. ~~are adequate to achieve the design bearing capacity.~~	—	X
2. Verify excavations are extended to ~~proper~~ specified depth and have reached ~~proper~~the specified material.	—	X
3. Perform classification and testing of ~~compacted~~ fill materials.	—	X
4. Verify use of ~~proper~~specified materials, densities and lift thicknesses during placement and compaction of ~~compacted~~ fill.	X	—
5. Prior to placement of ~~compacted~~ fill, inspect subgrade and verify that site has been prepared	—	X

~~properly~~ as specified.		

1705.7 Driven deep foundations. *Special inspections* and tests shall be performed during installation of driven deep foundation elements as specified in Table 1705.7. The approved ~~geotechnical report and the *construction documents* prepared by the *registered design professionals*~~ *construction documents* shall be used to determine compliance.

1705.8 Cast-in-place deep foundations. *Special inspections* and tests shall be performed during installation of cast-in-place deep foundation elements as specified in Table 1705.8. The *approved* ~~geotechnical report and the *construction documents* prepared by the *registered design professionals*~~ *construction documents* shall be used to determine compliance.

1705.9 Helical pile foundations. *Continuous special inspections* shall be performed during installation of helical pile foundations. The information recorded shall include installation equipment used, pile dimensions, tip elevations, final depth, final installation torque and other ~~pertinent~~ _installation data as required by the ~~*registered design professional in responsible charge*~~approved construction documents. The *approved* ~~geotechnical report and the *construction documents* prepared by the *registered design professional*~~ *construction documents* shall be used to determine compliance.

Reason: Owner Contractor agreements do not consider the geotechnical report to be a construction doeument. A key purpose of this change is to make the code provisions consistent with Section 107.4 which requires the work to be installed, and by inference inspected, in accordance with the construction doeuments. In addition this change is needed because the code is asking the geotechnical report to be something it is not. The content of geotechnical reports is inconsistent with their being used as construction documents, rather the appropriate provision in the geotechnical report need to be incorporated into the construction documents.

Elimination of the references to geotechnical reports is necessary because the authors of the geotechnical reports do not prepare them with the intention that they will be uses as the basis for inspections. Thus geotechnical reports do not have the information in the form that an inspector could use to perform inspections.

This change is necessary to make this section compatible with Section 107. This section makes it clear that the construction documents should completely define the work and the work shall be installed in accordance with the approved construciton documents. Similarly Section 1704.2.4 requires that inspector's reports "...shall indicate that work inspected was or was not completed in conformance to approved construction documents". This requires that the inspections be performed using the construction documents but Section 107.1, by listing both the geotechnical report and the construction documents, makes it clear that geotechnical reports are not construction doeuments. Thus it is inappropriate for the inspector to use the geotechnical report as a basis for inspection.

Owner Contractor agreements do not consider the geotechnical report to be a construction document.

Geotechnical reports are not prepared with the intention that they be used as construction documents and any attempt to make them fill this role would create ambiguity and expose the Owner to the potential for additional claims. Recognizing this SEAONC has issued "Guidelines for the use of Geotechnigal Reports" that clarifies the relationship between geotechnical reports and construction documents. These guidelines were prepared with input from several geotechnical engineering associations and thus reflects the recommendations of the geotechnical engineering community. http://www.seaonc.org/sites/default/files/content/guidance_for_geotech-reports-2015.pdf

The relevant portions of the geotechnical report needs to be incorporated into the plans and specificaitons which constitute the approved construction documents. This is similar to the process whereby the design professional incorporates code requirements in the construction documents.

For contractual reasons the geotechnical report is typically made available to the contractor to minimize the possibility of claims for changed field conditions but this is a contractual and not a building code consideration.

The function of the inspector is to verify that the w ork conforms to the approved construction documents and not to determine w hether the foundations are adequate to achieve the design bearing capacity. Determining w hether something is adequate is a job for the design professional w ho w il provide guidance for the inspector in the construction documents.

The w ords proper and properly are not w ell defined and have thus been deleted..

The phrase "prepared by the registered design professional" is redundant since licensing law s require the w ork be designed by a registered design professional.

The last sentence of Section 1705.6 is re dundant since the issues are addressed in Table 1705.6.

Cost Impact: Will not increase the cost of construction
The existing provisions create ambiguity w ith regards to w hat is required, thus as a result of the proposed changes there w ill likely be a reduciton in the cost of construction.

S139-16 : 1705.6-GILLIGAN11829

S140-16

IBC: 1705.7, 1705.8, 1705.9.

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

2015 International Building Code

Revise as follows:

1705.7 Driven deep foundations. *Special inspections* and tests shall be performed during installation of driven deep foundation elements as specified in Table 1705.7. The approved geotechnical report and the *construction documents* prepared by the *registered design professionals* shall be used to determine compliance.

> **Exception:** Driven deep foundations for ground-mounted *photovoltaic panel systems* with no use underneath.

1705.8 Cast-in-place deep foundations. *Special inspections* and tests shall be performed during installation of cast-in-place deep foundation elements as specified in Table 1705.8. The *approved* geotechnical report and the *construction documents* prepared by the *registered design professionals* shall be used to determine compliance.

> **Exception:** Cast-in-place deep foundations for ground-mounted *photovoltaic panel systems* with no use underneath.

1705.9 Helical pile foundations. *Continuous special inspections* shall be performed during installation of helical pile foundations. The information recorded shall include installation equipment used, pile dimensions, tip elevations, final depth, final installation torque and other pertinent installation data as required by the *registered design professional in responsible charge*. The *approved* geotechnical report and the *construction documents* prepared by the *registered design professional* shall be used to determine compliance.

> **Exception:** Helical pile foundations for ground-mounted *photovoltaic panel systems* with no use underneath.

Reason: A requirement for continuous Special Inspection for foundations for photovoltaic panel systems is overly restrictive. Large-scale (often called "utility scale") photovoltaic power plants often have tens of thousands of small piles. As project financing often involves third-party investors, existing measures of quality control are already in place. The developer and/or EPC (Engineer, Procure, Construct) contractor often use a rigorous design and testing process to optimize pile specifications, as part of value engineering. As part of their risk-management process, project financiers often use third-party Independent Engineers (IE's) to ensure quality controls are in place. Under current practice, it is extremely uncommon for local Building Officials to require Special Inspection for "deep" foundations for photovoltaic panel systems, regardless of the absence of an exception for these systems. Large-scale photovoltaic power plants usually incorporate rigorous design and quality control steps, as follows:

1. Foundation elements designed by analysis, based on geotechnical investigation.

2. As thousands of small piles are used in a photovoltaic power plant, optimization of design usually includes pre-construction pile load testing conducted on site. Independent Engineers (IE's) often review test reports.

3. EPC contractor has their own internal quality control.

4. A representative sample of production piles (for example, 1 percent) are usually proof-tested during construction, to ensure adequate pile capacities are being achieved. Adjustments are made if necessary to meet the demand.

5. County/AHJ inspectors usually conduct periodic observation of pile installation. For large-scale power plants, these inspectors are often third-party inspectors.

6. IE's usually conduct site visits to observe installation methods and review inspection reports and production pile load test reports. A final report is prepared by the IE.

Owing to this rigorous program of quality control, continuous special inspection of "deep" foundations is highly

redundant. A Special Inspector could be required to be on-site for one to three months watching piles being installed, even though the same piles are already being observed and monitored by the Developer, the EPC Contractor, the AHJ/County inspector, and the Independent Engineer.

For smaller installations -- such as residential ground-mounted photovoltaic panel systems -- continuous special inspection beyond the AHJ/County inspection adds project cost disproportionate to the risk to the project. Most AHJ/County Building Officials have agreed that special inspection is not necessary or reasonable for these small systems. This proposal formalizes the exemption that is commonly applied to small systems.

Cost Impact: Will not increase the cost of construction
This proposal will not increase the cost of construction, as continuous special inspection will no longer be a stated requirement for the small foundation piles used to support photovoltaic panel systems.

S140-16 : 1705.7-CAIN11049

S141-16

IBC: 1705.7.

Proponent : Mark Gilligan, representing Self (mark@gilligan.name)

2015 International Building Code

Revise as follows:

TABLE 1705.7
REQUIRED SPECIAL INSPECTIONS AND TESTS OF DRIVEN DEEP FOUNDATION ELEMENTS

TYPE	CONTINUOUS SPECIAL INSPECTION	PERIODIC SPECIAL INSPECTION
1. Verify element materials, sizes and lengths comply with the requirements.	X	—
2. ~~Determine capacities of test elements and conduct additional~~Verify the load tests, were performed as required and report the results.	X	—
3. Inspect driving operations and maintain complete and accurate records for each element.	X	—
4. Verify placement locations and plumbness, confirm type and size of hammer, record number of blows per foot of penetration, determine required penetrations to achieve design capacity, record tip and butt elevations and document any damage to foundation element.	X	—
5. For steel elements, perform additional special inspections in accordance with Section 1705.2.	—	—
6. For concrete elements and concrete-filled elements, perform tests and additional special inspections in accordance with Section 1705.3.	—	—
7. For specialty elements, perform additional inspections~~ as determined by the registered design professional in responsible charge~~listed in the statement of special inspections.	—	—

Reason: Inspectors monitor but do not directly perform the load tests of deep foundation eolements. The test asemblies are provided by the Contractor and tha actual applicaiton of the load tests is performed by the Contractor's personnel.

Determination of hte adequacy of load tests is a function of hte Responsible Design Professional. Inspectors are limited to reporting the test results and w hether the specified test criteria has been satisfied.

The text "as determined by the registered design professional" in item #7 is deleted since any additional special inspections are required to be listed in the statement of special inspections at the time of permit issuance. This change also addresses the additional special inspections required by the building official in accordance w ith Section 1705.1.1

Cost Impact: Will not increase the cost of construction
This change w ill not increase the construction costs since it provides clarity regarding the role of the special inspector and w ill not increase the time spent by the inspector.

S141-16 : TABLE 1705.7-GILLIGAN11883

S142-16

IBC: 1705.10 (New).

Proponent : Mark Gilligan, representing Self (mark@gilligan.name)

2015 International Building Code

Add new text as follows:

1705.10 Augered cast-In-place piles Continuous special inspections shall be performed during installation of augered cast-in-place piles. Verify use of the required grout design mix. Record and report results of load tests, pile installation procedures, installation sequence, auger diameter, depth of augered hole and tip elevation. size and length of reinforcement installed, grout volume and pressure snd sample grout to verify consistency and specified strength.

Reason: There is a need for special inspections of these critical foundation elements. There currently are no special inspection requirements for augered cast-in-place concrete piles.

Cost Impact: Will increase the cost of construction
Construction costs w ill increase in the amount of the inspection costs. Given that inspections are often required during the installation of augered cast-in-place piles this requirement may not result in any increase in cost of construction.

It should be noted that other deep foundation systems have comparable requirements.

S142-16 : 1705.10 (NEW)-GILLIGAN11875

S143-16

IBC: 1705.11 (New).

Proponent : Mark Gilligan (mark@gilligan.name)

2015 International Building Code

Add new text as follows:

<u>**1705.11 Micropiles** Continuous special inspections shall be performed during installation of micropiles. Verify use of the required grout design mix. Record and report type and size of reinforcement, mechanical couplers used and their location, length of steel installed, temporary or permanent casing, tip elevation, drilling mud used, flushing procedures, grout quantities and pumping pressures, and results of load and proof tests as required by the approved construction documents. Sample grout to verify consistency and specified strength.</u>

Reason: There is a need for special inspection of these critical foundation elements. There currently are no special inspection requirements for micropiles.

Cost Impact: Will increase the cost of construction
Construction costs w ill increase in the amount of the inspection costs. Given that inspections are often currently required during the installation of micro piles this requirement may not result in any cost of construction.

It should be noted that other comparable deep foundation systems have comparable requirements.

S143-16 : 1705.11 (NEW)-GILLIGAN11874

S144-16

IBC: 1705.11.

Proponent : Don Scott, representing National Council of Structural Engineering Associations (dscott@pcs-structural.com)

2015 International Building Code

Revise as follows:

1705.11 Special inspections for wind resistance. *Special inspections* for wind resistance specified in Sections 1705.11.1 through 1705.11.3, unless exempted by the exceptions to Section 1704.2, are required for buildings and structures constructed ~~in the following areas:~~

1. ~~In wind Exposure Category B, where~~ $\cancel{V_{asd}}$ ~~as determined in accordance with Section 1609.3.1 is 120 miles per hour (52.8 m/sec) or greater.~~
2. ~~In wind Exposure Category C or D, where~~ $\cancel{V_{asd}}$ ~~as determined in accordance with Section 1609.3.1 is 110 mph (49 m/sec) or greater.~~

<u>where V_{ult} is 130 mph (58 m/sec) or greater.</u>

Reason: Prior to IBC 2012, the wind speeds indicated in this section approximately aligned with the definition of wind borne debris regions. In IBC 2012, the definition of wind borne debris regions was adjusted to Vult = 130 and 140, along with changes to the wind maps. The new Vult is approximately Vasd = 100 and 110 mph. NCSEA believes that the wind speed trigger for special inspections should be adjusted in order to maintain the same level of special inspection in high wind areas that existed prior to IBC 2012, particularly for component and cladding elements. Further, it is recommended that a single wind speed be used for uniformity. A wind speed trigger of Vult = 130 mph is suggested.

Cost Impact: Will not increase the cost of construction
The is a change only to a Special Inspection trigger that was not applied correctly in previous editions of the code.

S144-16 : 1705.11-SCOTT12729

S145-16

IBC: 1705.11.1.

Proponent : Randy Shackelford (rshackelford@strongtie.com)

2015 International Building Code

Revise as follows:

1705.11.1 Structural wood. *Continuousspecial inspection* is required during field gluing operations of elements of the main windforce-resisting system. *Periodicspecial 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<u> and fasteners are installed in a single row</u>.

Reason: This proposal is meant to clarify w hen special inspection is required for shearw alls. It is possible that the designer may specify tw o row s of nails at 6" o.c. in an effort to circumvent the special inspection process.
In addition, there are types of w ood-framed shearw als that are designed to resist both shear and w ind uplift, that use nails in multiple row s. Special inspection of shearw alls that resist both shear and uplift from w ind are particularly important because this single system is being used as the main w ind-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 w ithout regard to w hether the nails are staggered or not, but this change w ill make sure it is interpreted that w ay.

Bibliography: These standards have requirements for nails in multiple row s for shearw alls resisting both shear and w ind uplift:

Special Design Provisions for Wood Construction (ANSI/AWC SDPWS-2015), American Wood Council, 2015, Page 39, Section 4.4.

w w w .aw c.org

Wood Frame Construction Manual (ANSI/AWC WFCM-2015), American Wood Council, 2015, Page 130, Figure 3.2f

w w w .aw c.org

Cost Impact: Will increase the cost of construction
It is possible that this proposal could increase the cost of construction if a designer w ere specifying fasteners in tw o row s at 6" o.c. in an effort to avoid special inspection. The additoinal cost w ould be the cost of special inspection, but this cost w oud be incurred only if no other triggers for special inspection w ere 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 row s. In this case, there w ould be no increase in construction.

S145-16 : 1705.11.1-SHACKELFORD13299

S146-16

IBC: 1705.12.1.1, 1705.12.1.2, 1705.13.1.1, 1705.13.1.2.

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~~ <u>in</u> 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.~~

<u>**Exceptions:**</u>

1. <u>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.</u>
2. <u>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.</u>

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.~~

<u>**Exceptions:**</u>

1. <u>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.</u>
2. <u>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</u>

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~~ 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.

> ~~**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 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~~ *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 less.~~

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 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-MANLEY12857

S147-16

IBC: 1705.12.6.

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. <u>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:</u>
    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.

<u>Where flexible sprinkler hose fittings are used, special inspection of minimum clearances is not required.</u>

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 NFPA 13 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.

S147-16 : 1705.12.6-GILLENGERTEN12294

S148-16

IBC: 1705.18 (New), 1705.18.1 (New), 1705.18.2 (New).

Proponent : Karl Rubenacker, Gilsanz Murray Steficek, representing Codes & Standards Committee, Structural Engineer's Association of New York (karl.rubenacker@gmsllp.com)

2015 International Building Code

Add new text as follows:

1705.18 Wall panels, curtain walls and veneers Special inspection is required for exterior architectural wall panels and the anchoring of veneers designed for installation on buildings above a height of 40 feet (12 192 mm). Special inspection of masonry veneer on such structures shall be in accordance with Section 1704.5.

Exceptions: Special inspection of wall panels, curtain walls and veneers shall not be required for:

1. Repairs and replacement in kind of gaskets or seals; or
2. Reglazing other than four-sided structural silicone glazing.

1705.18.1 Design and installation documents The special inspector shall become familiar with and retain a copy of the approved construction documents, and the following items, as applicable, approved by the registered design professional of record:

1. Shop drawings.
2. Instructions for the sequence of component installation.
3. Samples and/or mock-ups, if supplied.

1705.18.2 Inspection program The special inspector shall field check the site conditions at the time the structure is prepared for component installation, and periodically during component installation, to verify the following work is performed in compliance with the approved construction documents:

1. The supporting structure for components being inspected is aligned and within specified tolerances required for the components;
2. Required inserts are installed;
3. Framing components are installed and aligned as specified, and without structural defects or weakness;
4. Anchors are placed, welded, bolted and finished as specified, as applicable;
5. Weeps, flashings and tubes are installed as specified and functioning;
6. Joinery and end dams are sealed as specified;
7. Sealing materials with specified adhesive and movement capabilities are installed;
8. Gaskets , tapes, seals, insulation, flashing and other materials that are barriers to air and water movement, vapor drive, and heat loss are installed as specified;
9. Joint filler materials accommodate specified horizontal and vertical movement are installed in accordance with the manufacturers' instructions; and
10. Any other observations pertinent to safety of performance of the wall system.

Reason: Failure of w all panels, curtainw alls, and veneers in tall buildings have significant life safety consequences and hence their construction should be subject to special inspection.

Cost Impact: Will not increase the cost of construction
Will not increase the cost of construction, but w ill slightly increase cost of inspections.

S149-16

IBC: 1708.1, 1708.2, 1708.3, 1708.3.1, 1708.3.2.

Proponent : Gwenyth Searer, Wiss, Janney, Elstner Associates, Inc., representing self

2015 International Building Code

Revise as follows:

1708.1 General. 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~~ 1708.2 In-situ load tests. In-situ load tests shall be conducted in accordance with Section ~~1708.3.1~~ 1708.2.1 or ~~1708.3.2~~ 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~~ 1708.2.1 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 ~~1708.3.2~~ 1708.2.2 shall apply.

~~1708.3.2~~ 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 ~~a~~ 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 ~~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.

S149-16 : 1708.1-SEARER12058

S150-16

IBC: 1709.5.

Proponent : Jeff Inks, representing Window & Door Manufacturers Association (jinks@wdma.com)

2015 International Building Code

Revise as follows:

1709.5 Exterior window and door assemblies. The design pressure rating of exterior windows and doors in buildings shall be determined in accordance with Section 1709.5.1 or 1709.5.2. For ~~the purposes of this section~~ exterior windows and doors tested in accordance with Sections 1709.5.1 or 1709.5.2, ~~the~~ required design ~~pressure shall~~ wind presssures determined from ASCE 7 are permitted to be ~~determined using the allowable stress design load combinations of Section 1605.3~~ multiplied by 0.6.

> **Exception:** Structural wind load design pressures for window units smaller than the size tested in accordance with Section 1709.5.1 or 1709.5.2 shall be permitted to be higher than the design value of the tested unit provided such higher pressures are determined by accepted engineering analysis. All components of the small unit shall be the same as the tested unit. Where such calculated design pressures are used, they shall be validated by an additional test of the window unit having the highest allowable design pressure.

Reason: This proposal is intended to clarify that the use of the 0.6 conversion multiplier is allow ed w ith respect to the determination of design w ind pressures in accordance w ith ASCE 7 and testing of the respective assemblies in accordance w ith Sections 1709.5.1 and 1709.5.2 accordingly. While that is w hat the existing provision allow s, as currently w ritten, that is not entirely clear and has led to confusion regarding w ind load requirements. This proposed amendment expressly states that the use of 0.6 multiplier is allow ed and w ill alleviate the confusion that currently exists benefiting all – code officials, manufacturers and builders.

Cost Impact: Will not increase the cost of construction
This proposal is a clarification and not substantive. There are no new requirements.

S150-16 : 1709.5-INKS13401

S151-16

IBC: 1709.5, 1709.5.1, 1709.5.2, 1709.5.3 (New), 1709.5.3.1 (New).

Proponent : T. Eric Stafford, PE, representing Institute for Business and Home Safety

2015 International Building Code

Revise as follows:

1709.5 Exterior window and door assemblies. The design pressure rating of exterior windows and doors in buildings shall be determined in accordance with Section 1709.5.1 or 1709.5.2. For ~~the purposes of this section~~ exterior windows and doors tested in accordance with Sections 1709.5.1 and 1709.5.2, ~~the~~ required design ~~pressure shall~~ wind pressures determined from ASCE 7 are permitted to be ~~determined using the allowable stress design load combinations of Section 1605.3~~ multiplied by 0.6.

> **Exception:** Structural wind load design pressures for window units smaller than the size tested in accordance with Section 1709.5.1 or 1709.5.2 shall be permitted to be higher than the design value of the tested unit provided such higher pressures are determined by accepted engineering analysis. All components of the small unit shall be the same as the tested unit. Where such calculated design pressures are used, they shall be validated by an additional test of the window unit having the highest allowable design pressure.

1709.5.1 Exterior windows and doors. Exterior windows and sliding doors shall be tested and *labeled* as conforming to AAMA/WDMA/CSA101/I.S.2/A440. The *label* shall state the name of the manufacturer, the *approved* labeling agency and the product designation as specified in AAMA/WDMA/CSA101/I.S.2/A440. Exterior side-hinged doors shall be tested and *labeled* as conforming to AAMA/WDMA/CSA101/I.S.2/A440 or comply with Section 1709.5.2. Products tested and *labeled* as conforming to AAMA/WDMA/CSA 101/I.S.2/A440 shall not be subject to the requirements of Sections 2403.2 and 2403.3. Exterior windows and doors shall have a permanent label or marking applied in accordance with Section 1703.5.4 that provides traceability to the manufacturer and product.

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. Exterior windows and doors shall have a permanent label or marking applied in accordance with Section 1703.5.4 that provides traceability to the manufacturer and product.

Add new text as follows:

1709.5.3 Wind-borne debris protection. Protection of glazed openings in buildings located in windborne debris regions shall be in accordance with Section 1609.1.2.

1709.5.3.1 Fenestration assembly testing and labeling. Fenestration assemblies shall be tested by an approved independent laboratory, listed by an approved entity, and bear a label identifying manufacturer, performance characteristics, and approved inspection agency to indicate compliance with the requirements of the following specification(s):

1. ASTM E 1886 and ASTM E 1996; or

2. AAMA 506.

Fenestration assemblies shall have a permanent label or marking applied in accordance with Section 1703.5.4 that provides traceability to the manufacturer and product.

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
AAMA 506-11 Voluntary Specifications for Hurricane Impact and Cycle Testing of Fenestration Products

Reason: The primary purpose of this code change is to require that windows and 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. While the code does require a label that indicates the manufacturer, performance characteristics, and inspection agency to indicate compliance with AAMA/WDMA/CSA 101/I.S.2/A440, this label is not required to be permanent. Often, it is applied such that code enforcement can verify the appropriate performance characteristics required by the code, but is easily removable. 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 windows and doors after they've occupied the building. This proposal would simply require some type of permanent marking on the window or door indicating the manufacturer and model/series number so that the specific performance characteristics (wind, water infiltration, energy, etc.) could be retrieved. For clarity, language is also added addressing impact-resistant glazing that is similar to language found in the IRC. 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 with information needed to determine how critical components are expected to perform in the buildings they own or use. Windows and 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:

1710.5.1.1 Exterior windows and doors shall be labeled with a permanent label, marking, or etching providing traceability to the manufacturer and product. The following shall also be required either on a permanent label or on a temporary supplemental label applied by the manufacturer: information identifying the manufacturer, the product model/series number, positive and negative design pressure rating, product maximum size, glazing thickness, impact-resistance rating if applicable, Florida product approval number or Miami-Dade product approval number, applicable test standard(s), and approved product certification agency, testing laboratory, evaluation entity or Miami-Dade product approval.

This proposal also seeks to clarify the relationship between design wind loads calculated in accordance with ASCE 7 and the wind load testing requirements of AAMA/WDMA/CSA 101/I.S.2/A440. When the 2012 IBC and IRC were updated to reference ASCE 7-10, the codes did not address the conversion necessary for assemblies that are tested according to standards based on the ASD-level wind loads such AAMA/WDMA/CSA 101/I.S.2/A440. There was and continues to be much confusion in jurisdictions that have adopted the 2012 IBC and IRC. The 2015 IBC and IRC address the conversion but make a reference to the load combinations, which is not completely clear to those that are relatively unacquainted with the nuances of wind loading criteria. The 2010 Florida Codes adopted ASCE 7-10 and incorporated a specific reference to the 0.6 reduction similar to the language in this proposal. Jurisdictions that have adopted the 2012 IBC and IRC have often looked to the language in the 2010 FBC for justification of the conversion. The proposed language in Section 1709.5 doesn't result in any technical changes, but is simply a clarification consistent with the language used in the Florida Building Codes.

Approval of this proposal will assure, going forward, that new or replaced windows, doors and 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 of the building envelope or the products used to protect the building envelope in hurricane prone areas.

Cost Impact: Will increase the cost of construction
Will impact cost on some manufacturers. The code does not currently require a permanent label. However, many

window and door manufacturers voluntarily apply a permanent label that provides traceability to the manufacturer and performance characteristics. There will be no cost impact to those manufacturers.

Analysis: The standard proposed for inclusion in this code, AAMA 506, is referenced in the International Residential Code.

S151-16 : 1709.5-STAFFORD12298

S152-16

IBC: 1709.5.2.

Proponent : Mike Fischer, Kellen Company, representing the Plastic Glazing Coalition of the American Chemistry Council (mfischer@kellencompany.com)

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. <u>Plastic glazing in exterior window and door assemblies shall comply with Section 2608.</u> 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.

Reason: The current requirements for compliance to the NAFS standard for fenestration products includes a reference for glass to meet compliance of Section 2403. This proposal adds in a similar reference for plastic glazing, and the appropriate requirements of Chapter 26.

Cost Impact: Will not increase the cost of construction
The proposal is a clarification only; it does not add in any new requirements.

S152-16 : 1709.5.2-FISCHER3635

S153-16

IBC: 1709.5.2, 1709.5.3 (New).

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:

<u>**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.</u>

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 w ay for building ow ners, homeow ners, 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 ow ner to determine the structural w ind load resistance and/or energy efficiency of the garage doors after they've occupied the building. This proposal w ould 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 tow ards considering sustainability in the w ay our buildings are constructed in this country. As a result, building ow ners and occupants increasingly w ant more information about the sustainability of the buildings they occupy. Consequently, they need to be provided w ith w ays 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 w ind and w ater infiltration as w ell 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 follow ing 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 w ith 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 draw ing 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 w ill assure going forw ard that new or replaced doors w ill be labeled such that building ow ners and those considering the purchase of buildings w ith these products w ill 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.

S153-16 : 1709.5.2-STAFFORD12119

S154-16

IBC: 1709.5.3 (New), 1709.5.3.1 (New), 202 (New).

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 w ay for building ow ners, homeow ners, 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 ow ner to determine the structural w ind load resistance and impact resistance of the products after they've occupied the building. This proposal w ould 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 w ould 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 w as provided.
For the past 10-15 years, there has been a push tow ards considering sustainability in the w ay our buildings are constructed in this country. If this goal is to be successful and building ow ners and occupants increasingly w ant more information about the sustainability of the buildings they occupy, they need to be provided w ays 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 follow ing 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.

S154-16 : 1709.5.3 (NEW)-STAFFORD12151

S155-16

IBC: 1709.6.

Proponent : Mike Fischer, Kellen Company, representing the Plastic Glazing Coalition of the American Chemistry Council (mfischer@kellencompany.com)

2015 International Building Code

Revise as follows:

1709.6 Skylights and sloped glazing. Skylights and sloped glazing shall comply with the requirements of Chapter 24. Plastic Glazing in skylights and sloped glazing shall comply with Section 2610.

Reason: The IBC provides a reference to Chapter 24 for skylights and sloped glazing. This proposal adds a similar reference for plastic glazing to ensure that the user of the code recognizes the additional requirements in Chapter 26.

Cost Impact: Will not increase the cost of construction
The proposal is a clarification of existing requirements.

S155-16 : 1709.6-FISCHER3636

S156-16

IBC: 1801.3 (New).

Proponent : Donald Finocchio, representing Massachusetts Board of Building Regulations and Standards (don.finocchio@state.ma.us)

2015 International Building Code

Add new text as follows:

1801.3 Foundation types not covered by this section. Types of foundations not specifically covered by the provisions of this chapter, and ground modification treatments to improve soils with inadequate load bearing capacity or settlement characteristics, may be permitted subject to approval by the building official. A report shall be submitted to the building official that identifies the foundation as a type not covered by existing code provisions, and contains sufficient data and analyses to substantiate the adequacy of the proposed foundation. The report shall be prepared by a registered design professional knowledgeable in the design of the proposed type of foundation or ground modification. The building official may require that an independent peer review be performed to evaluate the adequacy of the proposed design.

Reason: This subsection allow s more design flexibility and w ill likely reduce cost.

Cost Impact: Will not increase the cost of construction
More flexibility in desing w ill likely reduce costs.

S156-16 : 1801.3 (NEW)-FINOCCHIO5056

S157-16

IBC: 1803.1, 1803.2, 202 (New).

Proponent : Woodward Vogt, Paradigm Consultants, Inc., representing GeoColaition (woody@paradigmconsultants.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Add new text as follows:

SECTION 202 DEFINITIONS

COMPRESSIBLE SOILS. Soils that exhibit volumetric reduction in response to the application of load even in the absence of wetting or drying.

Revise as follows:

1803.1 General. Geotechnical ~~investigations~~ engineering studies such as field exploration or testing, laboratory testing and engineering calculations, shall be conducted by a *registered design professional* in accordance with Section 1803.2 and reported in accordance with Section 1803.6. ~~Where required by the *building official* or where geotechnical investigations involve in-situ testing, laboratory testing or engineering calculations, such investigations shall be conducted by a *registered design professional*.~~

1803.2 ~~Investigations~~ Geotechnical engineering study required. ~~Geotechnical investigations~~ The *building official* shall be ~~conducted~~ permitted to require a geotechnical engineering study, including where the characteristics of the soil, such as moisture sensitive soils, *compressible soils* or liquifiable soils are of concern or where a lead-bearing value greater than that specified in ~~accordance with Sections 1803.3 through 1803.5~~ this code is claimed.

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

Reason: Click here to view the members of the GeoCoalition who developed this proposal.
The General subsection 1803.1 addresses how a "geotechnical investigation" is to be conducted and reported and who is to perform the work. To be consistent with another Proposal to change the title of Section 1803 to Geotechnical Engineering Studies, the new terminology has been used here.

The next subsection 1803.2 "Investigation required" uses essentially duplicative references and provides a caveat (exception for the *building official*). The two subsections can be combined to streamline the requirements avoiding duplications without losing any meaning or requirement.

In the "Exception", the adjective "satisfactory" is subjective and can be removed. There is no need to consider the alternative of unsatisfactory data. And again, the new terminology "Geotechnical engineering study(ies) is substituted.

Cost Impact: Will not increase the cost of construction
The change is a clarification so there is no change in construction requirements.

S157-16 : 1803.1-VOGT11998

S158-16

IBC: 1803.3, 1803.3.1.

Proponent : Woodward Vogt, Paradigm Consultants, Inc., representing GeoCoalition (woody@paradigmconsultants.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

1803.3 ~~Basis~~ Scope of ~~investigation.~~ geotechnical engineering study ~~Soil classification~~ The geotechnical engineering study shall ~~be based on observation and any necessary tests~~ address the adequacy of suporting soil or rock to provide a suitable factor of safety while controlling movement of the ~~materials disclosed by~~ foundations due to compression or expansion of supporting soils within tolerable limits for the structure. The scope of the geotechnical engineering study, including a review of site geology and history, site reconnaissance, the geotechnical exploration (the number, depth, and types of borings, test pits, or other subsurface ~~exploration made~~ sampling and testing) conducted in appropriate locations~~. Additional studies~~ , the equipment used, and the laboratory testing program performed on samples of the soil and/or rock, shall be ~~made as necessary to evaluate slope stability, soil strength, position and adequacy of load-bearing soils, the effect of moisture variation on soil-bearing capacity, compressibility, liquefaction and expansiveness~~ determined by a *registered design professional*.

Delete without substitution:

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

Reason: Click here to view the members of the GeoCoalition w ho developed this proposal.
The information in the revision to 1803.3 is more direct in addressing the scope and basis of the geotechnical engineering study. The information in the existing subsection 1803.3.1 has been included in the revised scope.

Cost Impact: Will not increase the cost of construction
The change is a clarification.

S158-16 : 1803.3-VOGT11999

S159-16

IBC: 1803.4.

Proponent : Woodward Vogt, Paradigm Consultants, Inc., representing GeoCoalition (woody@paradigmconsultants.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

1803.4 Qualified representative.

The ~~investigation~~*registered design professional* shall have a representative on site to supervise all exploration, testing, or sampling operations. The procedure and apparatus used in the geotechnical engineering study shall be in accordance with ~~generally accepted engineering practice~~ the standard of care as exercised by other *registered design professionals* providing similar services, under similar circumstances, and in the same locale, at the time the services are performed. ~~The *registered design professional* shall have a fully qualified representative on site during all boring or sampling operations.~~

Reason: Click here to view the members of the GeoCoalition who developed this proposal.
The requirement that the "procedure and apparatus" be in accordance with "generally accepted engineering practice" is another way of requiring the *registered design professional* to comply with the *standard of care,* which he/she is legally obligated to follow.

The second sentence requires the *registered design professional* to have a "fully-qualified representative" on but provides no requirements of the representative for one to judge his/her qualifications. First, a "qualified representative" should be just as qualified as a "fully-qualified representative" and thus the term "fully" can be deleted. Then would anyone believe that the *registered design professional's* representative could be "unqualified"? As far as the requirements of the representative, the representative should "supervise" the "exploration and sampling operations."

Cost Impact: Will not increase the cost of construction
The proposal is to clarify the responsibilities of the registered design professional's representative.

S159-16 : 1803.4-VOGT11986

S160-16

IBC: , 1803.5, 1803.5.1, 1803.5.2, 1803.5.3, 202 (New).

Proponent : Woodward Vogt, Paradigm Consultants, Inc., representing GeoCoalition (woody@paradigmconsultants.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Add new definition as follows:

SECTION 202 DEFINITIONS

COLLAPSIBLE SOILS. Soils that exhibit volumetric reduction in response to partial or full wetting under load.

SECTION 202 DEFINITIONS

EXPANSIVE SOILS. Soils that exhibit volumetric increase or decrease (swelling or shrinking) in response to partial or full wetting or drying under load.

Revise as follows:

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

1803.5.1 Soil and Rock Classification. ~~Soil materials~~ Soils shall be classified in accordance with ASTM ~~D 2487~~ D2487. Rock shall be classified in accordance with ASTM D5878.

1803.5.2 ~~Questionable soil~~ Soil and rock characteristics. Where the classification, strength, moisture sensitivity, or compressibility of the soil or rock is in doubt or where a load-bearing value superior to that specified in this code is claimed, the *building official* shall be permitted to require that a geotechnical ~~investigation~~ engineering study be conducted.

Delete and substitute as follows:

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

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

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

The geotechnical engineering study shall evaluate the presence of moisture sensitive soils, including *expansive soils* and *collapsible soils* that exhibit volumetric changes in response to wetting or drying under load. ASTM D4546 provides procedures for evaluating the expansion potential and the collapse of soils in response to full wetting under load.

1. The evaluation of volumetric change of *expansive soils* shall consider factors including

original material properties, the thickness of the layer, the surcharge loading on the layer, and the degree of wetting.

2. Evaluation of the *collapsible soils* shall consider the limitations of ASTM D4546 procedures or other procedures more appropriate for site-specific circumstances.

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:

ASTM D4546-14 Standard Test Methods for One-Dimensional Swell or Collapse of Soils[1]
ASTM D5878-08 Standard Guides for Using Rock-Mass Classification Systems for Engineering Purposes[1]

Reason: Click here to view the members of the GeoCoalition who developed this proposal.
The terminology of "geotechnical engineering study" has been used in lieu of "subsurface soil investigation," consistent with the proposed Section title change.

As the section following the new "Scope" section (1803.3), the purpose of this change is: 1) to add the standard method of classification of rock, and 2) to provide a broader explanation of soil and rock and moisture-sensitive soil conditions that must be evaluated as part of a geotechnical engineering study because of their potential to create very disruptive conditions that can affect earthwork and foundations both during and following construction.

Cost Impact: Will not increase the cost of construction
The change is a clarification so there will be no change to construction requirements.

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM D4546 & ASTM D5878, 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.

S160-16 : 1803.5-VOGT12006

S161-16

IBC: 1803.5.4.

Proponent : Woodward Vogt, Paradigm Consultants, Inc., representing GeoCoalition (woody@paradigmconsultants.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

1803.5.4 ~~Ground-water table~~ Groundwater. A ~~subsurface soil investigation~~ geotechnical engineering study shall be performed to determine ~~whether~~ the depth to the ~~existing ground-water~~ water table (phreatic surface) or to any perched water condition to determine if the depth is above or within 5 feet (~~1524 mm~~ 1.5 m) ~~below~~ of the elevation of the lowest floor level where such floor is located below the finished ground level adjacent to the foundation or if groundwater will affect earthwork or foundation construction.

> **Exception:** A ~~subsurface soil investigation~~ geotechnical engineering study to determine the location of the ~~ground-water table~~ groundwater shall not be required where ~~waterproofing is provided in accordance with Section 1805~~ available information confirms that groundwater will not adversely affect earthwork or foundations construction.

Reason: Click here to view the members of the GeoCoalition who developed this proposal.
The wording chage is to correct the general use of the hyphenated "ground-water" that is only appropriate when the term is used as two separate words that are being used as an adjective modifying, for example, "instrumentation." "Groundwater" as one word has become more commonly used.

Knowledge of the groundwater level, whether static or perched, is critical to determine whether groundwater may affect earthwork and foundation construction.

The terminology "geotechnical engineering study" has been used in lieu of "subsurface soil investigation."

If we follow the *Metrification Conversion Guide* of ACI 318M, 5 *feet* is appropriately converted to 1524 *mm* but it should then be rounded to two significant digits, e.g. 1500 *mm*. Then because it exceeds 1000 *mm*, it is expressed as *m*, e.g.1.5 *m.*

Finally, the existing **Exception** does not recognize the potential importance of groundwater on many elements of earthwork and foundation construction. Use of waterproofing does not address potential issues during construction due to the presence of water. Waterproofing is addressed in Section 1805.

Cost Impact: Will not increase the cost of construction
The proposed wording will better explain groundwater and clarify the potential effects on earthwork and foundation construction.

S161-16 : 1803.5.4-VOGT11992

S162-16

IBC: 1803.5.6.

Proponent : Woodward Vogt, Paradigm Consultants, Inc., representing GeoCoalition (woody@paradigmconsultants.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

1803.5.6 Rock strata. Where ~~subsurface explorations at the project site indicate variations~~ foundations can be supported on or in ~~the structure of~~ rock ~~upon which foundations are to be constructed~~, a sufficient number of borings shall be drilled to ~~sufficient~~ depths necessary to assess the competency of the rock and its load-bearing capacity.

Reason: Click here to view the members of the GeoCoalition who developed this proposal.
If foundations might be supported on or in rock, rock should be explored regardless of variability.

Cost Impact: Will not increase the cost of construction
The proposed change is a clarification only.

S162-16 : 1803.5.6-VOGT11996

S163-16

IBC: 1803.5.7.

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~~ for any foundation, a *registered design professional* shall prepare ~~an~~ a 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.

S163-16 : 1803.5.7-HUSTON13288

S164-16

IBC: 1803.5.10.

Proponent : Woodward Vogt, Paradigm Consultants, Inc., representing GeoCoalition (woody@paradigmconsultants.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

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

Reason: Click here to view the members of the GeoCoalition w ho developed this proposal.
The terminology of "geotechnical engineering study" has been used in lieu of "geotechnical investigation." Section 1803.1 already requires that the geotechnical engineering study be performed by a *registered design professional*.

Cost Impact: Will not increase the cost of construction
The change is a clarification.

S164-16 : 1803.5.10-VOGT11995

S165-16

IBC: 1803.5.11, 1803.5.12.

Proponent : Woodward Vogt, Paradigm Consultants, Inc., representing GeoCoalition (woody@paradigmconsultants.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

1803.5.11 Seismic Design Categories C through F. For structures assigned to *Seismic Design Category* C, D, E or F, a geotechnical ~~investigation~~ engineering study as required by Section 1803.3.4 shall be conducted, and shall include an evaluation of all of the following potential geologic and seismic hazards:

1. Slope instability.
2. Liquefaction.
3. Total and differential settlement.
4. Surface displacement due to faulting or seismically induced lateral spreading or lateral flow.

1803.5.12 Seismic Design Categories D through F. For structures assigned to *Seismic Design Category* D, E or F, the geotechnical ~~investigation~~ engineering study 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 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.

Reason: Click here to view the members of the GeoCoalition who developed this proposal.

Improve organization and provide terminology consistent with other proposed changes in terms.

Cost Impact: Will not increase the cost of construction
The proposal is a clarification.

S165-16 : 1803.5.11-VOGT11989

S166-16

IBC: 1803.5.12, 1809.13, 1810.3.11.2, 1810.3.12, 1810.3.6.1, 1810.3.9.4.

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_{DS}, 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 ~~12.4.3~~ 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~~ 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.

~~**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 12.4.3 or 12.14.3.2 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~~ 2.3.6 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~~ 2.3.6 or ~~12.14.3.2~~ 2.4.5 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~~ 2.3.6 or ~~12.14.3.2~~ 2.4.5 of ASCE 7.

Reason: This proposal is a coordination proposal to bring the 2018 IBC up to date w ith 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 ot Load Combinations including over-strength 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 w as underlined by the cdpAccesss system. The only change is to the ASCE 7 Section number.]

Cost Impact: Will not increase the cost of construction
The proposed changes w ill 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* w ill 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-

S167-16

IBC: 1804.1.

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

2015 International Building Code

Revise as follows:

1804.1 Excavation near foundations. Excavation for any purpose shall not reduce vertical or lateral support ~~from~~ for any foundation or adjacent foundation without first underpinning or protecting the foundation against detrimental lateral or vertical movement, or both.

Reason: Support of soil below foundations is required in all directions. The code notes that lateral support must be maintained, but if vertical support is reduced, the adjacent foundation will not have the required bearing.

Cost Impact: Will not increase the cost of construction
Most current practice currently follows this method, even though it is not clearly stated in the code. The cost of construction will not increase by specifying that vertical support must be maintained.

S167-16 : 1804.1-HUSTON13297

S168-16

IBC: 1804.1.

Proponent : theodore Maynard, Self empoyed Consultant (retired Chief Soils Engineer for the City of Chicago), representing GeoCoalition (trmagm@aol.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

1804.1 Excavation near foundations<u>, structures or infrastructure elements</u>. Excavation for any purpose shall not reduce lateral <u>or vertical</u> support from any <u>existing</u> foundation<u>, structure</u> or ~~adjacent foundation~~ <u>infrastructure element</u> without first underpinning or protecting the foundation <u>, structure or infrastructure element</u> against detrimental lateral or vertical movement, or both~~.~~ <u>*as determined by a registered design professional.*</u>

Reason: The proposal broadens the scope to include vertical support and to identify more elements of concern w ith regard to the possible risks associated w ith detrimental movement related to excavations and introduces the need for competent professional review before excavation begins.

Click here to view the members of the GeoCoalition w ho developed this proposal.

Cost Impact: Will not increase the cost of construction
This code change w ill not increase the cost of construction as successful excavation is of importance to reduce the need for costly repairs to adjacent properties; the cost of construction could possibly be reduced.

S168-16 : 1804.1-MAYNARD12503

S169-16

IBC: 1804.2.

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 <u>as permanent structural elements</u> in accordance with provisions of this chapter and Chapter 33 <u>and inspected in accordance with the provisions of Chapter 17</u>.

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.

S169-16 : 1804.2-HUSTON13303

S170-16

IBC: 1804.2.

Proponent : theodore Maynard, Self employed Consultant (retired Chief Soils Engineer of the City of Chicago), representing GeoCoalition (trmagm@aol.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

~~1804.2~~ <u>1804.3</u> **Underpinning.** Where underpinning is chosen to provide the protection or support of adjacent <u>foundations,</u> structures <u>or infrastructure elements</u>, the underpinning system shall be designed *<u>by a registered design professional</u>* and installed in accordance with provisions of this chapter and Chapter 33.

Reason: This proposal broadens the scope of possible adjacent property that might require underpinning and requires the use of a registered design professional.
Click here to view the members of the GeoCoalition w ho developed this proposal.

Cost Impact: Will not increase the cost of construction
This proposal w ill not increase the cost of construction. It reflects the typical current practice.

S170-16 : 1804.2-MAYNARD12521

S171-16

IBC: 1804.3.

Proponent : theodore Maynard, representing GeoCoalition (trmagm@aol.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

~~**1804.3**~~ **1804.4 Placement of backfill.** The excavation outside the foundation and foundation wall shall be backfilled with soil that is free of organic material, construction debris, cobbles and boulders , treated soil, or with a controlled low-strength material (CLSM). The backfill shall be placed in lifts and compacted or the CLSM allowed to set in a manner that does not damage the foundation or the waterproofing or dampproofing material.

~~**Exception:** CLSM need not be compacted.~~

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

Reason: Added foundation w all to broaden the application of this section. As treated soil and CLSM are being used more frequently, this proposal is to allow for use of treated soil and to clarify the requirements for using CLSM during backfilling outside the foundation and/or foundation w all. Accounting for settlement of the backfill w as moved to this section from "Site Grading" because it applies directly to backfilling.

Click here to view the members of the GeoCoalition w ho developed this proposal.

Cost Impact: Will not increase the cost of construction
This code change w ill not increase the cost of construction. Possible reduction by allow ing use of treated soil.

S171-16 : 1804.3-MAYNARD12681

S172-16

IBC: 1804.3.2 (New).

Proponent : theodore Maynard, Self employed Consultant (retired Chief Soils Engineer of the City of Chicago, representing GeoCoalition (trmagm@aol.com)

2015 International Building Code

Add new text as follows:

1804.3.2 Underpinning monitoring During installation of the underpinning system, the elements being underpinned and adjacent foundations, structures or infrastructure elements shall be monitored at the frequency determined by *a registered design professional* to determine lateral and vertical movements. Installation procedures shall be immediately modified if the *registered design professional* determines that the movements are detrimental.

Reason: It is important to monitor the effectiveness of the underpinning operation during its installation, along with its impact on adjacent property.

Click here to view the members of the GeoCoalition who developed this proposal.

Cost Impact: Will not increase the cost of construction
This code change proposal will not increase the cost of construction. This code change reflects the typical current practice.

S172-16 : 1804.3.2 (NEW)-MAYNARD12524

S173-16

IBC: 1804.4.

Proponent : theodore Maynard, representing GeoCoalition (trmagm@aol.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

~~1804.4~~ 1804.5 Site grading. The ground immediately adjacent to the foundation or foundation wall shall be sloped away from the building at a slope of not less than one unit vertical in 20 units horizontal (5-percent slope) for a minimum distance of 10 feet (3048 mm) measured perpendicular to the face of the wall. If physical obstructions or lot lines prohibit 10 feet (3048 mm) of horizontal distance, a 5-percent slope shall be provided to an *approved* alternative method of diverting water away from the foundation and foundation wall. Swales used for this purpose shall be sloped a minimum of 2 percent where located within 10 feet (3048 mm) of the ~~building~~ foundation or foundation wall. Impervious surfaces within 10 feet (3048 mm) of the ~~building~~ foundation or foundation wall shall be sloped a minimum of 2 percent away from the building.

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

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

Reason: Added foundation w all to broaden the application of this section. Accounting for settlement of the backfill w as moved to the "Placement of Backfill" section because it applies directly to backfill.

Click here to view the members of the GeoCoalition w ho developed this proposal.

Cost Impact: Will not increase the cost of construction
Adding the tern "foundation w all" is a clarification to include the structure's complete perimeter and w ill not affect the cost of construction. The deleted sentence regarding the "accounting for the settlement of backfill" has been included, verbatim, in the proposed new Section 1804.4 "Placement of Backfill"; therefore, this does not change the existing code requirements and w ill not increase the cost of construction.

S173-16 : 1804.4-MAYNARD12691

S174-16

IBC: 1804.4.

Proponent : Eirene Oliphant, BRR Architecture, representing BRR Architecture (eirene.oliphant@brrarch.com)

2015 International Building Code

Revise as follows:

1804.4 Site grading. The ground immediately adjacent to the foundation shall be sloped away from the building at a slope of not less than one unit vertical in 20 units horizontal (5-percent slope) for a minimum distance of 10 feet (3048 mm) measured perpendicular to the face of the wall. If physical obstructions or lot lines prohibit 10 feet (3048 mm) of horizontal distance, a 5-percent slope shall be provided to an *approved* alternative method of diverting water away from the foundation. Swales used for this purpose shall be sloped a minimum of 2 percent where located within 10 feet (3048 mm) of the building foundation. Impervious surfaces within 10 feet (3048 mm) of the building foundation shall be sloped a minimum of 2 percent away from the building<u>, except as otherwise permitted in Section 1010.1.5, 1012.3 or 1012.6.1</u>.

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

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

Reason: While the intent of this section is to require slope away from the building to allow for proper water drainage, it does not account for walking surfaces, door landings or ramp landings adjacent to a building to have a maximum cross slope of two percent. This leaves no room for error for construction purposes to provide not only drainage at a minimum of two percent but also the cross slope of no more than two percent. Designers often choose a cross slope of less than two percent in these areas, which according to this section, would not be compliant for site grading.

Cost Impact: Will not increase the cost of construction
The current code language needs clarification to acknowledge the cross slope for accessibility in terms of site grading. Changing the language to acknowledge this will not affect the construction cost.

S174-16 : 1804.4-OLIPHANT11348

S175-16

IBC: 1804.5.

Proponent : theodore Maynard, representing GeoCoalition (trmagm@aol.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

~~1804.5~~ <u>1804.6</u> Grading and fill in flood hazard areas. In *flood hazard areas* established in Section 1612.3, grading, fill, or both, shall not be *approved*:

1. Unless such fill is placed, compacted and sloped to minimize shifting, slumping and erosion during the rise and fall of flood water and, as applicable, wave action.
2. In floodways, unless it has been demonstrated through hydrologic and hydraulic analyses performed by a *registered design professional* in accordance with standard engineering practice that the proposed grading or fill, or both, will not result in any increase in flood levels during the occurrence of the *design flood*.
3. In *coastal high hazard areas*, unless such fill is conducted and/or placed to avoid diversion of water and waves toward any building or structure.
4. Where design flood elevations are specified but floodways have not been designated, unless it has been demonstrated <u>by a *registered design professional*</u> that the cumulative effect of the proposed *flood hazard area* encroachment, 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.

Reason: Inclusion of evaluation by a registered design professional w here one is necessary.
Click here to view the members of the GeoCoalition w ho developed this proposal.

Cost Impact: Will not increase the cost of construction
The inclusion of "a registered design professional" to demonstrate that flood hazard area encroachment w ill not raise the design flood elevation by more than 1 foot is a safety issue; reflects typical current practice and w ill not increase the cost of construction.

S175-16 : 1804.5-MAYNARD12701

S176-16

IBC: 1804.6.

Proponent : Gerald Gunny, City of Henderson Department of Building and Safety, representing Southern Nevada Chapter of the International Code Council

2015 International Building Code

Revise as follows:

1804.6 Compacted fill material. Where shallow foundations will bear on compacted fill material, the compacted fill shall comply with the provisions of an *approved* geotechnical report, as set forth in Section 1803.

> **Exception:** ~~Compacted fill material 12 inches (305 mm) in depth or less need~~ When a geotechnical investigation is not ~~comply with an *approved* report~~ required, ~~provided~~ the in-place dry density ~~is~~ within the building pad shall not be less than 90 percent of the maximum dry density at near optimum moisture content for granualr soils and at or above optimum moisture content for cohesive soils determined in accordance with ASTM D 1557. The compaction shall be verified by *special inspection* in accordance with Section 1705.6.

Reason: The exception w as amended because the approved geotechnical report should specify all compaction requirements. The amended exception is for w hen the building official w aives the requirement for a geotechnical investigation. If no geotechnical investigation is required then the minimum level of compaction shall be verified by special inspection.

Cost Impact: Will not increase the cost of construction
The proposal merely clarifies the compaction requirements for projects exempted from a geotechnical investigation and w ill not impact the cost.

S176-16 : 1804.6-GUNNY11596

S177-16

IBC: 1805.4.2.

Proponent : Donald Finocchio, Mass. Dept. of Public Safety, representing Board of Building Regulations and Standards

2015 International Building Code

Revise as follows:

1805.4.2 Foundation drain. A drain shall be placed around the perimeter of a foundation that consists of gravel or crushed stone containing not more than 10-percent material that passes through a No. 4 (4.75 mm) sieve. The drain shall extend a minimum of 12 inches (305 mm) beyond the outside edge of the footing. The thickness shall be such that the bottom of the drain is not higher than the bottom of the base under the floor, and that the top of the drain is not less than 6 inches (152 mm) above the top of the footing. The top of the drain shall be covered with an *approved* filter membrane material. Where a drain tile or perforated pipe is used, the invert of the pipe or tile shall not be higher than the floor elevation. The top of joints or the top of perforations shall be protected with an *approved* filter membrane material. The pipe or tile shall be placed on not less than 2 inches (51 mm) of gravel or crushed stone complying with Section 1805.4.1, and shall be covered with not less than 6 inches (152 mm) of the same material.

> <u>**Exception**. The foundation drain shall not be required where determined not to be necessary by a registered design professional.</u>

Reason: Adds flexibility that may reduce cost.

Cost Impact: Will not increase the cost of construction
Adds flexibility that may reduce cost.

S177-16 : 1805.4.2-FINOCCHIO5378

S178-16

IBC: 1805.5 (New).

Proponent : Donald Finocchio, Mass. Dept. of Public Safety, representing Board of Building Regulations and Standards

2015 International Building Code

Add new text as follows:

1805.5 Impacts on groundwater levels. Below-grade structures, their appurtenances and foundation drains shall be designed and constructed so as not to cause changes to the temporary or permanent groundwater level where such changes could adversely impact nearby structures or facilities including deterioration of timber piles, settlement, flooding or other impacts.

Reason: Provides clarity and emphasis to ensure w ork doesn't impact nearby structures.

Cost Impact: Will increase the cost of construction
May be significant cost saver as it may protect adjacent structures from adverse impacts.

S178-16 : 1805.5 (NEW)-FINOCCHIO5387

S179-16

IBC: 1806.2.

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, organic silt or unprepared fill is adequate for the support of lightweight or temporary structures.~~

> **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.

Reason: There should be a requirement that an approved method of anlaysis be used to substantitate the data submitted for approval.
Mud is not a reconized geotechnical "CLASS OF MATERIAL". A disposition is also not a recognized geotechnical term for use in determining soil classificaiton.

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 w ith subsurface explorations and in situ testing. Allow ing structures to be supported on undocumented fill is not in accordance w ith generally accepted engineering practice. Fill shall be engineered in accordance w ith Section 1804.5

Click here to view members of the GeoCoalition w ho prepared this proposal.

Cost Impact: Will not increase the cost of construction
This code change w ill not inclurease the cost of construction because it is the standard of practice.

S179-16 : 1806.2-SIMPSON12859

S180-16

IBC: 1806.3 (New).

Proponent : Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Add new text as follows:

<u>1806.3</u> **<u>Settlement</u>** <u>Settlement shall be considered in accordance with Section 1808.2 .</u>

Reason: Reference to settlement in Section 1806 Presumptive Load-Bearing Values of Soils bring consistency w ith Section 1808 Foundations in that allow able bearing capacity should not exceed limiting differential settlements.

[Click here to view the members of the GeoCoalition w ho prepared this proposal.]

Cost Impact: Will not increase the cost of construction
There is not cost increase because it just references another section of the code.

S180-16 : 1806.3 (NEW)-SIMPSON12863

S181-16

IBC: 1806.3.4.

Proponent : Donald Finocchio, Mass. Dept. of Public Safety, representing Board of Building Regulations and Standards

2015 International Building Code

Revise as follows:

1806.3.4 Increase for poles. Isolated poles for uses such as flagpoles or signs and poles used to support buildings that are not adversely affected by a $^{1}/_{2}$-inch (12.7 mm) motion at the ground surface due to short-term lateral loads shall be permitted to be designed using lateral bearing pressures equal to two times the tabular values of Table 1806.2.

Reason: Adds clarity.

Cost Impact: Will not increase the cost of construction
Adds clarity - no technical change.

S181-16 : 1806.3.4-FINOCCHIO5407

S182-16

IBC: 1807.1.6.

Proponent : Scott DiFiore, representing GeoCoalition (sjdifiore@sgh.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

1807.1.6 Prescriptive design of concrete and masonry foundation walls. Concrete and masonry foundation walls that are laterally supported at the top and bottom shall be permitted to be designed and constructed in accordance with this section, provided that they are not subject to net hydrostatic pressures or surcharge loadings, and the backfill adjacent to the walls is not subject to heavy compaction loads.

Reason: This proposal provides limitations to the prescriptive design of concrete and masonry foundation w alls. The additional language highlights loading conditions that are not captured by the design lateral soil loads provided in Section 1610.
Click here to view the members of the GeoCoalition w ho developed this proposal

Cost Impact: Will not increase the cost of construction
No increase. The code change proposal w ill not increase the cost of construction for w alls that are not subjected to w ater pressures, surcharges, or heavy compaction loads. The prescriptive design does not change.

S182-16 : 1807.1.6-DIFIORE12824

S183-16

IBC: 1807.2.

Proponent : Scott DiFiore, representing GeoCoalition (sjdifiore@sgh.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

1807.2 Retaining walls. Retaining walls shall be designed by a registered design professional in accordance with Sections 1807.2.1 through 1807.2.3. Retaining structures include, but are not necessarily limited to, conventional unbraced retaining walls, crib and bin wall systems, mechanically stabilized earth systems, multitiered systems, anchored walls, soil nail walls, or other types of retaining structures. The requirements of this section do not apply to facings whose purpose is only to protect against surface erosion.

Reason: Retaining w alls include various types of systems, and this clarification alerts designers and code officials to many of these systems to make sure they are designed by a registered design professional. An exception is provided to clarify that protection for surface erosion is not a retaining w all.
Click here to view the members of the GeoCoalition w ho developed this proposal.

Cost Impact: Will not increase the cost of construction
The code change reflects current practice and adds no new requirements or restrictions.

S183-16 : 1807.2-DIFIORE12387

S184-16

IBC: 1807.2.1.

Proponent : Scott DiFiore, representing GeoCoalition (sjdifiore@sgh.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

1807.2.1 ~~General.~~ Design Retaining walls shall be designed to resist the pressures of the retained materials, water pressures, and dead and live load surcharges to which the walls are subjected, and to ensure stability against overturning, sliding, excessive foundation pressure and water uplift. Where a keyway is extended below the wall base with the intent to engage passive pressure and enhance sliding stability, lateral soil pressures on both sides of the keyway shall be considered in the sliding analysis.

Reason: The retaining w all designer must consider all loads that influence the behavior of the retaining w all. This proposal provides examples, beyond lateral earth pressures, that require consideration by the designer.
Click here to view the members of the GeoCoalition w ho developed this proposal

Cost Impact: Will not increase the cost of construction
This code change proposal reflects current practice and does not add new requirements for design of retaining w alls.

S184-16 : 1807.2.1-DIFIORE12828

S185-16

IBC: 1807.2.1.

Proponent : Mark Swanson, representing Swanson & Associates Engineering (ms@swanson-engr.com)

2015 International Building Code

Revise as follows:

1807.2.1 General. Retaining walls shall be designed to ensure stability against overturning, sliding, excessive foundation pressure and water uplift.~~ Where a keyway is extended below the wall base with the intent to engage passive pressure and enhance sliding stability, lateral soil pressures on both sides of the keyway shall be considered in the sliding analysis.~~

Reason: The application of soils pressure on both sides of a keyw ay is a recent addition to the model codes, but is just now gaining attention and opposition by the geotechnical engineering community. The concept is in conflict w ith accepted engineering practice, and the principals of soils mechanics. The existing code language does not adequately describe the intended loading model. The existing code language is vague and ambiguous w ith respect to the loading on the opposite side of the passive pressure side of the keyw ay, w hich has resulted in overly conservative design assumptions.
The application of "lateral earth pressures on both sides of the keyw ay" is commonly interpreted to require a deepening of the active soil pressure to the bottom of the keyw ay. Active soil pressure requires movement of the key, w hich is contrary to the accepted design model.

The phrase "shall be considered in the design" has resulted on some soils engineers and geologists adding the follow ing phrase: "The mechanics of soils pressure on keyw ays has been considered. The active pressure on the keyw ay shall be taken as zero", a testement to the disagreement over the code language.

Cost Impact: Will not increase the cost of construction
Construction costs w ill nominally decrease. Design costs w ill nominally decrease.

S185-16 : 1807.2.1-SWANSON11901

S186-16

IBC: 1807.2.2.

Proponent : Scott DiFiore, representing GeoCoalition (sjdifiore@sgh.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

1807.2.2 Design lateral soil loads. Retaining walls shall be designed for the lateral soil loads set forth in Section 1610. For structures assigned to Seismic Design Category D, E, or F, the design shall incorporate the seismic lateral earth pressure in accordance with the approved geotechnical report.

Reason: Retaining w all design must consider seismic loads in locations w here the risk of seismic activity is high.
Click here to view the members of the GeoCoalition w ho developed this proposal.

Cost Impact: Will not increase the cost of construction
The code change reflects current practice.

S186-16 : 1807.2.2-DIFIORE12832

S187-16

IBC: 1807.2.3.

Proponent : Ali Fattah, City of San Diego Development Services Department (afattah@sandiego.gov)

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, 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 or wind loads are included, the minimum safety factor for retaining wall sliding and overturning shall be 1.1.

Reason: The IBC requires that retaining w all designs include a stability check for sliding and overturning failure and that a factor of safety be applied. Additionally the code includes an exception that addresses stability checks for w alls designed to include load combinations that include earthquake loads. The code how ever is silent in regards to w ind loads. There are cases w here the construction of a retaining w all extends above the grade of the retained earth or w here fences that are directly directly supported on retaining w alls w here the surface area is such that w ind load is more significant than the earthquake loads due to the w eight of the projecting element.
This code change is editorial in nature and addresses an anomaly in the exception. Both earthquake and w ind loads are lateral loads and generally factors of safety in the IBC for both are the same. Additionally the load combinations in IBC Section 1605 for strength design use the multiplier 1 for E and W and 1.7 for H the load due to lateral earth pressures. Generally the issue is the same for w orking stress and the alternative basic load combinations.

Cost Impact: Will not increase the cost of construction
The code change is editiorial.

S187-16 : 1807.2.3-FATTAH11495

S188-16

IBC: 1807.2.3, 1807.2.4 (New), 1807.2.5 (New), 1807.2.6 (New).

Proponent : Donald Finocchio, Mass. Dept. of Public Safety, representing Board of Building Regulations and Standards

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 safety 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, 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 of safety for retaining wall sliding and overturning shall be 1.1.

Add new text as follows:

1807.2.4 Overall stability. The overall global stability of a retaining wall, considering potential failure surfaces extending through the materials located below, in front of and behind the wall shall be evaluated.

1807.2.5 Discrete elements. For retaining walls constructed of discrete elements, such as unmortared masonry, rock, boulders, or stacked modular units, the elements shall be bonded or fastened together to prevent dislodgement under static and seismic loading conditions where dislodgement of the elements could pose a risk to public safety.

1807.2.6 Wall drainage. Retaining walls shall be designed to support a hydrostatic head of water pressure equal to the full height of the wall, unless a drainage system is provided to reduce or eliminate hydrostatic pressure on the wall. Drainage systems shall be designed with sufficient permeability and discharge capacity, and shall be provided with appropriate filters and other design features to prevent blockage due to siltation, clogging, or freezing.

Reason: This proposal provides clarity on the design requirements of retaining w alls and offers design flexibility.

Cost Impact: Will not increase the cost of construction
This proposal provides clarity on the design requirements of retaining w alls and offers design flexibility.

S188-16 : 1807.2.3-FINOCCHIO5408

S189-16

IBC: 1807.2.3.

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.

S189-16 : 1807.2.3-GUNNY11603

S190-16

IBC: 1807.2.4 (New).

Proponent : Scott DiFiore, representing GeoCoalition (sjdifiore@sgh.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Add new text as follows:

1807.2.4 Overall Stability The overall global stability of all types of retaining structures shall be evaluated where considered appropriate by the *registered design professional*.

Reason: This additional provision alerts the designer to a critical stability check required for retaining w all design, w hich is not mentioned in 1807.2.1.
Click here to view the members of the GeoCoalition w ho developed this proposal.

Cost Impact: Will not increase the cost of construction
The code change reflects current practice.

S190-16 : 1807.2.4 (NEW)-DIFIORE12834

S191-16

IBC: 1807.2.5 (New).

Proponent : Scott DiFiore, representing GeoCoalition (sjdifiore@sgh.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Add new text as follows:

1807.2.5 Discrete elements. For retaining walls constructed of discrete elements, such as mortarless masonry, rock, boulders, or stacked modular units, the elements shall have a load-transfer mechanism that prevents dislodgement.

Reason: Many retaining w alls are constructed w ith discrete elements w ith differing load-transfer or attachment mechanisms that are at risk of failure. This provision alerts the designer to a failure mode that needs consideration as part of design and construction.
Click here to view the members of the GeoCoalition w ho developed this proposal

Cost Impact: Will not increase the cost of construction
The code change proposal w ill not increase the cost of construction. The code change proposal reflects current practice.

S191-16 : 1807.2.5 (NEW)-DIFIORE12839

S192-16

IBC: 1807.2.6 (New).

Proponent : Scott DiFiore, representing GeoCoalition (sjdifiore@sgh.com); Lori Simpson, P.E.,G.E., representing GeoCoalition

2015 International Building Code

Add new text as follows:

1807.2.6 Wall drainage. Retaining walls shall be designed to consider water pressures, unless a drainage system is provided to reduce or eliminate water pressures that act on the wall. Drainage systems shall be provided with appropriate filters and other features to prevent blockage.

Reason: Water adds pressure to the retaining w all and reduces soil strength, and therefore w ater control affects w all performance.
Click here to view the members of the GeoCoalition w ho developed this proposal

Cost Impact: Will not increase the cost of construction
The code change proposal w ill not increase the cost of construction. The code change proposal reflects current practice.

S192-16 : 1807.2.6 (NEW)-DIFIORE12841

S193-16

IBC: 1808.2.

Proponent : Donald Finocchio, Mass. Dept. of Public Safety, representing Board of Building Regulations and Standards

2015 International Building Code

Revise as follows:

1808.2 Design for capacity and settlement. Foundations shall be so designed that the allowable bearing capacity of the soil is not exceeded, and that differential settlement is minimized to provide adequate load bearing capacity while limiting settlement, heave and lateral movement to tolerable levels. Foundations in areas with expansive soils shall be designed in accordance with the provisions of Section 1808.6.

Reason: Adds specificity.

Cost Impact: Will not increase the cost of construction
Adds specificity.

S193-16 : 1808.2-FINOCCHIO5423

S194-16

IBC: 1808.3, 1810.2.4.1.

Proponent : Ronald Hamburger, SIMPSON GUMPERTZ & HEGER, representing SELF (rohamburger@sgh.com)

2015 International Building Code

Revise as follows:

1808.3 Design loads. Foundations shall be designed for the most unfavorable effects due to the combinations of loads specified in ASCE 7 Sections 2.3 or 2.4 or Section 1605.2 ~~or 1605.3~~. The dead load is permitted to include the weight of foundations and overlying fill. Reduced live loads, as specified in Sections ~~1607.10~~ 1607.6 and ~~1607.12~~ 1607.7, shall be permitted to be used in the design of foundations.

1810.2.4.1 Seismic Design Categories D through F. For structures assigned to *Seismic Design Category* D, E or F, deep foundation elements on *Site Class* E or F sites, as determined in ~~Section 1613.3.2~~ accordance with ASCE 7 Chapter 20, shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains modified for soil-foundation-structure interaction coupled with foundation element deformations associated with earthquake loads imparted to the foundation by the structure.

> **Exception:** Deep foundation elements that satisfy the following additional detailing requirements shall be deemed to comply with the curvature capacity requirements of this section.
>
> 1. Precast prestressed concrete piles detailed in accordance with Section 1810.3.8.3.3.
> 2. Cast-in-place deep foundation elements with a minimum longitudinal reinforcement ratio of 0.005 extending the full length of the element and detailed in accordance with Sections 18.7.5.2, 18.7.5.3 and 18.7.5.4 of ACI 318 as required by Section 1810.3.9.4.2.2.

Reason: This proposal is a re-organization of the pointers in the IBC to the referenced loading standard ASCE 7 Minimum Design Loads and Associated Criteria fo r Buildings and Other Structures.
Section 1808.3 - This proposed change clarifies that Strength (LRFD) and Basic Allow able Stress Design Load Combinations used for foundation design are found in ASCE 7, rather than in 1605. This clarification is necessary if the companion proposal to remove duplication of the ASCE 7 Load Combinations in the building code leaving in place only the Alternate Allow able Stress Combinations, w hich are not included in ASCE 7. Other changes to this section are to correct Section references to ASCE 7-16.

Section 1810.2.4.1 - This proposal removed references to IBC Section 1613.3.2 for determination of Site Class and instead refers to Chapter 20 of the ASCE 7 Standard for this information. This is necessary to avoid confusion, should the companion proposal removing the Seismic Design Parameter maps, Site Class Coefficient, and Seismic Design Category standard and replacing these materials w ith reference to ASCE 7 is approved.

Cost Impact: Will not increase the cost of construction
The proposed changes w ill 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* w ill 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).

S194-16 : 1808.3-HAMBURGER13236

S195-16

IBC: 1808.8.7 (New).

Proponent : Gerald Gunny, representing Southern Nevada Chapter of the International Code Council

2015 International Building Code

Add new text as follows:

1808.8.7 Use of existing slabs on ground to resist loads Where loads are proposed to be resisted by existing slabs on ground, all of the following conditions shall be satisfied:

1. Structural investigation shall be provided to the *building official* that confirms existing slab thickness and condition assumed in structural calculations.
2. Structural calculations shall be provided to demonstrate that the existing slab can adequately support the proposed loads.
3. The maximum allowable assumed subgrade bearing pressure below the slab shall be no greater than 750 psf unless a greater value is justified in a geotechnical investigation report.

Reason: Non-structural slabs on ground are typically exempt from structural design and special inspection requirements in the code. There may be limited, if any, subgrade preparation below non-structural slabs. These slabs are often proposed to support concentrated loads from tenant improvements or additions (such as storage racks or mezzanines) and it is prudent to demonstrate that the existing slab can adequately support these proposed loads through structural investigation and analysis. The 750 psf allow able bearing limit takes into account that a slab on ground w ould have less than the required 12" embedment depth typical to conventional foundations**.**

Cost Impact: Will increase the cost of construction
Minimal increase in project fees associated w ith a registered design professional providing the structural investigation and analysis.

S195-16 : 1808.8.7 (NEW)-GUNNY11606

S196-16

IBC: 1809.5.

Proponent : James Smith, ICC Region III Code Development Committee, representing ICC Region III Code Development Committee

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 m^2) or less for light-frame construction or 400 square feet (37 m^2) 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 conditons 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.

S196-16 : 1809.5-SMITH4022

S197-16

IBC: 1810.1.2.

Proponent : Dale Biggers, P.E., 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:

1810.1.2 Use of existing deep foundation elements. Deep foundation elements left in place where a structure has been demolished shall not be used for the support of new construction unless satisfactory evidence is submitted to the *building official*, which indicates that the elements are ~~sound and meet~~ <u>deemed suitable for reuse by</u> the ~~requirements of this code~~<u>*registered design professional*</u>. Such elements shall be load tested or redriven to verify their capacities. The design load applied to ~~such elements~~ <u>each type of element</u> shall be the lowest allowable load ~~as~~ determined~~ by tests or redriving data~~.

Reason: The phrase about meeting current code requirements has been removed because it is impossible to determine some code requirements, such as, concrete cover for the full depth of the foundation. It is necessary to clarify w hich element (it applies to various types). The last phrase is redundant since previous sentence specifies the same tasks.

Click here to view the members of the GeoCoalition w ho developed this proposal

Cost Impact: Will not increase the cost of construction
The change reflects current practice.

S197-16 : 1810.1.2-BIGGERS10658

S198-16

IBC: 1810.2.1.

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:

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

Where deep foundation elements ~~stand unbraced~~ extend in air, water or fluid soils, it shall be permitted to consider them laterally supported at a point 5 feet (1524 mm) into stiff soil or 10 feet (3048 mm) into soft soil unless otherwise *approved* by the *building official* on the basis of a geotechnical investigation by a *registered design professional*.

Reason: This is a terminology change to avoid contradiction w ith bracing elements.

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Cost Impact: Will not increase the cost of construction
This is just a change in terminology but no change to actual practice.

S198-16 : 1810.2.1-BIGGERS10620

S199-16

IBC: 1810.2.2.

Proponent : Dale Biggers, P.E., 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:

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

Deep foundation elements supporting walls shall be placed alternately in lines spaced at least 1 foot (305 mm) apart and located symmetrically under the center of gravity of the wall load carried, unless effective measures are taken to provide for eccentricity and lateral forces, or the foundation elements are adequately braced to provide for lateral stability.

Exceptions:

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

Reason: The current w ording w as extremely confusing and interpreted in five different w ays by the GeoCoalition Codes Committee. The current code may limit a 2-ft-diameter cast-in-place deep foundation element to a maximum length of 24 ft. This is not justifiable w ith respect to soil mechanics and foundation design principles. By clarifying the code, you w ould allow additional cost-effective structural options.

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Cost Impact: Will not increase the cost of construction
The change w ould possibly allow additional cost-effective options as described in the above reasons.

S199-16 : 1810.2.2-BIGGERS10659

S200-16

IBC: 1810.2.3.

Proponent : Dale Biggers, P.E., 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:

1810.2.3 Settlement. The settlement of a single deep foundation element or group thereof shall be estimated based on ~~*approved*~~ generally accepted methods of analysis. The predicted settlement shall cause neither harmful distortion of, nor instability in, the structure, nor cause any element to be loaded beyond its capacity.

Reason: There are no approved methods of foundation design, only generally accepted methods. This suggestion makes it consistent w ith similar changes instituted in the 2015 code. This change covers commonly accepted methods of analysis, w hich may vary regionally due to local soil conditions, and places the responsibility of selection w ith the designer.

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Cost Impact: Will not increase the cost of construction
Generally accepted methods are w hat are currently being used.

S200-16 : 1810.2.3-BIGGERS10663

S201-16

IBC: 1810.2.5.

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:

1810.2.5 Group effects. ~~The~~ Group effects shall be evaluated by the *registered design professional*. At a minimum the analysis shall include group effects on lateral behavior where the center-to-center spacing of deep foundation elements in the direction of lateral force is less than eight times the least horizontal dimension of an element. ~~The~~ At a minimum the analysis shall include group effects on axial behavior where the center-to-center spacing of deep foundation elements is less than three times the least horizontal dimension of an element. Group effects shall be evaluated using a generally accepted method of analysis; the analysis for uplift of grouped elements with center-to-center spacing less than three times the least horizontal dimension of an element shall be evaluated in accordance with Section 1810.3.3.1.6.

Reason: The registered design professional should be required to make some consideration.
Group effects need to be analyzed to assure adequate factors of safety are achieved.
Group effect may be relevant even above the cited limit of three times center-to-center distance.

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Cost Impact: Will increase the cost of construction
The code change proposal may possibly (but not necessarily) increase the cost of construction but w ill assure that adequate factors of safety are achieved.

S201-16 : 1810.2.5-BIGGERS10664

S202-16

IBC: 1810.3.1.1.

Proponent : Dale Biggers, P.E., 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

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Revise as follows:

1810.3.1.1 Design methods for concrete elements. Where concrete deep foundations are laterally supported in accordance with Section 1810.2.1 for the entire height and applied forces cause bending moments ~~no~~ not greater than ~~those resulting from accidental eccentricities~~ the design cracking moment determined in accordance with Section 1605.3, structural design of the element, or portion thereof, using the load combinations of Section 1605.3 and the allowable stresses specified in this chapter shall be permitted. ~~Otherwise, the structural design~~ Those portions of ~~concrete~~ deep foundation elements where the bending moment exceeds the design cracking moment shall ~~use~~ be designed using the load combinations of Section 1605.2 and ~~*approved*~~the strength design methods of ACI 318. When designing structural materials for combined stresses in accordance with ACI 318, the design axial loads determined in accordance with Section 1605.2 shall be increased by 10 percent to account for mislocation of deep foundation elements. In addition, strength reduction factors for concrete in shear and compression shall be decreased by 5 percent for concrete deep foundation elements without permanent casing. Stresses from axial load shall not exceed the allowable stresses specified in Table 1810.3.2.6.

Reason: The term "accidental eccentricity" is replaced w ith "design cracking moment" in order to be consistent w ith section 1810.3.9.2. "Accidental eccentricity" is an arbitrary term, w hereas "design cracking moment" is a specific value. The reference to an approved strength design method is replaced by a reference to ACI 318, w hich is an industry accepted method. The 10% increase in design axial loads is intended to account for increased loads on piles due to pile mislocations.

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Cost Impact: Will not increase the cost of construction
The code change proposal w ill not increase the cost of construction since it is a clarification to the code (e.g. to replace an arbitrary and vague term w ith specific guidance).

S202-16 : 1810.3.1.1-BIGGERS10665

S203-16

IBC: 1705.10 (New), 1802.1, 1810.3.1.6 (New), 1810.3.5.4 (New), 1810.4.13 (New), 202 (New).

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 w ere established in committee (http://w w w .foundationperformance.org/projects/FPA-SC-08-1.pdf) to provide guidance for the use and installation.

References
ICC-Es AC349 Acceptance Criteria for Segmented Precast Concrete Pile Assemblies. International Code Council Evalutation Service, Inc. 2007

Bibliography: http://w w w .foundationperformance.org/projects/FPA-SC-08-1.pdf

Cost Impact: Will not increase the cost of construction
This product and methodology is currently in use for repair and remediation of foundations. This product and methodology offers clients an alternative to other more expensive and or invasive methods and products and therefore offers clients a cost savings in some situations.

S203-16 : 1704 (NEW)-SCHILDER4422

S204-16

IBC: 1810.3.1.7 (New).

Proponent : Dale Biggers, P.E., 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

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Add new text as follows:

1810.3.1.7 Design methods for structural steel elements. Where structural steel deep foundations are laterally supported in accordance with Section 1810.2.1 for the entire height and applied forces cause bending moments no greater than those resulting from accidental eccentricities, structural design of the element using the load combinations of Section 1605.3 and the allowable stresses specified in this chapter shall be permitted. Otherwise, the structural design of structural steel deep foundation elements shall use the load combinations of Sections 1605.2 or 1605.3 and AISC 360. When designing structural materials for combined stresses in accordance with AISC 360, the design axial loads determined in accordance with Section 1605.2 or 1605.3 shall be increased by 10 percent to account for mislocation of deep foundation elements. Stress from axial load shall not exceed the allowable stresses specified in Table 1810.3.2.6.

Reason: We are adding a new section to clarify the design of steel driven piles to allow for accepted design methods specific to structural steel. The 10% increase in design axial loads is intended to account for increased loads on piles due to pile mislocations, which is consistent with Section 1810.3.1.3.

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Cost Impact: Will not increase the cost of construction
The code change proposal will not increase the cost of construction because it is added to clarify the design methods to be used to check stresses referenced in section 1810.3.1.4.

S204-16 : 1810.3.1.7 (NEW)-BIGGERS10666

S205-16

IBC: 1802.1, 1810.3.11, 202 (New).

Proponent : Dale Biggers, P.E., 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

Add new definition as follows:

SECTION 202 DEFINITIONS

COMBINED PILE RAFT. A geotechnical composite construction that combines the bearing effect of both foundation elements, raft and piles, by taking into account interactions between the foundation elements and the subsoil.

Revise as follows:

1802.1 Definitions. The following words and terms are defined in Chapter 2:

COMBINED PILE RAFT
DEEP FOUNDATION.
DRILLED SHAFT.
Socketed drilled shaft.
HELICAL PILE.
MICROPILE.
SHALLOW FOUNDATION.

1810.3.11 Pile caps. Pile caps shall be of reinforced concrete, and shall include all elements to which vertical deep foundation elements are connected, including grade beams and mats. The soil immediately below the pile cap shall not be considered as carrying any vertical load, with the exception of a combined pile-raft. The tops of vertical deep foundation elements shall be embedded not less than 3 inches (76 mm) into pile caps and the caps shall extend at least 4 inches (102 mm) beyond the edges of the elements. The tops of elements shall be cut or chipped back to sound material before capping.

Reason: There is no existing definition for this type of deep foundation. This proposed code addition is to identify another commonly-used type of deep foundation along with drilled shafts, helical piles, and micropiles. This term is added to the definitions because the term "combined pile-raft" is a proposed change in Section 1810.3.11. Combined pile-rafts are increasingly common and can lower the foundation costs by relying partially on the soil under the raft.

The following definition is from the ISSMGE guideline for "Combined Pile Raft Foundations".

> "The Combined Pile Raft Foundation is a geotechnical composite construction that combines the bearing effect of both foundation elements raft and piles by taking into account interactions between the foundation elements and the subsoil."

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Cost Impact: Will not increase the cost of construction
The code change proposal to add a definition will not change the cost of construction since that is simply an addition of a definition. The code change proposal to exempt combined pile rafts will not increase the cost of construction and in favorable conditions (when combined pile rafts are feasible) will decrease the cost of construction.

S206-16

IBC: 1810.3.2.6.

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

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Revise as follows:

1810.3.2.6 Allowable axial stresses. *No change to text.*

TABLE 1810.3.2.6
ALLOWABLE STRESSES FOR MATERIALS USED IN DEEP FOUNDATION ELEMENTS

MATERIAL TYPE AND CONDITION	MAXIMUM ALLOWABLE AXIAL STRESS[a]
1. Concrete or grout in compression[b]	
A. Cast-in-place with a mandrel-driven permanent casing in accordance with Section 1810.3.2.7 B. Mandrel-driven elements not meeting Section 1810.3.2.7	$0.4\ f'_c$ $0.33f'_c$
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 D. Micropiles cased length	0.4~~33~~f'_c $0.4\ f'_c$
E. Cast-in-place without a permanent casing With verification of area versus depth[c] Without verification of area versus depth[c]	 $0.3f'_c$ $0.25\ f'_c$
F. Precast non-prestressed	$0.33f'_c$
G. Precast prestressed	$0.33f'_c - 0.27f_{pc}$
2. Nonprestressed reinforcement in compression A. Within Micropiles with rock sockets where strain compatibility is checked by the engineer	$0.4\ f_y \leq 30{,}000$ psi $0.4\ f_y \leq 48{,}000$ psi

3. Steel in compression[e]	
A. Cores within concrete-filled pipes or tubes	0.5 F_y ≤ 32,000 psi
B. Pipes, tubes (including cased sections of Micropiles) or H-piles, where justified in accordance with Section 1810.3.2.8	0.5 F_y ≤ 32,000 psi
C. Pipes or tubes for micropiles	0.4 F_y ≤ 32,000 psi
D. Other pipes, tubes or H-piles	0.35 F_y ≤ ~~16~~ 24,000 psi
E. Helical piles	0.5~~0.6~~F_y ≤ 32,000psi ~~0.5 F_u~~
4. Nonprestressed reinforcement in tension[d,e]	
A. Within micropiles	0.6 f_y ≤ 72,000 psi
B. Other conditions	0.5 f_y ≤ ~~24~~60,000 psi
5. Steel in tension[e]	
A. Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8	0.5 F_y ≤ 32,000 psi
B. Other pipes, tubes or H-piles	0.35 F_y ≤ ~~16~~24,000 psi
C. Helical piles	~~0.6~~0.5 F_y ≤ 32,000 psi ~~0.5 F_u~~
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; fy 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 w ord "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 low er compared to 1.A and 1.C.

• 1.C.: Concrete cast in a driven pipe or tube (in accordance w ith 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 allow able stresses.

1810.3.2.7 says "Confinement is the technical qualification that permits the use of increased allow able compressive stresses". The shape is know n prior to placing concrete and contamination from soil is not possible. "Suitable" means able to w ithstand 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 allow ed 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 w eak 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 allow s more economic use of the higher strength steels.

• 3.E.: The fixed limit provides consistency w ith the other pile types. The specific limit w as chosen based on the available material limits at the time of this code proposal. The multiplier for Fy has been low ered to be consistent w ith 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 w eak link in tension.

• 4.A.: A 150ksi bar, w hich has a yield strength of 120ksi, is commonly available. We added a specific upper limit to be consistent w ith 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, w hich 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 w eak 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 w ith the other pile types. The multiplier for Fy has been low ered to be consistent w ith other steel sections.

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• 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 w ill not increase the cost of construction, and could result in reduced costs since higher allow able 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 w ill enhance the reliability (and reduce the long-term cost).

S207-16

IBC: 1810.3.2.8.

Proponent : Dale Biggers, P.E., Boh Bros. Construction Co.,L.L.C. (dbiggers@bohbros.com); Lori Simpson, P.E.,G.E., representing GeoCoalition (lsimpson@langan.com); Daniel Stevenson, P.E., representing GeoCoalition; E. Anna Sellountou, PhD,PE, representing GeoCoalition

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Revise as follows:

1810.3.2.8 Justification of higher allowable stresses. Use of the higher allowable stresses ~~greater than those~~ specified in Section 1810.3.2.6 shall be permitted where supporting data justifying such higher stresses is filed with the *building official*. Such substantiating data shall include the following:

1. A geotechnical investigation in accordance with Section 1803.
2. Load tests in accordance with Section 1810.3.3.1.2, regardless of the load supported by the element.

The design and installation of the deep foundation elements shall be under the direct supervision of a *registered design professional* knowledgeable in the field of soil mechanics and deep foundations who shall submit a report to the *building official* stating that the elements as installed satisfy the design criteria.

Reason: This section as currently w ritten in the existing 2015 code w ould override the limits listed in TABLE 1810.3.2.6 and could allow excessive stresses if a pile passes a load test.

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Cost Impact: Will not increase the cost of construction
The code change proposal w ill not increase the cost of construction. It just allow s current practice to be codified. The proposed change herein could result in reduced costs if higher allow able stresses as per proposed table 1810.3.2.6 limits are adopted.

S207-16 : 1810.3.2.8-BIGGERS10669

S208-16

IBC: 1810.3.3.

Proponent : Lori Simpson, P.E.,G.E., representing GeoCoalition; E. Anna Sellountou, PhD,PE, representing GeoCoalition; Dale Biggers, P.E., representing GeoCoalition (dbiggers@bohbros.com); Daniel Stevenson, P.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

1810.3.3 Determination of allowable loads. The allowable axial and lateral loads on deep foundation elements shall be determined by ~~an *approved* formula, load tests or~~ a generally accepted method of analysis and load tests in accordance with the following sections. This determination shall be made by a registered design professional knowledgeable in the field of soil mechanics and deep foundations.

Reason:

-
  - There is no generally approved formula (e.g. driving formula or energy formula). Formulas have generally been replaced by more modern methods of analysis. There are many w ays to determine ultimate loads on an element, and it could include a formula if the registered design professional approves it.
  - The change from "or" to "and" requires a load test unless it is w aived by the registered design professional provision in the follow ing sections (e.g. 1810.3.3.1.1).
  - The method of load determination should be made by someone experienced in deep foundations design.

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Cost Impact: Will not increase the cost of construction
The change clarifies that the ultimate load determination method lies w ith the registered design professional.

S208-16 : 1810.3.3-SIMPSON10480

S209-16

IBC: 1810.3.3.1.

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

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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 conditons 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.

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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.

S209-16 : 1810.3.3.1-SIMPSON11076

S210-16

IBC: 1810.3.3.1.1.

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:

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

Reason:

- The section w as modified to reflect current practice and more modern pile hammers and methods of driveability analysis. Driving formulas can be unreliable and should be discouraged in the presence of more modern and reliable methods of analysis.
- The "40 tons" limit is an arbitrary limit and is not necessarily conservative. Many important structures are founded on piles w ith allow able loads of much low er than 40 tons.
- The responsibility to approve the use of a follow er is sw itched from the building official to the registered design professional, as the registered design professional is also responsible to monitor the installation.

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Cost Impact: Will not increase the cost of construction

The code change proposal w ill not increase the cost of construciton because it doesn't change current practice. It w ould probably reduce cost as it w ould result in cost-effective installations. In addition, the 40-ton limit likely increased pile count on many projects.

S210-16 : 1810.3.3.1.1-SIMPSON11077

S211-16

IBC: 1810.3.3.1.2.

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:

1810.3.3.1.2 Load tests. ~~Where design compressive loads are greater than those determined using the allowable stresses specified in Section 1810.3.2.6, where the design load for any deep foundation element is in doubt, or where cast-in-place deep foundation elements have an enlarged base formed either by compacting concrete or by driving a precast base, control test elements shall be tested~~
Unless waived in accordance with Section 1810.3.3.1, test shall be conducted in accordance with a method selected by the *registered design professional*, including but not limited to ASTM D 1143~~ or~~ , ASTM D 4945, or ASTM D7383. At least one element shall be load tested in each area of uniform subsoil conditions. ~~Where required by the *building official*, additional~~ Additional elements shall be load tested where deemed necessary by the *registered design professional* to establish the ~~safe design capacity~~ allowable geotechnical compression load. The ~~resulting~~ allowable ~~loads~~ load shall ~~not~~ be determined by a *registered design professional* and be not more than one-half of the ultimate axial load capacity of the test element ~~as assessed by one of the published methods listed in Section 1810.3.3.1.3 with consideration for the test type, duration and subsoil. The ultimate axial load capacity shall be determined by a *registered design professional*~~with consideration given to test type as well as tolerable total and differential settlements at~~ design~~ allowable load in accordance with Section 1810.2.3.~~ In subsequent installation of the balance of deep foundation elements, all elements shall be deemed to have a supporting capacity equal to that of the control element where such elements are of the same type, size and relative length as the test element; are installed using the same or comparable methods and equipment as the test element; are installed in similar subsoil conditions as the test element; and, for driven elements, where the rate of penetration (e.g., net displacement per blow) of such elements is equal to or less than that of the test element driven with the same hammer through a comparable driving distance.~~

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
ASTM D7383
Reason:

- • The w ord "compression" w as added in the title to distinguish from other types of load tests e.g. uplift, lateral etc.
  • The first sentence w as deleted as the performance of compression load tests should not be limited to the cases mentioned herein for the follow ing reasons:
  • Values larger than in table 1810.3.2.6 should no longer be allow ed. The intent is to separate the structural limits of 1810.3.2.6 and the geotechnical limits of this section.
  • Ultimate load capacity, and corresponding allow able loads based on the applied factor of safety, are inherently in doubt.
  • Enlarged base elements are rare and need no special mention. The registered design professional should understand the implications if an enlarged base situation is employed.
  • This is consistent w ith the current versions of sections 1810.3.3.1.1, w here it is stated "Allow able loads shall be verified by load tests in accordance w ith Section 1810.3.3.1.2".
  • The clause "Tests shall be conducted in accordance w ith a method selected by the registered design professional," w as added as the selection of the load test method should lie w ith the registered design

professional.

- ASTM D7383 method (rapid load testing) was added to the list of load test methods.
- The clause regarding "the additional elements to be tested" was rephrased in order to shift responsibility from the building official to the registered design professional.
- The term "safe design capacity" was replaced by the term "allowable geotechnical compression load", which is a term more widely used and therefore more generally understood.
- The term "ultimate axial load capacity" was replaced by the term "allowable load" and the "allowable load" clause was combined with the next sentence and unnecessary language was deleted.
- The phrase "with consideration given to test type as well as" was added to reflect the fact that different test types may have different effects on the results (e.g. a maintained load test versus a quick load test, or a dynamic test or rapid load test versus a static test).
- The term "design load" was replaced with the term "allowable load" as it is considered better terminology.
- The last sentence was moved to section 1810.3.3.1.

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Cost Impact: Will not increase the cost of construction
The code change proposal will not increase the cost of construction since the proposed changes to Table 1810.3.2.6 have increased several of the maximum allowable material stresses, which could decrease the costs; the old increased stress clause was rarely used. Most of the changes are just clarification of wording.

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM D7383, 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.

S211-16 : 1810.3.3.1.2-SIMPSON11250

S212-16

IBC: 1810.3.3.1.3.

Proponent : Lori Simpson, P.E.,G.E., representing GeoCoalition; E. Anna Sellountou, PhD,PE, representing GeoCoalition; Dale Biggers, P.E., representing GeoCoalition (dbiggers@bohbros.com); Daniel Stevenson, P.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

1810.3.3.1.3 Load test evaluation methods. It shall be permitted to evaluate load tests of deep foundation elements <u>by ASTM D1143 </u>using any of the following methods:

1. Davisson Offset Limit.
2. Brinch-Hansen 90-percent Criterion.
3. Butler-Hoy Criterion.
4. Other methods <u>proposed by the *registered design professional* and </u>*approved* by the *building official*.

Reason: Clarification is needed since these evaluation methods are used only w ith the static compression load test. The phrase " by ASTM D1143", w hich is the standard for static load tests, w as added for that reason.
The registered design professional is responsible to determine the allow able loads and should be responsible to select the method to interpret the load test. There are many methods to evaluate a load-displacement curve. These three are the most commonly used.

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Cost Impact: Will not increase the cost of construction
The above change is just a code clarification and w ill not increase the cost of construction.

S212-16 : 1810.3.3.1.3-SIMPSON11078

S213-16

IBC: 1810.3.3.1.4.

Proponent : Lori Simpson, P.E.,G.E., representing GeoCoalition; E. Anna Sellountou, PhD,PE, representing GeoCoalition; Dale Biggers, P.E., representing GeoCoalition (dbiggers@bohbros.com); Daniel Stevenson, P.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

1810.3.3.1.4 Allowable ~~frictional~~ shaft resistance. The assumed ~~frictional~~ shaft resistance developed by any uncased cast-in-place deep foundation element shall not exceed one-sixth of the bearing value of the soil material at minimum depth as set forth in Table 1806.2, up to a maximum of 500 psf (24 kPa), unless a greater value is allowed by the *building official* on the basis of a geotechnical investigation as specified in Section 1803 or a greater value is substantiated by a load test in accordance with Section 1810.3.3.1.2. ~~Frictional~~ Shaft resistance and ~~bearing~~ end-bearing resistance shall not be assumed to act simultaneously unless determined by a geotechnical investigation in accordance with Section 1803.

Reason: Change in w ording to the most commenly used terms of "shaft" rather than "frictional" resistance and "end-bearing" rather than "bearing" resistance.

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Cost Impact: Will not increase the cost of construction
The code change w ill not increase the cost of construction as it is just an improvement in w ording.

S213-16 : 1810.3.3.1.4-SIMPSON11079

S214-16

IBC: 1810.3.3.1.5.

Proponent : Lori Simpson, P.E.,G.E., representing GeoCoalition; Dale Biggers, P.E., Boh Bros. Construction Co.,L.L.C., representing GeoCoalition; Daniel Stevenson, P.E., representing GeoCoalition; E. Anna Sellountou, PhD,PE, representing GeoCoalition

2015 International Building Code

Revise as follows:

1810.3.3.1.5 Uplift ~~capacity~~ load of a single deep foundation element. Where required by the design, the allowable uplift ~~capacity~~ load of a single deep foundation element shall be determined by ~~an *approved*~~ a generally accepted method of analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D 3689. ~~The maximum allowable~~ With appropriate instrumentation and analyses by the *registered design professional*, the uplift ~~load shall not exceed the ultimate load~~ capacity ~~as~~ is permitted to be determined ~~in Section 1810.3.3.1.2,~~ using ~~the results of~~ bi-directional static axial load ~~tests conducted~~ testing or load testing performed in accordance with ASTM ~~D 3689~~ D1143, ~~divided~~ ASTM D4945 or ASTM D7383. When load testing is performed, the allowable load shall be determined by a ~~factor~~ *registered design professional* and be not more than one-half of ~~safety~~ the ultimate uplift load capacity of ~~two~~ the test element with consideration given to the test type. Consideration shall be given to tolerable total and differential movements.

> **Exception:** Where uplift is due to wind or seismic loading, the minimum factor of safety shall be two where ~~capacity~~ allowable load is determined by an analysis ~~and one and one-half~~ or 1.5 where ~~capacity~~ allowable load is determined by load ~~tests~~ testing.

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
ASTM D7383-10 Standard Test Methods for Axial Compressive Force Pulse (Rapid) Testing of Deep Foundations

Reason:

- This is a clarification since the term "factor of safety" is used and "capacity" is defined as "ultimate," the term "allow able uplift load" or simply "allow able load" is used in place of "capacity."
- To achieve consistency w ith other sections and to allow standard practice, the term "approved" w as replaced by the term "generally accepted."
- The second reference to ASTM D 3689 w as eliminated as it is redundant.
- Bi-directional static axial load testing, ASTM D1143, ASTM D4945, and ASTM D7383 w ere added because uplift capacity can be determined by these methods w ith commonly available instrumentation and appropriate analysis.
- The reference to section 1810.3.3.1.2 w as removed because that section refers to compression testing.
- The last sentence is a reminder that the total and differential upw ard movements must be considered.

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Cost Impact: Will not increase the cost of construction
The code change proposl w ill not increase the cost of construction as it is generally w ording changes and reflets current standard practice. Some tests are less expensive than others so changes may reduce cost.

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM D7383, w ith regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) w ill be posted on the ICC w ebsite on or before April 1, 2016.

S215-16

IBC: 1810.3.3.1.6.

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:

1810.3.3.1.6 ~~Uplift capacity~~ Allowable uplift load of grouped deep foundation elements. For grouped deep foundation elements subjected to uplift, the allowable ~~working~~ uplift load for the group shall be calculated by a generally accepted method of analysis. Where the deep foundation elements in the group are placed at a center-to-center spacing less than three times the least horizontal dimension of the largest single element, the allowable ~~working~~ uplift load for the group is permitted to be calculated as the lesser of:

1. The proposed individual allowable ~~working~~ uplift load times the number of elements in the group.
2. Two-thirds of the effective weight of the group and the soil contained within a block defined by the perimeter of the group and the length of the element, plus two-thirds of the ultimate shear resistance along the soil block.

Reason:

- This is a clarification to replace "capacity" w ith "load" since a safety factor is implied by "allow able" or "w orking", and "capacity" is by definition an "ultimate". It is the maximum "load" that is being "allow ed".
- The w ord "w orking" is confusing and further is redundant since "allow able" is alw ays present.

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Cost Impact: Will not increase the cost of construction
The code change w ill not increase the cost of construction since it is just a w ording change for clarity.

S215-16 : 1810.3.3.1.6-SIMPSON11081

S216-16

IBC: 1810.3.3.1.7.

Proponent : Lori Simpson, P.E.,G.E., representing GeoCoalition; Dale Biggers, P.E., representing GeoCoalition; Daniel Stevenson, P.E., representing GeoCoalition; E. Anna Sellountou, PhD,PE, representing GeoCoalition

2015 International Building Code

Revise as follows:

1810.3.3.1.7 ~~Load-bearing~~ Bearing capacity of underlying layers. ~~Deep foundation elements shall develop ultimate load capacities of at least twice the design working loads in the designated load-bearing layers.~~
Analysis shall show that no soil layer underlying the ~~designated~~ load-bearing ~~layers~~ layer(s) causes the ~~load-bearing capacity~~ safety factor of the entire foundation to be less than two and that the predicted settlement complies with Section 1810.2.3.

Reason:

- In title, "Load" is an applied force w hile "capacity" is the geotechnical strength of the supporting soil, so the terms are not compatible w hen describing one item. The purpose of this section is to check underlying layers so that is included in the title.
- First sentence is redundant as it is repeats the information contained in previous code sections (e.g. Section 1810.3.3.1).
- The influence of the "entire foundation" is the key to w hether the underlying w eak layer is of any significance for both capacity and settlement. The referenced section 1810.2.3 already has a good discussion of this issue, and is included as a friendly reminder to the designer.

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Cost Impact: Will not increase the cost of construction
The code change w ill not increase the cost of construction as it clarified the code and reflects current practice.

S216-16 : 1810.3.3.1.7-SIMPSON11082

S217-16

IBC: 1810.3.3.1.8.

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:

1810.3.3.1.8 Bent deep foundation elements. The load-bearing capacity of deep foundation elements discovered to have a sharp or sweeping bend shall be determined by ~~an~~ the~~*approved*~~ *registered design professional* ~~method of analysis or by load testing a representative element~~.

Reason: There is a lack of consensus as to w hat is an approved method of analysis. The registered design professional should determine the acceptable capacity. Load testing is listed in the commentary as a possible method of determining capacity.

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Cost Impact: Will increase the cost of construction
The code change proposal w ill not increase the cost of construction as it reflects current practice.

S217-16 : 1810.3.3.1.8-SIMPSON11083

S218-16

IBC: 1810.3.3.1.9.

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:

$P_a = 0.5\ P_u$ **(Equation 18-4)**

where P_u is the least value of:

1. ~~Sum~~ 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 ~~capacity~~ 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.
2. Ultimate capacity determined from well-documented correlations with installation torque. The installation torque shall not exceed the manufacturer's rated torque of the shaft, couplers, and helix plates.
3. ~~Ultimate capacity determined from load tests.~~
4. ~~Ultimate axial capacity of pile shaft.~~
5. ~~Ultimate axial capacity of pile shaft couplings.~~
6. ~~Sum of the ultimate axial capacity of helical bearing plates affixed to pile.~~

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 w ay allow s 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 allow s this provision to focus on the geotechnical capacity w hich is w hat 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 allow able 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 w here there is dow n drag, neglecting this skin friction is unconservative and should be included in the design. There are also some circumstances w ith larger diameter helical piles w here the designer may desire to include the effects of skin friction and the addition of this language allow s for the designer to utilize that skin friction at their option.

Item 2 has been revised to include the limits of the manufacture'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.

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Cost Impact: Will not increase the cost of construction
The code change proposal w ill not increase the cost of construction since it is a clarification to the code.

S218-16 : 1810.3.3.1.9-SIMPSON11084

S219-16

IBC: 1810.3.3.2.

Proponent : Lori Simpson, P.E.,G.E., representing GeoCoalition; E. Anna Sellountou, PhD,PE, 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:

1810.3.3.2 Allowable lateral load. Where required by the design, the allowable lateral load ~~capacity of~~ on a single deep foundation element or a group thereof shall be determined by ~~an~~ a~~*approved*~~ *registered design professional*using a generally accepted method of analysis or by lateral load tests according to ~~at least twice the proposed design working load~~ ASTM D3966. The resulting allowable lateral load shall not be more than one-half of the load that produces a gross lateral movement of 1 inch (25 mm) at the lower of (a) the top of foundation element ~~and~~ or (b) the ground surface, unless it can be shown that the predicted lateral movement shall cause neither harmful distortion of, nor instability in, the structure, nor cause any element to be loaded beyond its ~~capacity~~ permissible structural strength.

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
ASTM D3966/D3966M-07 Standard Test Methods for Deep Foundations Under Lateral Load
Reason:

- "Capacity" is a term that means "ultimate," and is in conflict w ith "allow able" (w hich is the title of the section).
- "Generally accepted" is the preferred w ording used in other sections of the code.
- The applicable ASTM standard for lateral loading is added for clarity.
- Reference to "w orking load" is removed because the allow able load is defined by the last sentence in this section.
- The w ord "low er" is a relative comparison betw een tw o items so "or" is a better term than "and." The (a) and (b) are added for clarity.
- "Permissible structural strength" clarifies the intent of the w ord "capacity" at the end of the section

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Cost Impact: Will not increase the cost of construction

The code change proposal is a clarification and w ill not increase the cost of construction.

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM D3966, w ith regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) w ill be posted on the ICC w ebsite on or before April 1, 2016.

S219-16 : 1810.3.3.2-SIMPSON11085

S220-16

IBC: 1810.3.4.

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 ~~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 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 w here the increase is applicable.
- **Definition of Neutral Plane:** The location w here equilibrium exists betw een 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 w here relative movement betw een 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 allow able stress limit near the neutral plane. The registered design professional needs to substantiate the actual allow able stress, w hich may be a factor less than 0.7.

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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

S221-16

IBC: 1810.3.5.2.1.

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:

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

Reason: The section title is changed for consistency w ith the title and definition of main section 1810.3.5.2 to w hich this subsection belongs.

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Cost Impact: Will not increase the cost of construction
The code change proposal w ill not increase the cost of construction since it merely is a change to make it consistent w ith its parent section.

S221-16 : 1810.3.5.2.1-STEVENSON10934

S222-16

IBC: 1810.3.5.2.2.

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:

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

> **Exception:** The length of the element is permitted to exceed 30 times the specified diameter, provided the design and installation of the deep foundations are under the direct supervision of a *registered design professional* knowledgeable in the field of soil mechanics and deep foundations. The *registered design professional* shall submit a report to the *building official* stating that the elements were installed in compliance with the *approvedconstruction documents*.

Reason: The wording is changed for consistency with the title and definition of main section 1810.3.5.2 to which this subsection belongs.
The word "average" would require a physical measurement that is not possible, so it has been replaced with "specified" (this word is also added to the Exception condition for clarity).

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Cost Impact: Will not increase the cost of construction
The code change proposal will not increase the cost of construction since it is a change to make it consistent with its parent section, and a change in terminology.

S222-16 : 1810.3.5.2.2-STEVENSON10936

S223-16

IBC: 1810.3.5.2.3.

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:

1810.3.5.2.3 Micropiles. Micropiles shall have ~~an outside~~ a nominal diameter of 12 inches (nominal 305 mm) or less. The minimum diameter set forth elsewhere in Section 1810.3.5 shall not apply to micropiles.

Reason: This proposed change is intended to clarify the Code relative to the upper-end of conventionally available diameters of pipe used for micropiles and correct terminology.

Steel micropile flush-joint pipe is available within the industry as prime or mill secondary material and generally conforming to ASTM A252 or API 5CT. In accordance with these standards, the true outside diameter for 12-inch-diameter pipe is 12-3/4 inches. Within standard terminology for describing pipe, this would be considered a "12-inch nominal diameter" size.

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Cost Impact: Will not increase the cost of construction
This code change proposal is simply in terminology and will not increase the cost of construction.

S223-16 : 1810.3.5.2.3-STEVENSON10938

S224-16

IBC: 1810.3.5.3.2.

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:

1810.3.5.3.2 Fully welded steel piles fabricated from plates. Sections of ~~fully welded~~ steel piles fabricated from plates shall have full-length welds complying with AWS D1-1 and shall comply with the following:

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

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
AWS D1.1/D1.1M 2015 Structural Welding Code - Steel

Reason: The purpose of this change is to reference a recognized standard. The American Welding Society has applicable requirements for structural welding.

Click here to view the members of the GeoCoalition who developed this proposal

Cost Impact: Will not increase the cost of construction
This code change proposal will not increase the cost of construction because any prudent fabricator is now following the AWS standard.

Analysis: A review of the standard(s) proposed for inclusion in the code, AWS D1.1, 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.

S224-16 : 1810.3.5.3.2-STEVENSON10937

S225-16

IBC: 1810.3.5.3.4.

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 mm^2) 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 $^3/_8$ 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 w ith 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 w ho developed this proposal

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.

S225-16 : 1810.3.5.3.4-STEVENSON10942

S226-16

IBC: 1810.3.6.

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 w ith 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" w ith "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 tw o sentences w ill 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 w ould be acceptable in many design situations, such as a splice located at significant depth. The design professional w ill 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 w ood-to-w ood 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 w ill be further evaluated by the *registered design professional.*

Click here to view the members of the GeoCoalition w ho developed this proposal

Cost Impact: Will not increase the cost of construction
The proposed code change w ill not increase the cost of construction. In certain cases, it w ill decrease the cost by allow ing more economical splices that still satisfy the service loading.

S226-16 : 1810.3.6-STEVENSON10943

S227-16

IBC: 1810.3.8.3.2, 1810.3.8.3.3.

Proponent : Satyendra Ghosh, representing Precast/Prestressed Concrete Institute (skghoshinc@gmail.com)

2015 International Building Code

Revise as follows:

1810.3.8.3.2 Seismic reinforcement in Seismic Design Category C. For structures assigned to *Seismic Design Category* C, precast prestressed piles shall have transverse reinforcement in accordance with this section. The volumetric ratio of spiral reinforcement shall not be less than the amount required by the following formula for the upper 20 feet (6096 mm) of the pile.

$\cancel{\rho}\underline{\rho_s = 0.04}\cancel{s}\underline{(f'_c/f_{yh})}\cancel{= 0.12}\underline{[2.8}\cancel{f}\underline{+}\cancel{'}\underline{2.34}\cancel{c}\underline{P/}\cancel{(}\underline{(}\cancel{f_{yh}}\underline{f'_c A_g)]}$ **(Equation 18-5)**

where:

f'_c	=	Specified compressive strength of concrete, psi (MPa).
f_{yh}	=	Yield strength of spiral reinforcement ≤ 85,000 psi (586 MPa).
$\underline{P}$	<u>=</u>	<u>Axial load on pile, pounds (kN), as determined from Equations 16-5 and 16-7.</u>
$\underline{A_g}$	<u>=</u>	<u>Pile cross-sectional area, square inches (mm^2).</u>
ρ_s	=	Spiral reinforcement index ~~(~~ <u>or volumetric ratio</u> (vol. spiral/vol. core).

At least one-half the volumetric ratio required by Equation 18-5 shall be provided below the upper 20 feet (6096 mm) of the pile.

> <u>**Exception:** The minimum spiral required by Equation 18-5 shall not apply in cases where the design includes full consideration of load combinations specified in ASCE/SEI 7 Section 12.4.3.2 and the applicable overstrength factor, Ω_0. In such cases, minimum spiral shall be as specified in 1810.3.8.1.</u>

1810.3.8.3.3 Seismic reinforcement in Seismic Design Categories D through F. For structures assigned to *Seismic Design Category* D, E or F, precast prestressed piles shall have transverse reinforcement in accordance with the following:

1. Requirements in ACI 318, Chapter 18, need not apply, unless specifically referenced.
2. Where the total pile length in the soil is 35 feet (10 668 mm) or less, the lateral transverse reinforcement in the ductile region shall occur through the length of the pile. Where the pile length exceeds 35 feet (10 668 mm), the ductile pile region shall be taken as the greater of 35 feet (10 668 mm) or the distance from the underside of the pile cap to the point of zero curvature plus three times the least pile dimension.
3. In the ductile region, the center-to-center spacing of the spirals or hoop reinforcement shall not exceed one-fifth of the least pile dimension, six times the diameter of the longitudinal strand or 8 inches (203 mm), whichever is smallest.
4. Circular spiral reinforcement shall be spliced by lapping one full turn and bending the end of each spiral to a 90-degree hook or by use of a mechanical or welded splice complying with Section 25.5.7 of ACI 318.
5. Where the transverse reinforcement consists of circular spirals, the volumetric ratio of spiral transverse reinforcement in the ductile region shall comply with the following:

   $\rho_s = \cancel{0.25}\ \underline{0.06}(f'_c/f_{yh})\cancel{(A_g/A_{ch} - 1.0)}[\cancel{0.5}\ \underline{2.8} + \cancel{1.4}\ \underline{2.34}P/(f'_c A_g)]$ **(Equation 18-6)**

   _but ~~not less than~~

$\cancel{\rho_s = 0.12(f'_c / f_{yh})[0.5 + 1.4P/(f'_c A_g)]\ 3\ 0.12 f'_c / f_{yh}}$ ~~**(Equation 18-7)**~~

~~and~~ need not exceed:

$\rho_s = 0.021$ **(Equation ~~18-8~~ <u>18-7</u>)**

where:

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

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

<u>**Exception:** The minimum spiral required by Equation 18-6 shall not apply in cases where the design includes full consideration of load combinations specified in ASCE/SEI 7 Section 12.4.3.2 and the applicable overstrength factor, Ω_0. In such cases, minimum spiral shall be as specified in 1810.3.8.1.</u>

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

$A_{sh} = 0.3s\, h_c\, (f'_c / f_{yh})(A_g / A_{ch} - 1.0)[0.5 + 1.4P/(f'_c A_g)]$ **(Equation ~~18-9~~ <u>18-8</u>)**

but not less than:

$A_{sh} = 0.12s\, h_c\, (f'_c / f_{yh})[0.5 + 1.4P/(f'_c A_g)]$ **(Equation ~~18-10~~ <u>18-9</u>)**

where:

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

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

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

Reason: 1) Replacement of Eq. (18-5): Recent research (Fanous et al., 2010; Sritharan et al., 2016) considered the

relationship among curvature ductility demand on prestressed piles and overall system ductility demand in the context of all soil profiles identified in ASCE/SEI 7-10, and concluded that the equation presented above would result in curvature ductility capacities exceeding 12, which has been established as a minimum limit needed for areas of moderate seismicity. In the lower portion of the element, the required reinforcement is reduced by one-half.

2) New exception in Section 1810.3.8.3.2: The exception statement is similar to other overstrength statements in this code and referenced load and material standards. It recognizes that the volumetric ratio of spiral reinforcement required need not be increased, beyond that required for driving and handling stresses, when the pile foundation system is designed for load combinations including overstrength. The minimum spiral reinforcement required per Section 1810.3.8.1 for driving and handling stresses is the minimum spiral reinforcement required for Seismic Design Categories A and B. In summary, when design includes the effect of overstrength, the increased axial forces, shear forces, and bending moments in the piling provide a large factor of safety against nonlinear pile behavior.

3) Replacement of Eq. (18-6): Findings from post-earthquake foundation evaluations as discussed in the literature, and concern over the accuracy of soil-structure interaction models under seismic loading, including the effects of liquefaction, has led to stringent code provisions that require significant pile ductility in the top 35 ft of the pile. Recent research (Fanous et al., 2010; Sritharan et al., 2016) considered the relationship among curvature ductility demand on prestressed piles and overall system ductility demand in the context of all soil profiles identified in ASCE/SEI 7-10, and concluded that the replacement equation proposed for Equation (18-6) will result in curvature ductility capacities exceeding 18 (based on average curvature ductility capacity minus one standard deviation), which has been established as a minimum limit needed for areas of high seismicity. The replacement for Equation (18-6) provides a volumetric steel ratio that is 50 percent higher than that required for SDC C. As a comparison, the highest codified ductility demand for buildings is in the New Zealand Standard (NZS 3101, 2006) which requires designs to be based on a curvature ductility capacity of 20. Similarly, ATC-32 (1996) sets the curvature ductility capacity target for vertical compression members at 13 with the expectation that 50 percent more capacity is available (i.e., maximum available curvature ductility is 19.5).

When the pile is designed for load combinations, including overstrength, the volumetric ratio of spiral reinforcement need not be increased beyond that required for driving and handling stresses.

This proposal is based on a prescriptive design philosophy that requires spiral confinement in accordance with maximum expected pile curvature ductility demands resulting from the design earthquake. Fanous et al. (2010), Sritharan et al. (2016) determined that the spiral ratio required could be expressed as a function of curvature ductility capacity of the prestressed pile as follows:

$$\rho_s = 0.06(f'_c/f_{yh})(\mu_\varphi/18)[2.8 + 1.25P/(0.53f'_c A_g)]$$

where μ_φ is the ductility capacity of the prestressed pile.

4) Deletion of Eq. (18-7): This equation is not part of the recommendation in Fanous et al. (2010), Sritharan et al. (2016).

5) New exception in Section 1810.3.8.3.3 Item 5: The exception statement is similar to other overstrength statements in this code and referenced load and material standards. It recognizes that the volumetric ratio of spiral reinforcement required should not be increased, beyond that required for driving and handling stresses, when the pile foundation system is designed for load combinations including overstrength. The minimum spiral reinforcement required per Section 1810.3.8.1 for driving and handling stresses is the minimum spiral reinforcement required for Seismic Design Categories A and B. In summary, when the design includes the effect of overstrength, the increased axial forces, shear forces, and moments in the piling provide a large factor of safety against nonlinear pile behavior.

The above changes are currently being balloted by ACI 318 Subcommittee F. They are expected to be approved by

ACI 318F and the full ACI 318 committee. These w ill then be part of ACI 318-19 and also the 2021 IBC, w hen that code adopts ACI 318-19. This proposal is being submitted because the proponent believes that the changes are ready to be implemented in the 2018 IBC, w ithout w aiting unnecessarily for the 2021 IBC.

Direct comparisons of the spiral required by the provisions of the 2015 IBC and the proposed modified requirements are provided in the supporting documentation accompanying this proposal. ISU (Iow a State University) report in the supporting documentation refers to Fanous et al. (2010). The pile sizes and shapes (24" and 16" Octagonal, 16" and 14" Square) are those included in Fanous et al. (2010). In the vast majority of cases the proposed sprial quantity exceeds the current IBC requirement.

Bibliography: ATC (Applied Technology Council). 1996. *Improved Seismic Design Criteria for California Bridges: Provisional Recommendations*. Redw ood City, CA: ATC.

Fanous, A., S. Sritharan, M. Suleiman, J. Huang, and K. Arulmoli. 2010. "Minimum Spiral Reinforcement Requirements and Lateral Displacement Limits for Prestressed Concrete Piles in High Seismic Regions." Final report to PCI. ISU-ERI-Ames Report ERIERI- 10321, Department of Civil, Construction, and Environmental Engineering, Iow a State University, Ames, Iow a.

Sritharan, S., A. M. Cox, J. Huang, M. Suleiman, and K. Arulmoli. 2016. "Minimum Confinement Reinforcement for Prestressed Concrete Piles and a Rational Seismic Design Framew ork." *PCI Journal*,Precast/Prestressed Concrete Institute, Chicago, IL. V. 61, No. 1 (January–February), pp. 51-70. http://w w w .pci.org/pci_archives.aspx

Standards New Zealand. 2006. *Concrete Structures Standard: Part 1—The Design of Concrete Structures*. NZS 3101. Wellington, New Zealand.

Standards New Zealand. 2006. *Part 2—Commentary on the Design of Concrete Structures*. NZS 3101. Wellington, New Zealand.

Cost Impact: Will increase the cost of construction
As the comparisons of current and proposed provisions in the supporting documentation show , the cost of precast concrete piles w ith confinement reinforcement in the form of spirals is likely to go up modestly in many situations, because of the increased confinement requirements proposed.

S227-16 : 1810.3.8.3.2-GHOSH12709

S228-16

IBC: 1810.3.9.3, 1810.3.9.3 (New).

Proponent : Charles Clawson, Pieresearch, representing Pieresearch (clawson6219@hotmail.com)

2015 International Building Code

Revise as follows:

1810.3.9.3 Placement of reinforcement. Reinforcement where required shall be assembled and tied together and shall be placed in the deep foundation element as a unit before the reinforced portion of the element is filled with concrete.

~~**Exceptions:**~~

~~1.~~ ~~Steel dowels embedded 5 feet (1524 mm) or less shall be permitted to be placed after concreting, while the concrete is still in a semifluid state.~~

~~2.~~ ~~For deep foundation elements installed with a hollow-stem auger, tied reinforcement shall be placed after elements are concreted, while the concrete is still in a semifluid state. Longitudinal reinforcement without lateral ties shall be placed either through the hollow stem of the auger prior to concreting or after concreting, while the concrete is still in a semifluid state.~~

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

Not less than 3 inches of concrete cover shall be maintained on sides and bottom of the reinforcing bar unit. The reinforcing bar unit shall be placed in the element using non-corrosive side and bottom spacers/centering devices. Side spacer/centering devices shall start 18 inches from the top and bottom of the reinforcing bar unit and spaced vertically at intervals not exceeding 10 feet. Cementitious bottom supports shall be uniformly spaced in accordance with Table 1810.3.9.3.

For plastic bottom supports provide one support on every other vertical bar for units under 48 inches in diameter. For cages over 48 inches in diameter use one support on each vertical bar.

Exceptions:

1. Steel dowels embedded 5 feet (1524 mm) or less shall be permitted to be placed after concreting, while the concrete is still in a semifluid state.
2. For deep foundation elements installed with a hollow-stem auger, tied reinforcement shall be placed after elements are concreted, while the concrete is still in a semifluid state. Longitudinal reinforcement without lateral ties shall be placed either through the hollow stem of the auger prior to concreting or after concreting, while the concrete is still in a semifluid state.
3. For Group R-3 and U occupancies not exceeding two stories of light-frame construction, reinforcement is permitted to be placed after concreting, while the concrete is still in a semifluid state, and the concrete cover requirement is permitted to be reduced to 2 inches (51 mm), provided the construction method can be demonstrated to the satisfaction of the *building official*.

Add new text as follows:

TABLE 1810.3.9.3
Cementitious Bottom Supports

Shaft Diameter In Inches	12" to 30"	31" to 48"	49" to 72"	73" to 96"	97" to 120"
Quantity Required Per Shaft	3	6	8	10	12

Reason: There is no provision in the building code that requires a reinforcing steel cage to be properly positioned in a drilled shaft by mechanical means. Accordingly, in the absence of specific requirements in the construction documents, the special Inspector has no authority to require use of centering devices or other mechanical means designed to ensure that the cage will be properly positioned in said shaft. The positioning of the rebar cage in the drilled shaft is the only component of any structure that when completed cannot be tested or inspected to see that the cage is properly positioned. When the cage is not properly positioned the reinforceing steel will not perform as designed thus possibly compromising the performance of the drilled shaft. When the cage is not properly positioned, the potential for increased corrosion and associated concrete deterioration will also be increased.
Below are examples of Rebar Cage centering requirements by various industry associations and governmental agencies substantiating the need for the above code revision:

American Concrete International (ACI), committee 336.1 "Specifications for the Construction of Drilled Piers, Section 3.4.6: "Place bars as shown on Project Drawings with cover of not less than 3 in.(75mm) where exposed to soil and not less that 4 in (100mm) in cased piers where the casing is to be withdrawn. Provide spacer rollers to maintain cover."

ADSC: The International Association of Foundation Drilling Standards and Specifications incorporates the above ACI language

Federal Highway Administration FHWA-NHI-10-016 8-Rebar Cages FHWA CEC 010 Drilled Shafts Manual May 2010

Chapter 8-Rebar Cages-page 8-15

8.8 CENTERING DEVICES--paragraph 2

"Centering devices should be composed of rollers aligned so as to allow the travel of the cage along the wall of the drilled shaft excavation without dislodging soil or debris and causing the accumulation of loose material in the bottom of the excavation prior to concrete placement. These rollers are typically constructed of plastic, concrete , or mortar: they should not be faricated of steel in such a way that a corrosion path to the reinforcement could be introduced."

Texas Department of Transportation, Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, Adopted by the Texas Department of Transportation November 1, 2014.

Item 416 Section 3.5, Drilled Shaft Foundations, Reinforcing Steel

"Center the reinforcing steel cage in the ecavation using aproved "roller" type centering devices unless otherwise approved. Use concrete or plastic chairs to keep the reinforcing cage off of the bottom of the hole. Use centering devices starting at 1.5ft. off from the bottom of the cage and spaced vertically at intervals not exceeding 10ft. Use a minimum of 3 centering devics per level at a spacing not to exceed 30in".

Other states and Governmental agencies have adopted similiar language that requires bottom and side spacers to maintain proper spacing and steel protection. Some of the other agencies are as follows:

Colorado Department of Transportation, 2011 Construction Specification Book, 503.05 Reinforcing Steel

Illinois Department of Transportation, Standard Specifications for Road and Bridge Construction, Section 516 Drilled Shafts

Louisiana Department of Transportation and Developmentm, Standard Specifications, Section 81414(d) reinforcing Steel Construction and

positioning-spacers

Massachusetts Department of Transportation-Highway Division, 945.58 Steel Reinforcement Configuration and Placement

Michigan Department of Transportation, Standard Specification for Construction, Section 718G Drilled Shafts-Placing Steel Reinforcement- para-

graph 5&6

New Jersey Department of Transportation, 2007 Standard specifications for Road and Bridge Construction, Section 503-Drilled Shaft Foundations

The above code change is to insure that the designer puts into the specification methods to insure that the steel is properly centered in the pier shaft.

Cost Impact: Will increase the cost of construction
If the drilled pier construction is properly designed and installed similar to slab foundations, centering devices would normally be included in the Engineering design specifications. In this case, **no additional cost** would occur.
If centering devices are not called for in the specifications, then the cost of adding the positioning devices, including installation and shipping, is approximately $39.00 perpier. This estimate is based on a 30" diameter shaft and 20' deep pier. Lesser or increased cost will depend on the size and depth of the piers.

The lack of centering steel in concrete is an easy target for a forensic engineer's determination of foundation or pier failure. Not included in any cost calculation is the additional expense of the inspector's or special inspector's time to insure the centering of the rebar steel if positioning devices are not used.

S228-16 : 1810.3.9.3-CLAWSON10968

S229-16

IBC: 202, 1802.1, 1810.3.9.6, 1810.4.9.

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; Woodward Vogt, representing GeoCoalition (woody@paradigmconsultants.com)

2015 International Building Code

Revise as follows:

SECTION 202 DEFINITIONS

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

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

Caisson Piles. Steel cased piles constructed by advancing a steel shell seated into rock and drilling of an uncased socket within the rock. The shell and socket are filled with a steel core section or steel reinforcing and concrete or grout.

1802.1 Definitions. The following words and terms are defined in Chapter 2:

DEEP FOUNDATION.
DRILLED SHAFT.
~~**Socketed drilled shaft.**~~ **CAISSON**
HELICAL PILE.
MICROPILE.
SHALLOW FOUNDATION.

1810.3.9.6 ~~**Socketed drilled shafts**~~ **Caisson piles**. ~~Socketed drilled shafts~~ Caisson piles shall have a permanent~~ pipe or tube~~ casing that extends down to bedrock and an uncased socket drilled into the bedrock, both filled with concrete. ~~Socketed drilled shafts~~ Caisson piles shall ~~have~~ include reinforcement or a structural steel core for the full length as ~~indicated~~ required by ~~an~~ the~~*approved*~~ *registered design professional*~~ method of analysis~~.

The depth of the rock socket shall be sufficient to develop the full load-bearing capacity of the element~~ with a minimum safety factor of two~~, but the depth shall not be less than the outside diameter of th~~e pipe or tube~~ casing. The design of the rock socket is permitted to be predicated on the sum of the allowable load-bearing pressure on the bottom of the socket plus bond along the sides of the socket.

Where a structural steel core is used, the gross cross-sectional area of the core shall not exceed 25 percent of the gross area of th~~e drilled shaft~~ caisson pile.

1810.4.9 ~~**Socketed drilled shafts**~~ **Caisson piles.** The rock socket ~~and pipe or tube~~ casing of ~~socketed drilled shafts~~ caisson piles shall be thoroughly cleaned of foreign materials before filling with concrete. Steel cores shall be bedded in cement grout at the base of the rock socket.

Reason: The term "socketed drilled shafts" has caused great confusion to many engineers. The region where this type foundation element is commonly used (New York City) still uses the term "caisson pile" (**and that was the term used by IBC prior to 2009**) in their building code. This proposal includes changing the term "Socketed drilled shaft" in the definitions of Chapter 2 and Section 1802 to "Caisson Pile". The suggested definition comes directly from the New York City building code.
Proposed changes in 1810.3.9.6 and 1810.4.9 also contain the proposed change of this term and were made to avoid further confusion in the industry with conventional drilled shafts. A "casing" (1810.3.9.6 and 1810.4.9) is by

definition a pipe or a tube, so that w ording can be deleted to simplify and shorten the code.

In 1810.3.9.6, the focus is to be shifted to the registered design professional. This professional w ill use an appropriate method of analysis. It is clarified that the intent is to have the reinforcement or core for the entire length, including the rock socket. Minimum factors of safety have been addressed elsew here in the code.

Click here to view the members of the GeoCoalition w ho developed this proposal

Cost Impact: Will not increase the cost of construction
The code change proposal w ill not increase the cost of construction because it is just a change in name only.

S229-16 : 1810.3.9.6-BIGGERS10697

S230-16

IBC: 1810.4.1.1.

Proponent : E. Anna Sellountou, PhD,PE, representing GeoCoalition; Lori Simpson, P.E.,G.E., representing GeoCoalition; Daniel Stevenson, P.E., representing GeoCoalition; Dale Biggers, P.E., Boh Bros. Construction Co.,L.L.C., representing GeoCoalition (dbiggers@bohbros.com)

2015 International Building Code

Revise as follows:

1810.4.1.1 Compressive strength of ~~precast~~ driven concrete piles. A ~~precast~~ driven concrete pile shall not be driven before the concrete has attained a compressive strength of at least 75 percent of the specified compressive strength (f'_c), but not less than the strength sufficient to withstand handling and driving forces.

Reason: The code specifically mentions and is about "driven" piles and "driven" should be in the section title. Driven piles are by definition "precast", so the term is redundant. Driven piles are either regularly reinforced or prestressed and both types should meet these requirements.

Click here to view the members of the GeoCoalition w ho developed this proposal

Cost Impact: Will not increase the cost of construction
The code change proposal w ill not increase the cost of construction, as it just removes the vague term "precast".

S230-16 : 1810.4.1.1-SELLOUNTOU10690

S231-16

IBC: 1810.4.1.2.

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~~ , the 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 w ords "concrete is placed in an open drilled hole" are redundant since the sentence begins w ith 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 elsew here.
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Cost Impact: Will not increase the cost of construction
The change w ill not increase costs and in some cases w ill reduce costs by allow ing alternate construction techniques.

S231-16 : 1810.4.1.2-SELLOUNTOU10691

S232-16

IBC: 1810.4.1.3.

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~~ from 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~~ The effects of potential heave should be ~~installed in soils that could cause heave~~ considered by the registered design professional.

Reason: The changes in w ording 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 w ho is responsible for consideration of heave and the determination of how much heave is acceptable. It is not clear w hat a driven uncased element is.

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Cost Impact: Will not increase the cost of construction
This proposal w ill 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.

S232-16 : 1810.4.1.3-SELLOUNTOU10692

S233-16

IBC: 1810.4.1.5.

Proponent : E. Anna Sellountou, PhD,PE, representing GeoCoalition; Lori Simpson, P.E.,G.E., representing GeoCoalition; Dale Biggers, P.E., Boh Bros. Constructuion Co.,L.L.C., representing GeoCoalition (dbiggers@bohbros.com); Daniel Stevenson, P.E., representing GeoCoalition

2015 International Building Code

Revise as follows:

1810.4.1.5 Defective timber piles. Any substantial sudden ~~increase~~ change in rate of penetration of a timber pile shall be investigated for possible damage. If the sudden ~~increase~~ change in rate of penetration cannot be correlated to soil strata, the pile shall be removed for inspection or rejected.

Reason: Either sudden increases or decreases in the rate of penetration of a pile may signal pile damage.

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Cost Impact: Will not increase the cost of construction
The code change proposal w ill not increase the cost of construction because it does not change current practice.

S233-16 : 1810.4.1.5-SELLOUNTOU10693

S234-16

IBC: 1810.4.10.

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:

1810.4.10 Micropiles. Micropile deep foundation elements shall be permitted to be formed in holes advanced by rotary or percussive drilling methods, with or without casing. The elements shall be grouted with a fluid cement grout. The grout shall be pumped through a tremie pipe extending to the bottom of the element until grout of suitable quality returns at the top of the element. The following requirements apply to specific installation methods:

1. For micropiles grouted inside a temporary casing, the reinforcing bars shall be inserted prior to withdrawal of the casing.The casing shall be withdrawn in a controlled manner with the grout level maintained at the top of the element to ensure that the grout completely fills the drill hole. During withdrawal of the casing, the grout level inside the casing shall be monitored to verify that the flow of grout inside the casing is not obstructed.
2. For a micropile or portion thereof grouted in an open drill hole in soil without temporary casing, the minimum design diameter of the drill hole shall be ~~verified~~ evaluated during grouting by a suitable ~~device during grouting~~ method *approved* by the *registered design professional*.
3. For micropiles designed for end bearing, a suitable means *approved* by the *registered design professional*shall be employed to ~~verify~~ assess that the bearing surface is properly cleaned prior to grouting. Suitable means can include cleaning by injection of compressed air, tremie-injected water or grout.
4. Subsequent micropiles shall not be drilled near elements that have been grouted until the grout has had sufficient time to ~~harden~~ achieve initial set and strength as required by the *registered design professional*.
5. ~~Micropiles~~ After drilling is completed, micropiles shall be grouted as soon as ~~possible after drilling is completed~~ necessary as determined by the *registered design professional*.
6. For micropiles designed with a full-length casing, the casing shall be pulled back to the top of the bond zone and reinserted or some other suitable means employed to assure grout coverage outside the casing.
7. The installation method shall be appropriate for the soil or rock conditions encountered, to advance the hole, and to prevent drill hole instability or ground loss.
8. Grout strength shall be verified as specified by the *registered design professional*.
9. Reinforcing bars shall be free of deleterious substances and have firmly fixed and suitable centralizers.
10. Permanent casing threaded joints shall be sufficiently torqued to seat the pipe thread shoulders prior to drilling.

Reason: Item 2 – Word change from "verified" to "evaluated" eliminates a w ord no longer commonly used by industry professionals. The focus has also been shifted to the registered design professional. The follow ing examples are then suggested for the Commentary ("Suitable means could include direct measurement or estimation of tremie and pressure-injected grout volumes").
Item 3 – Word change from "verify" to "assess" eliminates a w ord no longer commonly used by industry

professionals. The focus has also been shifted to the registered design professional. The following examples are then suggested for the Commentary ("Suitable means could include cleaning by injection of compressed air, tremie-injected water or grout").

Item 4 – The suggested code change provides clarification to the word "harden." In order to construct new micropiles adjacent to previously drilled and grouted micropiles, it is necessary for the neat cement grout to achieve initial set and reach an initial compressive strength (commonly accepted within the state of practice as about 1,000 psi). The focus has also been shifted to the registered design professional.

Item 5 – The suggestion to change the word "possible" to "necessary" recognizes that not all projects require immediate grouting. The focus has also been shifted to the registered design professional.

Item 7 – Micropile construction equipment, tooling, and procedures have a significant effect on design and performance, and must be carefully selected to reduce the potential for instability, particularly in soil below the static groundwater table. The Code should explicitly recognize the link between construction and performance.

Item 8 – Grout strength is an important element of the structural strength of the micropile. Several methods are available to test grout strength. The registered design professional is responsible for all aspects of the micropile design. The Commentary could address the following: "Several methods are available to test grout strength. Cured samples can be subjected to compression tests. Specific gravity measurements conducted in accordance with ASTM D4380 or API 13B-1 provide a rapid field method for evaluating likely long term compressive strength and establishing the suitability of the grout for installation in the micropile."

Item 9 – It is important that the bars be clean and centered in the pile.

Item 10 – Structural performance of the cased portion of a micropile relies on proper threading and tight shouldering of flush joint pipe sections. This process occurs above ground surface and can be visually observed by the drill operator and representatives of the registered design processional or other third-party construction monitoring personnel.

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Cost Impact: Will not increase the cost of construction
These code change proposals will not increase the cost of construction. Good installation practice already incorporates these items.

S234-16 : 1810.4.10-SELLOUNTOU10698

S235-16

IBC: 1810.4.11.

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:

1810.4.11 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 w ith 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 "allow able". 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.
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Cost Impact: Will not increase the cost of construction
The code change proposal w ill not increase the cost of construction since it is a clarification to the code.

S235-16 : 1810.4.11-SELLOUNTOU10703

S236-16

IBC: 1810.4.13 (New).

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

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 D5882 - 07 (2013) Standard Test Method for Low Strain Impact Integrity Testing of Deep Foundations
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 allow able stress table 1810.3.2.6 w ould suggest some verification is appropriate. Some foundation types or circumstances require no integrity testing and the registered design professional is w ell equipped to distinguish w hen such testing is needed.
ASTM D4945 is "Standard Test Method for High-Strain Dynamic Testing of Deep Foundations."
ASTM D5882 is "Standard Test Method for Low Strain Impact Integrity Testing of Deep Foundations."
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."

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Cost Impact: Will not increase the cost of construction
The code change proposal w ill 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, w ith regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) w ill be posted on the ICC w ebsite on or before April 1, 2016.

S236-16 : 1810.4.13 (NEW)-SELLOUNTOU10719

S237-16

IBC: 1810.4.4.

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:

1810.4.4 Preexcavation. The use of jetting, augering or other methods of preexcavation shall be subject to the approval of the *building official*. Where permitted, preexcavation shall be carried out in the same manner as used for deep foundation elements subject to load tests and in such a manner that will not impair the carrying capacity of the elements already in place or damage adjacent structures. Element tips shall be ~~driven~~ advanced below the preexcavated depth until the required resistance or penetration is obtained.

Reason: Pile types in addition to driven piles should also meet this requirement, e.g. piles that are screw ed in, pushed in, or vibrated in.

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Cost Impact: Will not increase the cost of construction
The code change proposal w ill not increase the cost of construction since this is already common practice.

S237-16 : 1810.4.4-SELLOUNTOU10694

S238-16

IBC: 1810.4.5.

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 w ith vibratory hammer and then completed w ith an impact hammer have other means of verifying capacity. Piles that are installed only and fully w ith a vibratory hammer w ill still be required to have an axial load test, w hen axial loads are required.
A load test for axial capacity is not needed for piles used for lateral resistance and w ould not confirm lateral performance of the pile. Lateral load testing requirements, if needed, are specified in Section 1810.3.3.2.

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Cost Impact: Will not increase the cost of construction
The code change proposal w ill not increase the cost of construction. It can decrease the cost of construction if the piles are used only for lateral resistance.

S238-16 : 1810.4.5-SELLOUNTOU10695

S239-16

IBC: 1810.4.6.

Proponent : Dale Biggers, P.E., 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:

1810.4.6 Heaved elements. Deep foundation elements that have heaved more than 1/2 inch for end-bearing piles or more than 1 1/2 inch where shaft resistance is dominent during the driving of adjacent elements shall be redriven ~~as~~ if directed by the *registered design professional* when necessary to develop the required capacity and penetration, or the capacity of the element shall be verified by load tests in accordance with Section 1810.3.3.1.2.

Reason: There needs to be guidance for a reasonable tolerance as to the amount of acceptable heave. AASHTO (American Association of State Highw ay Transportation Organizations) and PDCA (Pile Driving Contractors Association) Installation Specifications both recommend these limits.

The follow ing section 4.4.3.1 for Heaved Piles is directly copied from the AASHTO LRFD Bridge Construction Specifications, 3rd Edition (2010). The document is copyrighted and no URL link is therefore allow ed. It can be purchased from FHWA. The actual text below w as furnished to the Proponents by Silas Nichols, Principal Bridge Engineer - Geotechnical, FHWA (Federal Highw ay Administration):

4.4.3.1—Heaved Piles

If pile heave is observed, level readings referenced to a fixed datum shall be taken by the Engineer on all piles immediately after installation and periodically thereafter as adjacent piles are driven to determine the pile heave range.
During the driving process for adjacent piles, piles shall be redriven:

• if they heave more than 0.5 in. and end bearing is dominant, or

• if they heave more than 1.5 in. and shaft friction is dominant.

If pile heave is detected for pipe or shell piles that have been filled w ith concrete, the piles shall be redriven to original position after the concrete has obtained sufficient strength, and a proper hammer-pile cushion system, satisfactory to the Engineer, is used. The Contractor shall be paid for all w ork performed in conjunction w ith redriving piles because of pile heave provided the initial driving w as done in accordance w ith the specified installation sequence.

Follow ing from: INSTALLATION SPECIFICATION FOR DRIVEN PILES JANUARY 2007

RECOMMENDED BY PDCA (PILE DRIVING CONTRACTORS ASSOCIATION)

4.3.1 Heaved Piles

If pile heave is observed, level readings referenced to a fixed datum shall be taken by the Engineer on all piles immediately after installation and periodically thereafter as adjacent piles are driven to determine the pile heave range.
If during the driving process for adjacent piles, piles shall be re-driven:
• If they heave more than ½ inch (13 mm) and end bearing is dominant.
• If they heave more than 1-½ inches (38 mm) and shaft friction is dominant.
If pile heave is detected for pipe or shell piles w hich have been filled w ith concrete, the piles shall be redriven to original position after the concrete has obtained sufficient strength and a proper hammer pile cushion system, satisfactory to the Engineer, is used. The Contractor shall be paid for all w ork performed in conjunction w ith redriving piles due to pile heave, provided the initial driving w as done in accordance w ith the specified installation sequence.

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Cost Impact: Will not increase the cost of construction
This proposal w ill not increase the cost of construction. It w ill possibly decrease the cost of construction by allow ing some heave, depending on the type of soil.

S239-16 : 1810.4.6-BIGGERS11550

S240-16

IBC: 1802.1, 1810.4.8, 202 (New).

Proponent : Dale Biggers, P.E., 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

Add new text as follows:

SECTION 202 DEFINITIONS

[BS] AUGERED CAST-IN-PLACE (ACIP) ELEMENT. A cast-in-place deep foundation element, such as hollow-stem augered, cast-in-place elements, continuous flight auger pile (CFA), auger pressure grouted piles (APG), augercast piles, or drilled displacement piles, constructed by drilling a hole into soil and/or rock using a hollow stem auger with continuous flights. Upon reaching the desired depth, the auger is extracted as concrete or grout is pumped through the hollow stem.

Revise as follows:

1802.1 Definitions. The following words and terms are defined in Chapter 2:

AUGERED CAST-IN-PLACE (ACIP) ELEMENT.
DEEP FOUNDATION.
DRILLED SHAFT.
Socketed drilled shaft.
HELICAL PILE.
MICROPILE.
SHALLOW FOUNDATION.

1810.4.8 Hollow-stem augered, cast-in-place elements. Where concrete or grout is placed by pumping through a hollow-stem auger, or placed by pumping through a hollow-stem displacement tool, the auger or tool shall be permitted to rotate in a clockwise direction during withdrawal. As the auger or tool is withdrawn at a steady rate or in increments not to exceed 1 foot (305 mm), concreting or grouting pumping pressures shall be measured and maintained high enough at all times to offset hydrostatic and lateral earth pressures. Concrete or grout volumes shall be measured to ensure that the volume of concrete or grout placed in each element is equal to or greater than the theoretical volume of the hole created by the auger or tool. Where the installation ~~process~~ process of any element is interrupted or a loss of concreting or grouting pressure occurs, the element shall be redrilled to 5 feet (1524 mm) below the elevation of the tip of the auger when the installation was interrupted or concrete or grout pressure was lost and reformed. Augered and displacement cast-in-place elements shall not be installed ~~within~~ less than six diameters center to center ~~of~~ from an element filled with concrete or grout less than 12 hours old, unless *approved* by the *building official*. For displacement type elements this minimum installation spacing shall be reviewed by the *registered design professional*. If the concrete or grout level in any completed element drops ~~due to installation of an adjacent element~~, the completed element shall be reviewed by the registered design professional to determine if it shall be replaced.

Reason: There is no existing definition for this common type of deep foundation.
The purpose of the proposed code addition to Chapter 2 Definitions is to identify another commonly-used type of deep foundation along with drilled shafts, helical piles, and micropiles. The code uses "Hollow-stem augered, cast-in-place elements" which is not a common term. This suggested term is the common industry term for this foundation type (section 1810.4.8 should be changed to reflect this better name).

• Hollow-stem can be either conventional full length flighting, or of the displacement type. The displacement type was specifically mentioned to assure these provisions also apply to that common type of hollow-stem auger. Additional review for the displacement was also assigned to the registered design professional.

• The phrase "at a reduced rate" was added for clarity because the rate of auger rotation needs to be slowed to minimize caving and prevent excessive mining of the soils surrounding the auger. A slower rate of auger rotation during withdrawal is proper practice.

• The phrase "elements shall not be installed within six diameters center to center of an element" has been changed to "elements shall not be installed less than six diameters center to center from an element" to clarify that it is permitted to install elements spaced six diameters apart on the same day (less than 12 hours apart), as this is current industry practice.

• The sentence "If the concrete or grout level in any completed element drops due to installation of an adjacent element, the element shall be replaced" has been revised to "If the concrete or grout level in any completed element drops, the completed element shall be reviewed by the registered design professional to determine if it shall be replaced". The phrase "due to installation of an adjacent element" was deleted because the grout level can drop for several different reasons. It is common for the grout level in cast-in-place elements to drop slightly due to water loss, and this should not automatically be cause for the element to be replaced. The registered design professional should make this decision; small drops are usually considered insignificant. The word "completed" has been added to clarify which element is to be replaced.

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Cost Impact: Will not increase the cost of construction
The change will not increase costs, as it only clarifies existing code provisions and describes proper installation procedures. The cost also is not changed due to name change which better reflects industry terminology.

S240-16 : 1810.4.8-BIGGERS10696

S241-16

IBC: 1804.2 (New), 1811 (New), 1811.1 (New), 1811.1.1 (New), 1811.1.2 (New), 1811.2 (New), 1811.3 (New), 1811.4 (New), 1811.4.1 (New).

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 w alls 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 betw een temporary earth retention systems and underpinning. Both concepts are described in Section 1804, as revised by our proposals, as methods for protecting structures w hich are adjacent to excavations required for new construction. The proposal indicates w hen 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 w alls of excavated areas w here there is a need to contain earth movements in order to protect adjacent structures and public infrastructure elements as w ell as provide for the safety and w ell being of the public at large.

This proposal introduces the term "temporary earth retention system" and refers to the proposed new Section 1811 w hich describes the basic requirements for this type of system. It is much more common to use temporary earth retention systems to support the w alls 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.

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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 w alls of excavated areas during building construction in order to protect both the new construction and adjacent property reflects typical current practice. For projects w here it is recognized that there is a need to install a temporary earth retention system, the proposed code change w ill not increase the cost of construction. If a temporary earth retention system is not used and w as needed, the resulting damage w hich may occur to either the new construction or to adjacent structures and surrounding infrastructure elements can be extensive and very expensive to repair.

S241-16 : 1811 (NEW)-MAYNARD12713

S242-16

IBC: 1613.1, 1901.2.

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 Chapter 14~~ Chapters 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.

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 modrate-deformability connectors with the Basic Design Option may result in modest cost increases.

S242-16 : 1613.1-GHOSH11652

S243-16
Part I:
IBC: 202.
Part II:
IRC: R606.2.3.

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC-STRUCTURAL CODE COMMITTEE. PART II WILL BE HEARD BY THE IRC-BUILDING CODE COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

Proponent : Phillip Samblanet, representing The Masonry Society

Part I

2015 International Building Code

Delete without substitution:

~~**AUTOCLAVED AERATED CONCRETE (AAC).** AUTOCLAVED AERATED CONCRETE (AAC). Low density cementitious product of calcium silicate hydrates, whose material specifications are defined in ASTM C1386.~~

Reference standards type: This is an update to reference standard(s) already in the ICC Code Books
Add new standard(s) as follows:
~~ASTM C1386~~

Part II

2015 International Residential Code

Revise as follows:

R606.2.3 AAC masonry. AAC masonry units shall conform to ASTM C1691 and ASTM ~~C 1386~~ C1693 for the strength class specified.

Reference standards type: This contains both new and updated standards
Add new standard(s) as follows:
~~ASTM C1386~~
ASTM C1691- 11 Standard Specification for Unreinforced Autoclaved Aerated Concrete (AAC) Masonry Units

ASTM C1693-11 Standard Specification for Autoclaved Aerated Concrete (AAC)

Reason: The definition is not needed and is incorrect. ASTM C1386 was withdrawn by ASTM in 2013, and AAC is now manufactured to different ASTM standards (ASTM C1691 for AAC masonry and ASTM C1693 for AAC in general). In addition, IBC Section 202 already contains a definition for AAC Masonry, which is both more approrpriate and correct. While this definition could apply AAC as used in conjunction with Chapter 19, that Chapter does not address AAC. Deleting the definition of Autoclaved Aerated Concrete thus removes the reference to an ASTM standard no longer used, and it cleans up the IBC as a whole.
Part II updates references to it in the IRC. .

Cost Impact: Will not increase the cost of construction

Revision of this section does not impact the cost of construction. The definition is not needed, and the referenced standard has been withdrawn. The change merely eliminates this error in the IBC

Analysis:

Part II: A review of the standard(s) proposed for inclusion in the code, ASTM C1691 & ASTM C1693, 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.

S243-16 : R606.2.3-SAMBLANET5091

S244-16

IBC: 2101.2, 2103.1, 2104.1.

Proponent : Jason Thompson, Masonry Alliance for Codes and Standards (MACS), representing Masonry Alliance for Codes and Standards (jthompson@ncma.org); Phillip Samblanet, The Masonry Society, representing The Masonry Society

2015 International Building Code

Revise as follows:

2101.2 Design methods. Masonry shall comply with the provisions of TMS 402/ACI 530/ASCE 5<u>, TMS 403,</u> or TMS ~~403~~ <u>404</u> as well as applicable requirements of this chapter.

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<u> and TMS 504</u>.

> **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.

2104.1 Masonry construction. Masonry construction shall comply with the requirements of Sections 2104.1.1 and 2104.1.2 and with <u>the requirements of either</u> TMS 602/ACI 530.1/ASCE 6 <u>or TMS 604</u>.

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
TMS 404-16 – Standard for the Design of Architectural Cast Stone
TMS 504-16 – Standard for the Fabrication of Architectural Cast Stone

TMS 604-16 – Standard for the Installation of Architectural Cast Stone

Reason: Architectural cast stone is a non-structural masonry system typically used as architectural accents such as balusters, quoins, sills, etc. While Chapter 21 requires architectural cast stone to comply with the material requirements of ASTM C1364 and Chapter 14 includes minimum criteria for the use of architectural cast stone as a cladding system, the vast majority of design, fabrication, and installation guidance for these systems has historically stemmed from industry-generated best practices; a gap now filled with the creation of these three new standards. Topics covered collectively under these three new standards include:

1) Minimum requirements for reinforcement, ties, and anchors used with cast stone along with the associated corrosion protection requirements for these materials.

2) Additional requirements for cast stone materials not covered within ASTM C1364.

3) Tolerance requirements for individual cast stone elements as well as finished assemblies.

4) Information to be included in shop drawings and submittal packages.

5) Ancillary materials used during the installation of cast stone including mortar, grout, and jointing materials.

6) Minimum quality assurance requirements including testing frequency, sample panels, and inspection.

7) Installation criteria for both wet-setting (laying cast stone elements in mortar) as well as dry-setting (where cast stone units are shimmed and caulked).

Cost Impact: Will not increase the cost of construction
The addition of these new standards simply provides consensus-based guidance for the design, fabrication, and installation of cast stone consistent with existing industry guidlines.

Analysis: A review of the standard(s) proposed for inclusion in the code, TMS 404, TMS 504 & TMS 604, 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.

S244-16 : 2101.2-THOMPSON12338

S245-16

Part I:
IBC: 2103.1.
Part II:
IRC: R606.2.6 (New).

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC-BUILDING COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

Proponent : Jason Thompson, Masonry Alliance for Codes and Standards (MACS), representing Masonry Alliance for Codes and Standards (jthompson@ncma.org)

Part I

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. <u>Adhered manufactured stone masonry veneer units shall conform to ASTM C1670.</u>

> **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:
<u>ASTM C1670-15 Standard Specification for Adhered Manufactured Stone Masonry Veneer Units</u>

Part II

2015 International Residential Code

Add new text as follows:

<u>**R606.2.6 Adhered manufactured stone masonry veneer units.**</u> <u>Adhered manufactured stone masonry veneer units shall conform to ASTM C1670.</u>

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
<u>ASTM C1670-15 Standard Specification for Adhered Manufactured Stone Masonry Veneer Units</u>

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.

S245-16 : 2103.1-THOMPSON12357

S246-16

IBC: 202, 2104.1.3 (New).

Proponent : Jason Thompson, representing Masonry Alliance for Codes and Standards (jthompson@ncma.org)

2015 International Building Code

Revise as follows:

SECTION 202 DEFINITIONS

[BS] MASONRY. A built-up construction or combination of building units or materials of clay, shale, concrete, glass, gypsum, stone or other *approved*units bonded together with or without *mortar*or grout or other accepted methods of joining.

Glass unit masonry. Masonry composed of glass units bonded by *mortar*.

Masonry pilaster. A vertical masonry member, built integrally with a wall. Unreinforced masonry pilasters shall have a portion of its cross-section projecting from one or both faces of the wall.

~~**Plain masonry.**~~ **Plain masonry.** Masonry in which the tensile resistance of the masonry is taken into consideration and the effects of stresses in reinforcement are neglected.

~~**Reinforced masonry.**~~ **Reinforced masonry.** Masonry construction in which reinforcement acting in conjunction with the masonry is used to resist forces.

~~**Solid masonry.**~~ **Solid masonry.** Masonry consisting of solid masonry units laid contiguously with the ~~*joints*~~ *joints* between the units filled with ~~*mortar*~~*mortar*.

~~**Unreinforced (plain) masonry.**~~ **Unreinforced (plain) masonry.** Masonry in which the tensile resistance of masonry is taken into consideration and the resistance of the reinforcing steel, if present, is neglected.

Add new text as follows:

2104.1.3 TMS 402, Section 5.4, masonry pilasters. Modify Section 5.4 as follows:
5.4 - Pilasters. Walls interfacing with pilasters shall not be considered as flanges, unless the construction requirements of Sections 5.1.1.2.1 and 5.1.1.2.5 are met and the pilaster projects a minimum of 4 inches (102 mm) nominally from at least one face of the wall. When these construction requirements are met, the pilaster's flanges shall be designed in accordance with Sections 5.1.1.2.2 through 5.1.1.2.4.

Reason: There exists a perceived technical gap in the TMS 402 standard. Consider a long, flat unreinforced masonry assembly. There is no provision that would prohibit a designer from arbitrarily designating a portion of this unreinforced assembly a 'masonry pilaster' and taking advantage of the additional strength and stiffness afforded pilasters. Likewise, if this same assembly were to be grouted and reinforced at some specified spacing, there is no provision that prohibits arbitrarily designating one or more of those grouted and reinforced cells a pilaster. While it is unlikely there are dubious attempts to take advantage of this perceived loophole, sufficient concern has been raised to warrant this modification while the TMS 402 Committee continues to resolve this topic – hopefully before the conclusion of the Group B cycle.
This change accomplishes two primary goals:

1) For unreinforced masonry assemblies, pilasters must project from at least one surface of the masonry assembly, as this is the only feasible means of detailing an unreinforced pilaster.

2) A minimum pilaster projection of 4 inches is introduced. If this minimum projection is not provided this proposed change requires that the pilaster/wall flanging effects be ignored.

The language being added by this change proposal is shown here:

Walls interfacing with pilasters shall not be considered as flanges, unless the construction requirements of Sections 5.1.1.2.1 and 5.1.1.2.5 are met and the pilaster projects a minimum of 4 inches (102 mm) nominally from at least one

surface of the wall. When these construction requirements are met, the pilaster's flanges shall be designed in accordance with Sections 5.1.1.2.2 through 5.1.1.2.4.

Cost Impact: Will not increase the cost of construction
This change is largely a clarification of the historical inperpretation of masonry pilasters.

S246-16 : 2104.1.3 (NEW)-THOMPSON12561

S247-16

IBC: 2107.2.1.

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

2015 International Building Code

Revise as follows:

2107.2.1 Lap splices. The minimum length of lap splices for reinforcing bars in tension or compression, l_d, shall be

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

For SI: $l_d = 0.29 d_b f_s$

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

where:

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

In regions of moment where the design tensile stresses in the reinforcement are greater than 80 percent of the allowable steel tension stress, F_s, the lap length of splices shall be increased not less than 50 percent of the minimum required length, but need not be greater than $72d_b$. Other equivalent means of stress transfer to accomplish the same 50 percent increase shall be permitted. Where epoxy coated bars are used, lap length shall be increased by 50 percent.

Reason: In the TMS 402-11 standard, the allowable stress, Fs of Grade 60 Reinforcement was increased from 24,000 psi to 32,000 psi. Lap splices are calculated as a basis of this allowable stress. Equation 21-1 previously calculated a lap splice of 48 bar diameters for a Grade 50 bar. The 1.5 increase led to a maximum lap splice length of 72 bar diameters. The TMS 402-11 increase in allowable stress had the unintended consequence of increasing the 48 bar diameter length to 64 bar diameters. The 1.5 increase yields a 96 bar diameter lap length. That this was an unintended consequence can be seen by comparing Section 2107.2.1 to 2108.2, the strength design lap splice provision, which maintained the 72 bar diameter limit. There is no rational reason to have a longer lap splice in Allowable Stress Design than in Strength Design. This code change proposal reestablishes parity and harmonizes the code between these two design methods for lap splice lengths.

Cost Impact: Will not increase the cost of construction
This code change will result in a reduction of construction costs.

S247-16 : 2107.2.1-HUSTON13305

S248-16

IBC: 2107.4.

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

Delete without substitution:

~~**2107.4 TMS 402/ACI 530/ASCE 5, Section 8.3.6, maximum bar size.** Add the following to Chapter 8:~~

~~8.3.6 – Maximum bar size. The bar diameter shall not exceed one-eighth of the nominal wall thickness and shall not exceed one-quarter of the least dimension of the cell, course or collar joint in which it is placed.~~

Reason: Background – TMS 402 contains tw o alternatives for the design of conventional masonry systems: allow able stress design (Chapter 8 of the reference standard) and strength design (Chapter 9 of the reference standard). In previous versions of TMS 402 limits on the maximum bar size w ere included for the strength design provisions consistent w ith the requirements of Section 2107.4, but w ere absent for the corresponding allow able stress design provisions; hence the modification language of Section 2107.4. Recently the reference standard has been revised to include maximum bar size limits consistent w ith that of Section 2107.4 that is applied to both the allow able stress and strength design provisions of the reference standard (Section 6.1.2.2) making this modification redundant and unnecessary.

Cost Impact: Will not increase the cost of construction
No technical change. Removes requirements now covered under the reference standard.

S248-16 : 2107.4-THOMPSON12350

S249-16

IBC: , 2109, 2109 (New), 2109.1, 2109.1 (New), 2109.1.1, 2109.2, 2109.2 (New), 2109.2.1, 2109.2.2, 2109.3, 2109.3 (New), 2109.3.1, 2109.3.1.1, 2109.3.1.2, 2109.3.1.2.1, 2109.3.1.2.2, 2109.3.1.2.3, 2109.3.1.2.4, 2109.3.1.3, 2109.3.1.4, 2109.3.2, 2109.3.2.1, 2109.3.2.2, 2109.3.3, 2109.3.3.1, 2109.3.4, 2109.3.4.1, 2109.3.4.2, 2109.3.4.2.1, 2109.3.4.2.2, 2109.3.4.3, 2109.3.4.4, 2109.3.4.5, 2109.3.4.5.1, 2109.3.4.5.2, 2109.3.4.6, 2109.3.4.7, 2109.3.4.7.1, 2109.3.4.7.2, 2109.3.4.8, 2109.3.4.9, 2109.4 (New), 2109.5 (New).

Proponent : Jason Thompson, Masonry Alliance for Codes and Standards (MACS), representing Masonry Alliance for Codes and Standards (jthompson@ncma.org)

2015 International Building Code

Delete without substitution:

~~**SECTION 2109 EMPIRICAL DESIGN OF MASONRY**~~

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

~~**2109.1.1 Limitations.** The use of empirical design of masonry shall be limited as noted in Section 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_{asd} as determined in accordance with Section 1609.3.1 of the *International Building Code* exceeds 110 mph.~~

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

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

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

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

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

~~**2109.3 Adobe construction.** Adobe construction shall comply with this section and shall be subject to the requirements of this code for Type V construction, Appendix A of TMS 402/ACI~~

~~530/ASCE 5, and this section.~~

~~**2109.3.1 Unstabilized adobe.** Unstabilized adobe shall comply with Sections 2109.3.1.1 through 2109.3.1.4.~~

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

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

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

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

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

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

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

~~where, for the purposes of this section only:~~

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

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

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

~~**2109.3.2 Stabilized adobe.** Stabilized adobe shall comply with Section 2109.3.1 for unstabilized adobe in addition to Sections 2109.3.2.1 and 2109.3.2.2.~~

~~**2109.3.2.1 Soil requirements.** Soil used for stabilized adobe units shall be chemically compatible with the stabilizing material.~~

~~**2109.3.2.2 Absorption requirements.** A 4-inch (102 mm) cube, cut from a stabilized adobe unit dried to a constant weight in a ventilated oven at 212°F to 239°F (100°C to 115°C), shall not absorb more than~~ $\cancel{2^1/_2}$ ~~percent moisture by weight when placed upon a constantly water-saturated, porous surface for seven days. A minimum of five specimens shall be tested and each specimen shall be cut from a separate unit.~~

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

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

~~**TABLE 2109.3.3.1**~~
~~**ALLOWABLE SHEAR ON BOLTS IN ADOBE MASONRY**~~

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

~~**2109.3.4 Detailed requirements.** Adobe construction shall comply with Sections 2109.3.4.1 through 2109.3.4.9.~~

~~**2109.3.4.1 Number of stories.** Adobe construction shall be limited to buildings not exceeding one *story*, except that two *story* construction is allowed when designed by a *registered design professional*.~~

~~**2109.3.4.2 Mortar.** Mortar for adobe construction shall comply with Sections 2109.3.4.2.1 and 2109.3.4.2.2.~~

~~**2109.3.4.2.1 General.** Mortar for stabilized adobe units shall comply with this chapter or adobe soil. Adobe soil used as mortar shall comply with material requirements for stabilized adobe. Mortar for unstabilized adobe shall be Portland cement mortar.~~

~~**2109.3.4.2.2 Mortar joints.** Adobe units shall be laid with full head and bed joints and in full running bond.~~

~~**2109.3.4.3 Parapet walls.** Parapet walls constructed of adobe units shall be waterproofed.~~

~~**2109.3.4.4 Wall thickness.** The minimum thickness of *exterior walls* in one-story buildings shall be 10 inches (254 mm). The walls shall be laterally supported at intervals not exceeding 24 feet (7315 mm). The minimum thickness of interior *load bearing walls* shall be 8 inches (203 mm). In no case shall the unsupported height of any wall constructed of adobe units exceed 10 times the thickness of such wall.~~

~~**2109.3.4.5 Foundations.** Foundations for adobe construction shall be in accordance with Sections 2109.3.4.5.1 and 2109.3.4.5.2.~~

~~**2109.3.4.5.1 Foundation support.** Walls and partitions constructed of adobe units shall be supported by foundations or footings that extend not less than 6 inches (152 mm) above adjacent ground surfaces and are constructed of solid masonry (excluding adobe) or concrete. Footings and foundations shall comply with Chapter 18.~~

~~**2109.3.4.5.2 Lower course requirements.** Stabilized adobe units shall be used in adobe walls for the first 4 inches (102 mm) above the finished first-floor elevation.~~

~~**2109.3.4.6 Isolated piers or columns.** Adobe units shall not be used for isolated piers or columns in a load-bearing capacity. Walls less than 24 inches (610 mm) in length shall be considered isolated piers or columns.~~

~~**2109.3.4.7 Tie beams.** *Exterior walls* and interior *load bearing walls* constructed of adobe units shall have a continuous tie beam at the level of the floor or roof bearing and meeting the following requirements.~~

~~**2109.3.4.7.1 Concrete tie beams.** Concrete tie beams shall be a minimum depth of 6 inches (152 mm) and a minimum width of 10 inches (254 mm). Concrete tie beams shall be continuously reinforced with a minimum of two No. 4 reinforcing bars. The specified compressive strength of~~

~~concrete shall be at least 2,500 psi (17.2 MPa).~~

~~**2109.3.4.7.2 Wood tie beams.** Wood tie beams shall be solid or built-up of lumber having a minimum nominal thickness of 1 inch (25 mm), and shall have a minimum depth of 6 inches (152 mm) and a minimum width of 10 inches (254 mm). Joints in wood tie beams shall be spliced a minimum of 6 inches (152 mm). No splices shall be allowed within 12 inches (305 mm) of an opening. Wood used in tie beams shall be *approved* naturally decay-resistant or preservative-treated wood.~~

~~**2109.3.4.8 Exterior finish.** *Exterior walls* constructed of unstabilized adobe units shall have their exterior surface covered with a minimum of two coats of Portland cement plaster having a minimum thickness of $^3/_4$ inch (19.1 mm) and conforming to ASTM C 926. Lathing shall comply with ASTM C 1063. Fasteners shall be spaced at 16 inches (406 mm) on center maximum. Exposed wood surfaces shall be treated with an *approved* wood preservative or other protective coating prior to lath application.~~

~~**2109.3.4.9 Lintels.** Lintels shall be considered structural members and shall be designed in accordance with the applicable provisions of Chapter 16.~~

Add new text as follows:

SECTION 2109 DRY-STACK MASONRY

2109.1 General. The design of dry-stack masonry structures shall comply with the requirements of Chapters 1 through 8 of TMS 402 except as modified by Sections 2109.2 through 2109.5.

2109.2 Limitations. Dry-stack masonry shall be prohibited in Risk Category IV structures.

2109.3 Materials. Concrete masonry units complying with ASTM C90 shall be used.

2109.4 Strength. Dry-stack masonry shall be of adequate strength and proportions to suport all superimposed loads without exceeding the allowable stresses listed in Table 2109.4. Allowable stresses not specified in Table 2109.1.1 shall comply with the requirements of Chapter 8 of TMS 402.

TABLE 2109.4
GROSS CROSS-SECTIONAL AREA ALLOWABLE STRESS FOR DRY-STACK MASONRY

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

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

2109.5 Construction. Construction of dry-stack masonry shall comply with ASTM C946.

Reason: Section 2109 of the IBC currently addresses the design and construction of: empirically designed conventional masonry; dry-stack masonry, and adobe masonry construction. This change effectively removes the provisions for empirical design and adobe construction while retaining the existing dry-stack provisions. Adobe construction, while still used in some niche markets, is almost exclusively limited to single family construction and as such is proposed to be removed from the IBC. (A separate code change proposal addresses incorporating the adobe design and construction requirements into the IRC.)
Codified empirical design provisions for masonry have existed in the US for nearly a century. This cookbook methodology of laying out and proportioning masonry elements is largely based on lessons learned through field performance rather than any analytical or research-based approach to design. As such, some have begun to question the practicality as well as safety of this design methodology. Given these concerns as well as the restrictions placed on empirical design (low wind and seismic) limiting its use geographically combined with the design community gravitating away from this method, the general consensus is that it is time to sunset empirical design.

Currently the reference standard TMS 402 still contains an Appendix A covering empirically designed masonry. The Committee's intent is to remove empirical design, but did not want to do so until the requirements for adobe and dry-stack construction were appropriately resolved in the IBC.

The provisions for dry-stack construction proposed here, while reformatted and cleaned up, are technically consistent with the existing IBC requirements for dry-stack construction. Minor differences include:

1) The term 'dry-stacked' is replaced with 'dry-stack'; as this is consistent with existing industry terminology.

2) The existing IBC language simply requires that 'concrete masonry units' be used for dry-stack construction. An explicit reference to ASTM C90 for loadbearing concrete masonry units is added in this proposal to avoid any ambiguity.

3) The existing IBC provisions requires that the 'allowable stresses' of TMS 402 be used for stresses not specified in Table 2109.4. The reference to 'allowable stresses' is replaced with a direct reference to Chapter 8 of TMS 402 (allowable stress design of masonry).

Cost Impact: Will not increase the cost of construction
While some many argue that not having a simple-to-use empirical method for designing masonry structures would increase the cost of construction, a counter arguement could claim that engineered masonry structures yield more economical designs.

S249-16 : 2109 (NEW)-THOMPSON12566

S250-16

IBC: 2109.1.1.

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_{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.

S250-16 : 2109.1.1-THOMPSON12353

S251-16

IBC: 2109.3.4.2.1.

Proponent : Karl Loescher, Loescher Meachem Architects, representing The Earthbuilders' Guild (bloescher@lmarchitectsinc.com)

2015 International Building Code

Revise as follows:

2109.3.4.2.1 General. Mortar for ~~stabilized~~ adobe units shall ~~comply~~ be in accordance with ~~this chapter~~ Section 2103.9, or be comprised of adobe soil of the same composition and stabilization as the adobe brick units. ~~Adobe~~ Unstabilized adobe soil ~~used as~~ mortar ~~shall comply~~ is permitted in conjunction with ~~material requirements for stabilized adobe. Mortar for~~ unstabilized adobe ~~shall be Portland cement mortar~~ brick units.

Reason: Overview:

Through experience and testing, it has been demonstrated that earthen w all systems perform best structurally and are most durable w hen they are constructed of homogenous materials, both w ithin the brick units themselves, as w ell as betw een the brick units and binding mortar.

Traditionally, adobe brick w all construction used mortars comprised of the same earthen constituents as the bricks themselves. As the slow set of earthen mortars determines the pace of adobe construction, w ith the availability of cement mortars, adobe masons began using lime and portland cement to increase production. This innovation became integrated into model code language for unstabilized adobe bricks. How ever, not only is there no positive evidence to support the requirement, but research has show n that the use of portland cement mortars as prescribed in the code yields w alls that are less cohesive, and more prone to degradation than mortars made of adobe soil, w hether stabilized or unstabilized. The code should be amended to allow mortars of characteristics and components as similar to those of the adobe brick as possible.

Accepted Best Practices:

Although the use of Portland cement mortars is currently prescribed by code for unstabilized adobes, accepted best practices do not encourage their use. For example, ASTM E2392/E2392M "Standard Guide for Design of Earthen Wall Building Systems" advises the follow ing:

> *7.1.5 Mortars should be as nearly as possible the same material as the masonry in terms of strength, stiffness, and vapor permeability. Unstabilized earthen mortars should not be used on the exterior of cement-stabilized or stone masonry, and cement-based mortars should not be used with unstabilized earthen masonry. In all cases, mortar joints should be kept as thin as practicable; the thinner the mortar joints, the stronger the wall.*

The same advice is echoed in the National Park Service's Preservation of Historic Adobe Buildings brief, this time w ith respect to building durability, w ith the implication that the use of mortars incompatible w ith the adobe brick units can cause structural deterioration of the w all:

> *"In repairing loose and deteriorated adobe mortar, care should also be taken to match the original material, color, and texture. Most important, never replace adobe mud mortar with lime mortar or portland cement mortar. It is a common error to assume that mortar hardness or strength is a measure of its suitability in adobe repair or reconstruction. Mortars composed of portland cement or lime do not have the same thermal expansion rate as adobe brick. With the continual thermal expansion and contraction of adobe bricks, portland cement or lime mortars will cause the bricks—the weaker material—to crack, crumble, and eventually disintegrate."*

Earth Masonry: Design & Construction Guidelines further makes the case against the use of Portland Cement mortars:

> *"Traditional clay mortars... illustrate the fundamental rule that the mortar should have strength and movement characteristics that are compatible with the masonry unit... Cementitious mortars are never appropriate for earth masonry, as their dramatically greater stiffness stresses the mortar-brick bonds through differential thermal and moisture movements." (Morton, 54)*

Structural Considerations:

Structural testing on compressed earth blocks confirms this guidance, suggesting "little benefit to be gained from using comparatively high strength mortars with most pressed earth blocks" (Morgan, 7), cement mortars were "ineffective" at increasing the strength of adobe/mortar sandwiches (Islam & Iwashita, 286), and that when used with cement stabilized compressed earth block units "composite mortars such as cement-lime mortar and cement-soil mortars have better tensile bond strength as compared to cement mortar" (Reddy & Gupta, 42).

The greater strength offered by Portland Cement mortars (acknowledged in the code as Type N, S, and M) has no documented benefit when used in conjunction with comparatively weak masonry units such as adobe brick and compressed earth block.

Durability Considerations:

Outside of the laboratory, it should be further understood that adobe bricks have been shown to be most durable when laid with mortars that have similar qualities (Garrison & Ruffner, 40), (Guillaud et al, 24). In addition to having similar strength between mortar and brick unit (which is discussed above), other material characteristics where dissimilarity has proven negative consequences on wall durability include permeability/porosity, density, resistance to erosion, and coefficient of expansion. This similarity of property would be guaranteed if the code were revised as proposed.

Conclusion:

Modifying the model language to allow the use of adobe mortars comprised of the same materials as the bricks themselves will have no negative impact on life safety, but will allow builders and designers to increase the structural performance and durability of adobe buildings.

Bibliography: *Preservation Brief 5: Preservation of Historic Adobe Buildings.* U.S. Department of the Interior National Park Service Heritage Preservation Services, 1978. http://www.nps.gov/tps/how-to-preserve/briefs/5-adobe-buildings.htm
E2392/E2392M Standard Guide for Design of Earthen Wall Building Systems. ASTM International, 2010: p. 6.

Garrison, James, and Ruffner, Elizabeth. *Practical & Technical Aspects of Adobe Conservation*. Heritage Foundation of Arizona. Tuscon, 1983. p. 40.

Guillaud, H., Joffrey, T, and Odul, P. *Compressed Earth Blocks: Manual of Design and Construction.* Deutsches Zentrum für Entwicklungstechnologien. Eschborn, 1995.

Islam, Mohammad Shariful, and Iwashita, Kazuyoshi. *"Seismic Strengthening of Adobe Structures Using Low-Cost Materials"*. Terra 2008 : Proceedings of the 10th International Conference on the Study and Conservation of Earthen Architectural Heritage, Bamako, Mali, February 1–5, 2008. Getty Conservation Trust, Los Angeles, 2011: p. 286.

Morgan, P. *"Bond Characteristics of Earth Block Masonry."* Journal of Materials in Civil Engineering 11.3 :p. 7

Morton, Tom. *Earth Masonry Design and Construction Guidelines.* IHS BRE Press, Bracknell, 2008.

Venkatarama Reddy, B. V., and Ajay Gupta. *"Tensile Bond Strength of Soil-Cement Block Masonry Couplets Using Cement-Soil Mortars."* Journal of Materials in Civil Engineering 18.1 (2006): 42.

Cost Impact: Will not increase the cost of construction
This code change will not increase the cost of construction. Designers and contractors will still have the option of selecting faster-setting, portland cement mortars.

S251-16 : 2109.3.4.2.1-LOESCHER10959

S252-16

IBC: 2203.1, 2203.2, 2210.2, 2211.1, 2211.1.1 (New), 2211.1.1.1 (New), 2211.1.1.2 (New), 2211.2, 2211.2 (New), 2211.3, 2211.3.1, 2211.3.2, 2211.3.3, 2211.3.4, 2211.4, 2211.5, 2211.6, 2211.7.

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 ~~S110~~ 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:

2211.1.1 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.

2211.1.1.1 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.

2211.1.1.2 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:

~~2211.7~~ 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~~ comply with ~~AISI S214,~~ the additional provisions of Sections ~~2211.3.1~~ 2211.1.3.1 through ~~2211.3.4 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 ~~B2.3~~ I1 of AISI ~~S214~~ 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 ~~B6(a) or B 6(c)~~ I1.6 of AISI ~~S214~~ S202 where these methods are utilized to provide restraint/bracing.

~~2211.3.3~~ 2211.1.3.2 Trusses spanning 60 feet or greater. *No change to text.*

~~2211.3.4~~ 2211.1.3.3 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~~ fabricated 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, 2211.1, Table 2603.12.1,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 S213—07/S1-09 (2012) *North American Standard for Cold-Formed Steel Framing-Lateral Design*, 2007, with Supplement 1, dated 2009, (Reaffirmed 2012), 2211.6~~

~~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 dow nload at: w w w .aisistandards.org.

AISI S240, *North American Standard for Cold-Formed Steel Structural Framing*, addresses requirements for construction w ith cold-formed steel structural framing that are common to prescriptive and engineered light frame construction. This comprehensive standard w as formed by merging the follow ing 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 w ith Supplement No. 1-09, and the seismic portions of AISI S213, *North American Standard for Cold-Formed Steel Framing – Lateral Design*, 2007 w ith Supplement No. 1-09. The layout and many of the seismic design requirements are draw n from ANSI/AISC 341-10, *Seismic Provisions for Structural Steel Buildings*, w hich 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 w ith 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 betw een various parties in a construction project w here 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 draw ings.

Modifications specific to Chapter 22 include the follow ing:

- 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.

S252-16 : 2203.1-MANLEY11247

S253-16

IBC: 2208.2.

Proponent : Jennifer Goupil, representing SELF (jgoupil@asce.org)

2015 International Building Code

Delete without substitution:

~~**2208.2 Seismic requirements for steel cable.** The design strength of steel cables shall be determined by the provisions of ASCE 19 except as modified by these provisions.~~

1. ~~A load factor of 1.1 shall be applied to the prestress force included in T_3 and T_4 as defined in Section 3.12.~~
2. ~~In Section 3.2.1, Item (c) shall be replaced with "1.5 T_3" and Item (d) shall be replaced with "1.5 T_4."~~

Reason: This proposal is a coordination proposal to bring the 2018 IBC up to date with the provisions of the 2016 edition of *ASCE 19 Structural Applications for Steel Cables in Buildings*. The proposal removes the exceptions to ASCE 19 for seismic requirements for steel cables. The exceptions are no longer applicable because the load combinations in ASCE 19 have been harmonized with the load combinations in *ASCE 7 Minimum Design Loads for Buildings and Other Structures* as of the 2010 edition of that standard. The load combinations and safety factors in ASCE 19 have been updated for the past two cycles of the standard, yet this outdated exception remained in the code erroneously.

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 standard *ASCE 19 Structural Applications for Steel Cables in Buildings*. ASCE 19 will be updated from the 2010 edition tot he 2016 edition as an Administrative Update to the 2018 I-Codes.
As of the submission date of this code change proposal, the ASCE 19 Standards Committee has completed the committee balloting on the technical changes and the standard is expected to be open for public comment in February of 2016. The document designated *ASCE 19-16StructuralApplications for Steel Cables in Buildings* is expected to be completed, published, and available for purchase prior to the ICC Public Comment Hearings in October 2016. Any person interested in obtaining a public comment copy of ASCE 19-16 may do so by contacting James Neckel at ASCE (jneckel "at" asce.org).

S253-16 : 2208.2-GOUPIL12416

S254-16

IBC: 2211.6 (New).

Proponent : Randy Shackelford, representing Simpson Strong-Tie (rshackelford@strongtie.com)

2015 International Building Code

Add new text as follows:

2211.6 Joist connectors. Joist connectors for cold-formed steel framing shall be tested and load rated in accordance with AISI S914

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
AISI S914-15 Test Standard for Joist Connectors Attached to Cold-Formed Steel Structural Framing 2211.6

Reason: The reason for this proposal is to reinstate some requirements for testing of joist hangers and connectors w hen used w ith cold-forrmed steel framing.
Prior to the 2015 IBC, the requirements for testing and load rating of joist hangers w ere in Chapter 17, so they applied to all materials. For the 2015 IBC, the American Wood Council moved those requirements to Chapter 23, Wood. That is fine for w ood, but now there are no requiremens for testing of joist connector for cold-formed steel framing.

AISI has recently developed AISI S914-15, *Test Standard for Joist Connectors Attached to Cold-Formed Steel Structural Framing*. This document specifies the test methods to be used to test joist connectors, and also the sections of AISI S100 to be used to evaluate the test data. This is an ANSI consensus document suitable for adoption by the code.

Adoption of this standard w ill give connector manufacturers a consistent standard to be used for rating their products.

Cost Impact: Will not increase the cost of construction
Should not increase the cost of construction. Will simply give manufacturers a needed, consistent and level basis for load rating of manufacturered joist connectors.

Analysis: A review of the standard(s) proposed for inclusion in the code, AISI S914, w ith regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) w ill be posted on the ICC w ebsite on or before April 1, 2016.

S254-16 : 2211.6 (NEW)-SHACKELFORD13445

S255-16

IBC: 2209.2 (New).

Proponent : Victor Azzi, representing Rack Manufacturers Institute (victorazzi@comcast.net)

2015 International Building Code

Add new text as follows:

2209.2 Cantilevered steel storage racks. The design, testing, and utilization of cantilevered storage racks made of cold-formed or hot-rolled steel structural members shall be in accordance with RMI/ANSI MH 16,3. Where required by ASCE 7, the seismic design of cantilevered steel storage racks shall be in accordance with Section 15.5.3.3 of ASCE 7.

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

Add new standard(s) as follows:

RMI/ANSI MH 16.3-2016. Specification for the Design, Testing, and Utilization of Industrial Steel Cantilevered Storage Racks.

Reason: This proposal is intended to coordinate the definition of cantilevered storage racks with the 2015 IBC definition of steel storage racks as well as the ASCE 7 Standard, and to include the new Cantilevered Storage Rack standard, RMI/ANSI MH 16.3, in both the ASCE 7 and the IBC by reference. Having a separate standard for cantilevered storage racks will help clarify, for the designers and users of industrial steel storage racks, the characteristics and essential differences and requirements in the design, construction, use, and behavior of cantilevered storage racks as distinguished from the more conventional systems commonly known as "pallet rack" or "selective rack."

Cost Impact: Will not increase the cost of construction

There will be no cost impact. There will be no increase in the cost of construction and installation of this manufactured product. Cantilevered Storage Racks have been manufactured and successfully utilized in large-scale warehouses and distribution centers for many years. Having a new Standard developed and introduced to the industry at this time is intended to identify, clarify, and memorialize the essential differences in the behaviors of such structural systems and the important considerations and requirements in their detailed design, installation, and utilization. This product will continue to be designed and produced primarilly by the same manufacturers who have been producing cantilevered storage racks, along with the more common pallet racks, for many years.

Analysis: A review of the standard(s) proposed for inclusion in the code, RMI/ANSI MH 16.3, 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.

S255-16 : 22XX (NEW)-AZZI13456

S256-16

IBC: 2303.1.1.3 (New).

Proponent : Daniel Nichols, New York State Division of Building Standards and Codes, representing New York State Division of Building Standards and Codes (dnichols@dos.state.ny.us)

2015 International Building Code

Add new text as follows:

2303.1.1.3 Lumber mills In lieu of a grade mark or issued a certificate of inspection by a lumber grading or inspection agency, sawn lumber shall be permitted under the following conditions when authorized by the building official.

1. The lumber mill shall sell or provide the lumber directly to the building owner or the owner's contract builder.
2. The producing mill shall certify in writing to the building owner or owner's contract builder that the quality of the safe working stresses of the lumber is equal or exceeds the No. 2 grade of the species in accordacne with DOC PS 20. The documentation shall be filed with the building permit application.
3. The use of such lumber shall be limited to buildings that do not exceed 10,000 square feet of building area or 35 feet of building height.

Reason: The purpose of this code change is to address the use of lumber produced by smaller lumber mills for local projects. Typically, smaller, local lumber mills rely on sales of lumber to the area right around the lumber mill for the construction of homes and small commerical buildings. How ever, many of these mills have been in business for several decades and do not have their saw n lumber stamped in accordance w ith DOC PS 20.
The proposal has been used in the State of New York for 12 years w ithout any documented issues. It has been successful in allow ing local lumber mills to provide products to local construction; esspecially in rural areas. The contents of the proposal require direct sale of the products to the end-user, w hich is important in filing the documentation w ith the building official for the exact product being purchased and used in construction. The limiation of 10,000 square feet and 35 feet w ere found to be acceptable limitations in construction that w ould desire the use of ungraded lumber from local lumber mills.

Cost Impact: Will not increase the cost of construction
The allow ance of ungraded lumber in certain circumstances w ill potentially reduce the cost of construction in areas that have access to local saw mills; due to the availability of more choices in product.

S256-16 : 2303.1.1.3 (NEW)-NICHOLS5412

S257-16

IBC: 2303.1.10.1 (New).

Proponent : Paul Coats, PE CBO, American Wood Council, representing American Wood Council (pcoats@awc.org)

2015 International Building Code

Add new text as follows:

2303.1.10.1 Treated structural composite lumber. Design values for treated structural composite lumber shall account for the effects of the anticipated temperature and humidity to which the treated structural composite lumber will be subjected, the type of treatment, and, if applicable, re-drying procedures.

Reason: Clarify that design values for treated product must take into consideration the effects of the anticipated temperature and humidity to w hich the treated SCL w ill be subjected, the type of treatment, and re-drying procedures. The purpose of this change is to reduce potential for post-manufacture treatment of SCL w ithout consideration of the effect on the design properties.

Cost Impact: Will not increase the cost of construction
Although the current code does not explcitly require evaluation of design properties for treated SCL, it is understood that treatment can affect design properties and therefore treated SCL must be evaluated similarly to lumber and w ood structural panels. SCL manufacturers are doing this already.

S257-16 : 2303.1.10.1 (NEW)-COATS11411

S258-16

IBC: 2303.1.7, [BS] 1404.3, [BS] 1404.3.1, [BS] 1404.3.2.

Proponent : David Tyree, representing American Wood Council (dtyree@awc.org)

2015 International Building Code

Revise as follows:

[BS] 1404.3 Wood. *Exterior walls* of wood construction shall be designed and constructed in accordance with Chapter 23.

[BS] 1404.3.1 Basic hardboard. Basic hardboard shall conform to the requirements of ~~AHA~~ ANSI A135.4.

[BS] 1404.3.2 Hardboard siding. Hardboard siding shall conform to the requirements of ~~AHA~~ ANSI A135.6 and, where used structurally, shall be so identified by the *label* of an *approved* agency.

2303.1.7 Hardboard. Hardboard siding shall conform to the requirements of ANSI A135.6 and, where used structurally shall be identified by the label of an *approved agency*. ~~conforming to CPA/ANSI A135.6.~~ Hardboard underlayment shall meet the strength requirements of $^{7}/_{32}$-inch (5.6 mm) or $^{1}/_{4}$-inch (6.4 mm) service class hardboard planed or sanded on one side to a uniform thickness of not less than 0.200 inch (5.1 mm). Prefinished hardboard paneling shall meet the requirements of ~~CPA/~~ANSI A135.5. Other basic hardboard products shall meet the requirements of ~~CPA/~~ANSI A135.4. Hardboard products shall be installed in accordance with manufacturer's recommendations.

Reason: This proposal references various CPA stadnards in a consistent manner and also clarifies that hardboard siding must conform to the requirements of A135.6 in 2303.1.7 in a consistent manner with reference to hardboard siding in 1404.3.2.

Cost Impact: Will not increase the cost of construction
This proposal clarifies the code and does not place any additional costs on the user.

S258-16 : 2303.1.7-TYREE11261

S259-16

IBC: 2303.1.9.3 (New), 2303.2.5, 2306.1.3.

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

2015 International Building Code

Add new text as follows:

2303.1.9.3 Strength Adjustments. Design values for *preservative-treated wood* in accordance with Section 2303.1.9 need no adjustment for treatment. Other adjustments are applicable except that the impact load duration shall not apply.

Revise as follows:

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 duration shall not apply.

Delete without substitution:

~~**2306.1.3 Treated wood stress adjustments.** The allowable unit stresses for *preservative-treated wood* need no adjustment for treatment, but are subject to other adjustments.~~

~~The allowable unit stresses for *fire-retardant-treated wood*, including fastener values, shall be developed from an *approved* method of investigation that considers the effects of anticipated temperature and humidity to which the *fire-retardant-treated wood* will be subjected, the type of treatment and the redrying process. Other adjustments are applicable except that the impact load duration shall not apply.~~

Reason: Section 2306.1.3 is redundant w ith Section 2303.2.5 and can be deleted. Location of design value information in 2303.2.5 as opposed to 2306 on Allow able Stress Design is preferable as information in 2305 is generally applicable and addresses use for both ASD and LRFD. Portions of 2306.1.3 not addressed by 2303.2.5 are moved to 2303.2.5 and a new section (2303.1.9.3) on strength adjustments for preservative treated w ood. Preservative treated w ood in accordance w ith 2303.1.9 complies w ith applicable AWPA standards and adjustment for preservative treatment is not required. In some cases, preservative treated w ood must be incised to facilitate treatment and for such cases the incising factor is applicable along w ith "other adjustments" for end use such as temperature , moisture, and load duration.

Cost Impact: Will not increase the cost of construction
This proposal does not increase the cost of construction as it merely correlates and clarifies various requirements from standards.

S259-16 : 2303.1.9.3 (NEW)-RICHARDSON12693

S260-16

IBC: 2303.2.

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*~~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~~1/ ~~2~~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:

ASTM E2768 (2011) Standard Test Method for Extended Duration Surface Burning Characteristics of Building Materials (30 min Tunnel Test).

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:
1.3.1. Method A "Test Method for Accelerated Weathering of Fire-Retardant-Treated Wood for Fire Testing" in ASTM D 2898, for fire-retardant treated
wood, wood-plastic composite and plastic lumber materials.
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.

S260-16 : 2303.2-HIRSCHLER12079

S261-16

Part I:
IBC: 202, 2303.2, 2303.2.1, 2303.2.2, 2303.2.3, 2303.3 (New).
Part II:
IRC: R802.1.5, R802.1.6 (New).

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC-BUILDING & ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

Proponent : Joseph Holland, representing Hoover Treated Wood Products (jholland@frtw.com); David Bueche, representing Hoover Treated Wood Products (dbueche@frtw.com)

Part I

2015 International Building Code

Revise as follows:

SECTION 202 DEFINITIONS

[BS] TREATED WOOD. Wood products ~~that are conditioned~~ impregnated with chemicals to enhance fire-retardant or preservative properties.

~~**Fire-retardant-treated wood.** Wood products that, when impregnated with chemicals by a pressure process or other means during manufacture, exhibit reduced surface-burning characteristics and resist propagation of fire.~~

~~**Preservative-treated wood.** Wood products that, conditioned with chemicals by a pressure process or other means, exhibit reduced susceptibility to damage by fungi, insects or marine borers.~~

Fire-retardant-treated wood. Wood products that, when impregnated with chemicals exhibit reduced surface-burning characteristics and resist propagation of fire.

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,~~ and when tested in accordance with ASTM E 84 or UL 723, shall have 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~~ impregnated 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.

Add new text as follows:

2303.3 Surface coatings. Surface coatings shall be in accordance with Chapter 5 of NFPA 703.

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
NFPA 703-15 Standard for Fire-Retardant Treated Wood and Fire-Retardant Coating for Building Materials. 2015 edition

Part II

2015 International Residential Code

Revise as follows:

R802.1.5 Fire-retardant-treated wood. Fire-retardant-treated wood (FRTW) is any wood product ~~that, when~~ impregnated with chemicals ~~by a pressure process or other means during manufacture,~~ . Fire-retardant-treated wood shall ~~have, when~~ be tested in accordance with ASTM E 84 or UL 723~~,~~ . Fire-retardant-treated wood shall have a listed flame spread index of 25 or less ~~and shows~~ . Fire-retardant-treated wood shall exhibit no evidence of significant progressive combustion where the test is continued for an additional 20-minute period. In addition, the flame front shall not progress more than 10.5 feet (3200 mm) beyond the center line of the burners at any time during the test.

Add new text as follows:

R802.1.6 Surface Coating Surface coating shall be inaccordance with Chaper 5 NFPA 703, Standard for Fire Retardant-Treated Wood and Fire Retardant Coating for Building Materials.

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
NFPA 703-15 Standard for Fire-Retardant Treated Wood and Fire-Retardant Coating for Building Materials. 2015 edition
Reason:

Part I: Discussion during past code development cycles have shown there is confusion as to what process the phrase "other means during manufacture" is referring. Testimony often leaves out the "during manufacture" part of the phrase leading one to assume coating applied after manufacture is permitted. Attempts to clarify have only been partially successful.
Dictionary Definition: impregnate

im-preg-nate (im-pregnat)v.tr. im-preg-nat-ed, im-preg-nat-ing, im-preg-nates. **1.** To make pregnant; inseminate. **2.** To fertilize (an ovum, for example). **3. To fill throughout; saturate**: *a cotton wad that was impregnated with ether.* **4.** To permeate or imbue: *impregnate a speech with optimism.* Excerpted from *American Heritage Talking Dictionary*. Copyright © 1997 The Learning Company, Inc. All Rights Reserved.

Impregnate describes the process mandated by the code with the phrase "other means during manufacture." The current Section 2303.2.2 states the treatment is an integral part of the manufacturing process. A presentation by Benjamin Floyd and Alan Ross, Kop-Cote, Inc., at the 2010 Forest Products Society conference in Orlando, FL explains what integral means for wood treatments. It is: "The term "integral treatments" refers to combining the active ingredients with the wood furnish (i.e., chips, flakes, strands, etc.) before processing." The dictionary definition of "impregnate" #3 shown above eliminates any confusion as to what the code expects for FRTW.

A review of the available literature shows all the testing done for acceptance of FRTW into the codes was performed on wood impregnated with chemicals. The testing ranged from small scale (ASTM E160), to large scale

(ASTM E84 and E119) to full scale (White House, (UL 1256 part 2).

The revision clarifies what is expected and eliminates possible confusion pertaining to the "other means during manufacture" statement.

The addition of the new section titled Surface Coating let the user know that surface coatings are not covered by section 2303.2 and provides the information as to what standard covers surface coatings: NFPA 703, Standard for Fire Retardant–Treated Wood and Fire-Retardant Coatings for Building Materials.

Part II: Dictionary Definition: impregnate
im-preg-nate (im-pregnat)v. tr. im-preg-nat-ed, im-preg-nat-ing, im-preg-nates. **1.** To make pregnant; inseminate. **2.** To fertilize (an ovum, for example). **3. To fill throughout; saturate**: *a cotton wad that was impregnated with ether.* **4.** To permeate or imbue: *impregnate a speech with optimism.*[i]

[i]Excerpted from *American Heritage Talking Dictionary*. Copyright © 1997 The Learning Company, Inc. All Rights Reserved.

Reason: Discussion during past code development cycles have shown there is confusion as to what process the phrase "other means during manufacture" is referring. Testimony often leaves out the "during manufacture" part of the phrase leading one to assume coating applied after manufacture is permitted. Attempts to clarify have only been partially successful.

Impregnate describes the process mandated by the code with the phrase "other means during manufacture." The current Section 2303.2.2 states the treatment is an integral part of the manufacturing process. A presentation by Benjamin Floyd and Alan Ross, Kop-Cote, Inc., at the 2010 Forest Products Society conference in Orlando, FL explains what integral means for wood treatments. It is: "The term "integral treatments" refers to combining the active ingredients with the wood furnish (i.e., chips, flakes, strands, etc.) before processing." The dictionary definition of "impregnate" #3 shown above eliminates any confusion as to what the code expects for FRTW.

A review of the available literature shows all the testing done for acceptance of FRTW into the codes was performed on wood impregnated with chemicals. The testing ranged from small scale (ASTM E160), to large scale (ASTM E84 and E119) to full scale (White House, (UL 1256 part 2).

The revision clarifies what is expected and eliminates possible confusion pertaining to the "other means during manufacture" statement.

The addition of the new section titled Surface Coating let the user know that surface coatings are not covered by section 2303.2 and provides the information as to what standard covers surface coatings: NFPA 703, Standard for Fire Retardant–Treated Wood and Fire-Retardant Coatings for Building Materials.

Cost Impact:

Part I: Will not increase the cost of construction
The change eliminates confusion and is not connected with cost.

Part II: Will not increase the cost of construction
The changes are to clarify the intent of the code.

Analysis: A review of the standard(s) proposed for inclusion in the code, NFPA 703, 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.

S261-16 : 2303.2-HOLLAND4430

S262-16

IBC: 2303.2.2.

Proponent : Joseph Holland (jholland@frtw.com)

2015 International Building Code

Revise as follows:

2303.2.2 Other means during manufacture. For wood products ~~produced~~ impregnated with chemicals 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. The use of paints, coating, stains or other surface treatment shall not be permitted.

Reason: This section is subject to misinterpretation. The phrase "other means during manufacture" is often quoted as "other means" leaving it open to nonconforming material such as paints, stains and other surface treatments. These surface treatments are not permanent. They are subject to abrasion, degradation from exposure to rain during installation, and flaking or peeling due to the difference in the expansion coefficient of the two materials. When used as roof sheathing the material can be subjected to temperature swings of 100 degrees F or more and during winter months exposure to substantial moisture can be expected.
All of the testing (full scale, large scale and small scale) done on fire-retardant-treated wood in order to be recognized in the code was done on pressure impregnated lumber and plywood.

Cost Impact: Will not increase the cost of construction
Material now recognized is pressure impregnated or the furnish (chips, strands, and flakes) is treated during the manufacturing process. There is no change in those requirements.

S262-16 : 2303.2.2-HOLLAND11588

S263-16

Part I:
IBC: 2303.2.3.
Part II:
IRC: R802.1.5.3.

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC-BUILDING & ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

Proponent : Tim Earl, representing GBH International (tearl@gbhinternational.com)

Part I

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~~
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.

Part II

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~~
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 w ritten, this section requires additional testing for fire-retardant treated w ood materials that have been manufactured by a process different than pressure treatment. All fire retardant treated w ood products must meet the requirements of section 2303.2 (w hich contains the fire test requirements). Moreover, all fire retardant tread w ood products, irrespective of how they are manufactured, must meet the requirements (also contained in section 2303.2) that they must be impregnated w ith chemicals. A w ood material can only be fire retardant treated w ood if it is impregnated w ith chemicals and that w ill differentiate it from fire retardant coated materials. Testing requirements should be a function of performance and not of the w ay a product is made. There is no reason that some fire retardant treated w ood 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 w ill reduce the cost for manufacturers of some materials by eliminating overly burdensome testing requirements based solely on how they are manufactured.

S263-16 : 2303.2.3-EARL12134

S264-16

IBC: 2303.2.3.

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 w ood materials that have been manufactured by a process different than pressure treatment. Note that all fire retardant treated w ood products must meet the requirements of section 2303.2 (w hich contains the fire test requirements). Moreover, all fire retardant tread w ood products, irrespective of how they are manufactured, must meet the requirements (also contained in section 2303.2) that they must be impregnated w ith chemicals. A w ood material can only be fire retardant treated w ood if it is impregnated w ith chemicals and that w ill differentiate it from fire retardant coated materials.
Testing requirements should be a function of performance and not of the w ay a product is made. There is no reason that some fire retardant treated w ood 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 w ood product be tested for fire safety, then 2303.2.3 can be rew ritten as follow s, in w hich case also all products are treated the same w ay, w ithout differences as a function of how they are manufactured:

"2303.2.3 Testing. All sides of the fire retardant treated w ood product shall be tested in accordance w ith 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 w ith this proposal is also being made to IRC 802.1.5.

Cost Impact: Will not increase the cost of construction
This proposal w ill low er the excessive burden of fire testing for some materials as a function of how they are manufactured and not of their performance.

S264-16 : 2303.2.3-HIRSCHLER11637

S265-16

IBC: 2303.2.4.

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~~
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 *approvedagency* 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.
6. Method of drying after treatment.
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.

S265-16 : 2303.2.4-HOLLAND11762

S266-16

IBC: 2303.4.1.1.

Proponent : Stephen Kerr, representing Self (skerr@jwa-se.com)

2015 International Building Code

Revise as follows:

2303.4.1.1 Truss design drawings. The written, graphic and pictorial depiction of each individual truss shall be provided to the *building official* for approval prior to installation. Truss design drawings shall also be provided with the shipment of trusses delivered to the job site. Truss design drawings shall include, at a minimum, the information specified below:

1. Slope or depth, span and spacing;
2. Location of all joints and support locations;
3. Number of plies if greater than one;
4. Required bearing widths;
5. Design loads as applicable, including;
    - 5.1. Top chord live load;
    - 5.2. Top chord dead load;
    - 5.3. Bottom chord live load;
    - 5.4. Bottom chord dead load;
    - 5.5. Additional loads and locations; and
    - 5.6. Environmental design criteria and loads (wind, rain, snow, seismic, etc.).
6. Other lateral loads, including drag strut loads;
7. Adjustments to wood member and metal connector plate design value for conditions of use;
8. Maximum reaction force and direction, including maximum uplift reaction forces where applicable;
9. ~~Metal connector plate~~ Joint connection type~~,~~ and description such as size and thickness or gage, and the dimensioned location of each ~~metal~~ joint connector ~~plate~~ except where symmetrically located relative to the joint interface;
10. Size, species and grade for each wood member;
11. Truss-to-truss connections and truss field assembly requirements;
12. Calculated span-to-deflection ratio and maximum vertical and horizontal deflection for live and total load as applicable;
13. Maximum axial tension and compression forces in the truss members;
14. Required permanent individual truss member restraint location and the method and details of restraint/bracing to be used in accordance with Section 2303.4.1.2.

Reason: Under section 2303.4.1 wood trusses are permitted to be joined by nails, glue, bolts timber connectors, metal connector plates or other approved devices. As currently written IBC 2304.1.1 item 9 is limited to metal-connector plates, and would not require other jointing methods to specify the joint details. While the vast majority of wood trusses may be metal-plate type other options are permitted and the joint connections should be specified as part of the truss design drawings. For reference, the proposed language is taken from IRC R202.10.1 item 7. This will not only clarify the joint detail requirements for non metal-connector plate trusses but also harmonize the IBC and IRC codes.

Cost Impact: Will not increase the cost of construction
The proposal it to clarify the requirements of the truss design drawings. No technical requirements are proposed for the design of trusses.

S266-16 : 2303.4.1.1-

S267-16

IBC: 2303.4.1.2.

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_{ult}, 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_g 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.

S267-16 : 2303.4.1.2-HUSTON13341

S268-16

IBC: 2303.4.6.

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 w ith 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 Restrain/Bracing 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 to 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.

S268-16 : 2303.4.6-HUSTON13346

S269-16

IBC: 2303.6, 2303.6.1 (New).

Proponent : Scott Douglas, Douglas Engineering (sdouglasscott@gmail.com)

2015 International Building Code

Revise as follows:

2303.6 Nails and staples. Nails and staples shall conform to requirements of ASTM F 1667 as modified in Section 2303.6.1. Nails used for framing and sheathing connections shall have minimum average bending yield strengths as follows: 80 kips per square inch (ksi) (551 MPa) for shank diameters larger than 0.177 inch (4.50 mm) but not larger than 0.254 inch (6.45 mm), 90 ksi (620 MPa) for shank diameters larger than 0.142 inch (3.61 mm) but not larger than 0.177 inch (4.50 mm) and 100 ksi (689 MPa) for shank diameters of at least 0.099 inch (2.51 mm) but not larger than 0.142 inch (3.61 mm).

Add new text as follows:

2303.6.1 Package marking of nails and staples Modify ASTM F 1667, Section 12.2 to read as follows:

12.2 Individual packages and shipping containers shall be *specified and* be marked with the part identifying number, including the type, length, diameter (or gage, as applicable) of the fastener, the name of the manufacturer or distributer, and the quality or net weight.

Reason: Section 2303.6
At present a significant percentage of nail packaging specifies only the pennyweight (i.e. 16d) , the type (i.e box) , and the length of the nail, it does not specify the diameter. For example, 16d nails specified on *construction drawings* could be 16d common (3-1/2" x 0.162"), 16d sinkers (3-1/4" x 0.148"), or 16d box (3-1/2 x 0.135"). Numerous contractors believe nails of the same pennyweight are interchangable, frequently resulting in a smaller diameter nail being installed than what was specified. The 2015 IBC and NDS has addressed this problem by identifing nails by length and diameter as well as pennyweight. This has alliveated significant confusion in design, with *construction drawings* typically now specifing nail length and diameter in conjunction with pennyweight and often only nail length and diameter. However due to package identification without nail diameter information there is still confusion and misinterpretation by the contractor regarding the installation of the correct nail size.

Currently ASTM F 1667 Section 12.2 leaves it optional for a purchaser to require ASTM F 1667 nail identification requirements on packaging. Acceptance of this proposal would require nail packaging to include diameter and length. This will make it very clear the difference between types of nails of the same pennyweight. The format used in this proposal for modifing ASTM F1667 Section 12.2 was based on the format used in modifing ACI 318 in the 2015 IBC, such as Section 1905.1.4.

Bibliography: ASTM F 1667 Current Wording (for reference)
12. Packaging and Package Marking

12.1 Unless otherwise specified, fasteners shall be in substantial commercial containers of the type, size, and kind commonly used for the purpose, so constructed as to preserve the contents in good condition and to ensure acceptance and safe delivery by common or other carriers to the point of delivery. In addition, the containers shall be so made that the contents can be removed partially without destroying the container's ability to serve as a receptacle for the remainder of the contents.
12.2 When specified, individual packages and shipping containers shall be marked with the part-identifying number and type, length, diameter (or gage, as applicable) of the fastener, the name of the manufacturer or distributor, and the quantity or net weight.

Cost Impact: Will not increase the cost of construction
Section 2303.6 - There would be a minimal initial cost impact to add nail length and diameter to existing packaging for those manufacturers who have not already added this information.

S269-16 : 2303.6-DOUGLAS13002

S270-16

IBC: 2303.6.

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

2015 International Building Code

Revise as follows:

2303.6 Nails and staples. Nails and staples shall conform to requirements of ASTM F 1667, including Supplement 1. Nails used for framing and sheathing connections shall have minimum average bending yield strengths as follows: 80 kips per square inch (ksi) (551 MPa) for shank diameters larger than 0.177 inch (4.50 mm) but not larger than 0.254 inch (6.45 mm), 90 ksi (620 MPa) for shank diameters larger than 0.142 inch (3.61 mm) but not larger than 0.177 inch (4.50 mm) and 100 ksi (689 MPa) for shank diameters of at least 0.099 inch (2.51 mm) but not larger than 0.142 inch (3.61 mm). Staples used for framing and sheathing connections shall have minimum average bending moment as follows: 3.6 in.-lbs (0.41 N-m) for No. 16 gage staples, 4.0 in.-lbs (0.45 N-m) for No. 15 gage staples, and 4.3 in.-lbs (0.49 N-m) for No. 14 gage staples. The test procedure for staples shall be approved by the building official.

Reason: The referenced ASTM F1667 contains requirements for nail and staple sizes, however it only addresses bending yield strength requirements for nails in the supplementary requirements. ASTM F1667 section S1 is a set of supplementary requirements, not enforceable as a mandatory unless specifically referenced. For structural use nails the American Wood Council's National Design Specification (AWC NDS), the referenced standard for wood construction, does not contain any mandatory language enforcing nail strength requirements. As it currently stands AWC NDS requirements for fastener yield strengths are contained Table I1 which is non-mandatory and as such is not enforceable. This is why section 2303.6 contains the requirements for bending yield strength values for nails, however 2303.6 does not contain any testing methods for determining the average bending yield strength. To determine the yield strength ASTM F1667 Supplement S1 section S1.3 requires the procedure of ASTM F 1575 as the test method for determining the yield strength. If the supplement is not enforced then the test requirements of ASTM F 1575 are not enforced then there are no IBC requirements for testing nail strength.
In the past for structural applications when staples were used, one would rely on the Alternative Materials provisions of IBC section 104.11 and reference the International Staple, Nail and Tool (ISANTA) ICC Evaluation Services Report ESR-1539 for Power-Driven Staples and Nails. With the past several code cycles, staples have become fully integrated into the building code, and are recognized directly in the IBC as an option to nails for structural applications. Consequentially, for staples the Alternative Material procedure is no longer needed and it is possible that a staple manufacturer could produce code staples which are outside of the provisions of ESR-1539 but still acceptable by the IBC requirements.

Section 2303.6 contains strength requirements for nails only, the IBC lacks any similar strength requirements for staples. In addition, the referenced standards for wood fasteners - both AWC NDS and ASTM F 1667 Supplement S1 do not include staples. This proposal will add language for bending moment requirements for staples. The average bending moment values for staples are taken from the values currently found in ISANTA ICC ESR report 1539. Unfortunately since AWC NDS and ASTM do not contain any testing methods for evaluating staples, the testing methodology is left to the discretion of the building official. The only known source of staple performance testing is found in ICC Evaluation Services AC201. AC201 uses ASTM F1575 nail testing as the basis for the testing procedure and modifies testing as needed for the unique conditions of staples. Until the ASTM standards are updated to include test methods for staples the testing methods of AC201, as approved by the building official, is the logical approach to include staple test methods.

Cost Impact: Will increase the cost of construction
For nails there should be not be any cost implication, the proposal is only adding the test methodology for achieving the already code prescribed strength requirement.
For staples manufactured in conformance to the ISANTA ESR-1539 there will be no cost impact. If staples are manufactured to a lesser standard then there may be a slight cost increase. However it should be pointed out that

for any fasteners not meeting ESR-1539 would not be meeting the intention of the IBC structural provisions.

S271-16

IBC: 2304.8.

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

2015 International Building Code

Revise as follows:

TABLE 2304.8 (5)
ALLOWABLE LOAD (PSF) FOR WOOD STRUCTURAL PANEL ROOF SHEATHING CONTINUOUS OVER TWO OR MORE SPANS AND STRENGTH AXIS PARALLEL TO SUPPORTS (Plywood Structural Panels Are Five-Ply, Five-Layer Unless Otherwise Noted)[a~~, b~~]

PANEL GRADE	THICKNESS (inch)	MAXIMUM SPAN (inches)	LOAD AT MAXIMUM SPAN (psf)	
			Live	Total
Structural I sheathing	$\frac{7}{16}$	24	20	30
	$\frac{15}{32}$	24	35[~~c~~b]	45[~~c~~b]
	$\frac{1}{2}$	24	40[~~c~~b]	50[~~c~~b]
	$\frac{19}{32}$, $\frac{5}{8}$	24	70	80
	$\frac{23}{32}$, $\frac{3}{4}$	24	90	100
Sheathing, other grades covered in DOC PS 1 or DOC PS 2	$\frac{7}{16}$	16	40	50
	$\frac{15}{32}$	24	20	25
	$\frac{1}{2}$	24	25	30
	$\frac{19}{32}$	24	40[~~c~~b]	50[~~c~~b]
	$\frac{5}{8}$	24	45[~~c~~b]	55[~~c~~b]
	$\frac{23}{32}$, $\frac{3}{4}$	24	60[~~c~~b]	65[~~c~~b]

For SI: 1 inch = 25.4 mm, 1 pound per square foot = 0.0479 kN/m^2.

~~a. Roof sheathing complying with this table shall be deemed to meet the design criteria of Section 2304.8.~~

~~b~~ a. Uniform load deflection limitations $^{1}/_{180}$ of span under live load plus dead load, $^{1}/_{240}$ under live load only. Edges shall be blocked with lumber or other approved type of edge supports.

~~c~~ b. For composite and four-ply plywood structural panel, load shall be reduced by 15 pounds per square foot.

TABLE 2304.8 (4) ALLOWABLE SPAN FOR WOOD STRUCTURAL PANEL COMBINATION SUBFLOOR-UNDERLAYMENT (SINGLE FLOOR)[a, ~~b~~] (Panels Continuous Over Two or More Spans and Strength Axis Perpendicular to Supports)

IDENTIFICATION	MAXIMUM SPACING OF JOISTS (inches)				
	16	20	24	32	48
Species group[~~c~~b]	Thickness (inches)				
1	$^{1}/_{2}$	$^{5}/_{8}$	$^{3}/_{4}$	—	—
2, 3	$^{5}/_{8}$	$^{3}/_{4}$	$^{7}/_{8}$	—	—
4	$^{3}/_{4}$	$^{7}/_{8}$	1	—	—
Single floor span rating[~~d~~c]	16 o.c.	20 o.c.	24 o.c.	32 o.c.	48 o.c.

For SI: 1 inch = 25.4 mm, 1 pound per square foot = 0.0479 kN/m ^{2}.

a. Spans limited to value shown because of possible effects of concentrated loads. Allowable uniform loads based on deflection of $^{1}/_{360}$ of span is 100 pounds per square foot except allowable total uniform load for 1 $^{1}/_{8}$ -inch wood structural panels over joists spaced 48 inches on center is 65 pounds per square foot. Panel edges shall have approved tongue-and-groove joints or shall be supported with blocking, unless $^{1}/_{4}$ -inch minimum thickness underlayment or 1 $^{1}/_{2}$ inches of approved cellular or lightweight concrete is placed over the subfloor, or finish floor is $^{3}/_{4}$ -inch wood strip.

~~b. Floor panels complying with this table shall be deemed to meet the design criteria of Section 2304.8.~~

~~c~~ b. Applicable to all grades of sanded exterior-type plywood. See DOC PS 1 for plywood species groups.

~~d~~ c. Applicable to Underlayment grade, C-C (Plugged) plywood, and Single Floor grade wood structural panels.

TABLE 2304.8 (3) ALLOWABLE SPANS AND LOADS FOR WOOD STRUCTURAL PANEL SHEATHING AND SINGLE-FLOOR GRADES CONTINUOUS OVER TWO OR MORE SPANS WITH STRENGTH AXIS PERPENDICULAR TO SUPPORTS[a, ~~b~~]

SHEATHING GRADES		ROOF[~~c~~b]		FLOOR[~~d~~c]

Panel span rating roof/ floor span	Panel thickness (inches)	Maximum span (inches)		Load[~~e~~d] (psf)		Maximum span (inches)
		With edge support[~~f~~e]	Without edge support	Total load	Live load	
16/0	$^{3}/_{8}$	16	16	40	30	0
20/0	$^{3}/_{8}$	20	20	40	30	0
24/0	$^{3}/_{8}$, $^{7}/_{16}$, $^{1}/_{2}$	24	20[~~g~~f]	40	30	0
24/16	$^{7}/_{16}$, $^{1}/_{2}$	24	24	50	40	16
32/16	$^{15}/_{32}$, $^{1}/_{2}$, $^{5}/_{8}$	32	28	40	30	16[~~h~~g]
40/20	$^{19}/_{32}$, $^{5}/_{8}$, $^{3}/_{4}$, $^{7}/_{8}$	40	32	40	30	20[g, h~~, i~~]
48/24	$^{23}/_{32}$, $^{3}/_{4}$, $^{7}/_{8}$	48	36	45	35	24
54/32	$^{7}/_{8}$, 1	54	40	45	35	32
60/32	$^{7}/_{8}$, $1^{1}/_{8}$	60	48	45	35	32
SINGLE FLOOR GRADES		**ROOF[~~e~~b]**				**FLOOR[~~d~~c]**
Panel span rating	**Panel thickness (inches)**	**Maximum span (inches)**		**Load[~~e~~d] (psf)**		**Maximum span (inches)**
		With edge support[~~f~~e]	**Without edge support**	**Total load**	**Live load**	
16 o.c.	$^{1}/_{2}$, $^{19}/_{32}$, $^{5}/_{8}$	24	24	50	40	16[~~h~~g]

20 o.c.	19/32, 5/8, 3/4	32	32	40	30	20[g, h~~, i~~]
24 o.c.	23/32, 3/4	48	36	35	25	24
32 o.c.	7/8, 1	48	40	50	40	32
48 o.c.	1 3/32, 1 1/8	60	48	50	40	48

For SI: 1 inch = 25.4 mm, 1 pound per square foot = 0.0479 kN/m^2.

a. Applies to panels 24 inches or wider.

~~b. Floor and roof sheathing complying with this table shall be deemed to meet the design criteria of Section 2304.8.~~

~~c~~ b. Uniform load deflection limitations 1/180 of span under live load plus dead load, 1/240 under live load only.

~~d~~ c. Panel edges shall have approved tongue-and-groove joints or shall be supported with blocking unless 1/4 -inch minimum thickness underlayment or 1 1/2 inches of approved cellular or lightweight concrete is placed over the subfloor, or finish floor is 3/4 -inch wood strip. Allowable uniform load based on deflection of 1/360 of span is 100 pounds per square foot except the span rating of 48 inches on center is based on a total load of 65 pounds per square foot.

~~e~~ d. Allowable load at maximum span.

~~f~~ e. Tongue-and-groove edges, panel edge clips (one midway between each support, except two equally spaced between supports 48 inches on center), lumber blocking or other. Only lumber blocking shall satisfy blocked diaphragm requirements.

~~g~~ f. For 1/2 -inch panel, maximum span shall be 24 inches.

~~h~~ g. Span is permitted to be 24 inches on center where 3/4 -inch wood strip flooring is installed at right angles to joist.

~~i~~ h. Span is permitted to be 24 inches on center for floors where 1 1/2 inches of cellular or lightweight concrete is applied over the panels.

TABLE 2304.8 (1)
ALLOWABLE SPANS FOR LUMBER FLOOR AND ROOF SHEATHING[~~a, b~~]

SPAN (inches)	MINIMUM NET THICKNESS (inches) OF LUMBER PLACED			
	Perpendicular to supports		Diagonally to supports	
	Surfaced dry[~~c~~a]	Surfaced unseasoned	Surfaced dry[~~c~~a]	Surfaced unseasoned
Floors				
24	3/4	25/32	3/4	25/32
16	5/8	11/16	5/8	11/16

Roofs				
24	$^{5}/_{8}$	$^{11}/_{16}$	$^{3}/_{4}$	$^{25}/_{32}$

For SI: 1 inch = 25.4 mm.

~~a. Installation details shall conform to Sections 2304.8.1 and 2304.8.2 for floor and roof sheathing, respectively.~~

~~b. Floor or roof sheathing complying with this table shall be deemed to meet the design criteria of Section 2304.8.~~

b.

~~c~~ a. Maximum 19-percent moisture content.

Reason: The purpose of this code change is to remove the redundant language contained within the footnotes. Section 2304.8.1 for roof sheathing and Section 2304.8.2 for floor sheathing state that sheathing conforming to the provisions of the Tables "shall be deemed to meet the requirements of this section." Repeating the language in the footnotes is unnecessary and should be deleted for simplicity. Also, in table 2304.8 (1) footnote a is removed because there are no installation details in either Sections 2304.8.1 or 2304.8.2.

Cost Impact: Will not increase the cost of construction
This proposal is intended to clarify the code and does not contain any new requirements nor is it removing any requirements for construction.

S271-16 : TABLE 2304.8-HUSTON13388

S272-16

IBC: 2304.10.1.

Proponent : Paul Coats, PE CBO, American Wood Council, representing American Wood Council (pcoats@awc.org)

2015 International Building Code

Revise as follows:

TABLE 2304.10.1
FASTENING SCHEDULE

DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION
Roof		
1. Blocking between ceiling joists, rafters or trusses to top plate or other framing below	3-8d common ($2^{1}/_{2}$″ × 0.131″); or 3-10d box (3″ × 0.128″); or 3-3″ × 0.131″ nails; or 3-3″14 gage staples, $^{7}/_{16}$″ crown	Each end, toenail
Blocking between rafters or truss not at the wall top plate, to rafter or truss	2-8d common (21 $/_{2}$″ × 0.131″) 2-3″ × 0.131″ nails 2-3″ 14 gage staples	Each end, toenail
	2-16 d common (31 $/_{2}$″ × 0.162″) 3-3″ × 0.131″ nails 3-3″ 14 gage staples	End nail
Flat blocking to truss and web filler	16d common ($3^{1}/_{2}$″ × 0.162″) @ 6″ o.c. 3″ × 0.131″ nails @ 6″ o.c. 3″ × 14 gage staples @ 6″ o.c	Face nail
2. Ceiling joists to top plate	3-8d common ($2^{1}/_{2}$″ × 0.131″); or 3-10d box (3″ × 0.128″); or 3-3″ × 0.131″ nails; or 3-3″ 14 gage staples, $^{7}/_{16}$″ crown	Each joist, toenail
3. Ceiling joist not attached to parallel rafter, laps over partitions (no thrust) (see Section 2308.7.3.1, Table 2308.7.3.1)	3-16d common ($3^{1}/_{2}$″ × 0.162″); or 4-10d box (3″ × 0.128″); or 4-3″ × 0.131″ nails; or 4-3″ 14 gage staples, $^{7}/_{16}$″ crown	Face nail

4. Ceiling joist attached to parallel rafter (heel joint) (see Section 2308.7.3.1, Table 2308.7.3.1)	Per Table 2308.7.3.1	Face nail
5. Collar tie to rafter	3-10d common (3″ × 0.148″); or 4-10d box (3″ × 0.128″); or 4-3″ × 0.131″ nails; or 4-3″ 14 gage staples, $^{7}/_{16}$ ″ crown	Face nail
6. Rafter or roof truss to top plate (See Section 2308.7.5, Table 2308.7.5)	3-10 common (3″ × 0.148″); or 3-16d box ($3^{1}/_{2}$ ″ × 0.135″); or 4-10d box (3″ × 0.128″); or 4-3″ × 0.131 nails; or 4-3″ 14 gage staples, $^{7}/_{16}$ ″ crown	Toenail[c]
7. Roof rafters to ridge valley or hip rafters; or roof rafter to 2-inch ridge beam	2-16d common (31 $/_{2}$ ″ × 0.162″); or 3-10d box (3″ × 0.128″); or 3-3″ × 0.131″ nails; or 3-3″ 14 gage staples, 7 $/_{16}$ ″ crown; or	End nail
	3-10d common (31 $/_{2}$ ″ × 0.148″); or 3-16d box (31 $/_{2}$ ″ × 0.135″); or 4-10d box (3″ × 0.128″); or 4-3″ × 0.131″ nails; or 4-3″ 14 gage staples, 7 $/_{16}$ ″ crown	Toenail

DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION
Wall		
8. Stud to stud (not at braced wall panels)	16d common ($3^{1}/_{2}$ ″ × 0.162″);	24″ o.c. face nail
	10d box (3″ × 0.128″); or 3″ × 0.131″ nails; or 3-3″ 14 gage staples, $^{7}/_{16}$ ″ crown	16″ o.c. face nail
9. Stud to stud and abutting studs at intersecting wall corners (at braced wall panels)	16d common ($3^{1}/_{2}$ ″ × 0.162″); or	16″ o.c. face nail
	16d box ($3^{1}/_{2}$ ″ × 0.135″); or	12″ o.c. face nail
	3″ × 0.131″ nails; or 3-3″ 14 gage staples,	

	$^7/_{16}$ ″ crown	12″ o.c. face nail
10. Built-up header (2″ to 2″ header)	16d common ($3^1/_2$ ″ × 0.162″); or	16″ o.c. each edge, face nail
	16d box ($3^1/_2$ ″ × 0.135″)	12″ o.c. each edge, face nail
11. Continuous header to stud	4-8d common ($2^1/_2$ ″ × 0.131″); or 4-10d box (3″ × 0.128″)	Toenail
12. Top plate to top plate	16d common ($3^1/_2$ ″ × 0.162″); or	16″ o.c. face nail
	10d box (3″ × 0.128″); or 3″ × 0.131″ nails; or 3″ 14 gage staples, $^7/_{16}$ ″ crown	12″ o.c. face nail
13. Top plate to top plate, at end joints	8-16d common ($3^1/_2$ ″ × 0.162″); or 12-10d box (3″ × 0.128″); or 12-3″ × 0.131″ nails; or 12-3″ 14 gage staples, $^7/_{16}$ ″ crown	Each side of end joint, face nail (minimum 24" lap splice length each side of end joint)
14. Bottom plate to joist, rim joist, band joist or blocking (not at braced wall panels)	16d common ($3^1/_2$ ″ × 0.162″); or	16" o.c. face nail
	16d box ($3^1/_2$ ″ × 0.135″); or 3″ × 0.131″ nails; or 3″ 14 gage staples, $^7/_{16}$ ″ crown	12" o.c. face nail
15. Bottom plate to joist, rim joist, band joist or blocking at braced wall panels	2-16d common ($3^1/_2$ ″ × 0.162″); or 3-16d box ($3^1/_2$ ″ × 0.135″); or 4-3″ × 0.131″ nails; or 4-3″ 14 gage staples, $^7/_{16}$ ″ crown	16" o.c. face nail
16. Stud to top or bottom plate	4-8d common ($2^1/_2$ ″ × 0.131″); or 4-10d box (3″ × 0.128″); or 4-3″ × 0.131″ nails; or 4-3″ 14 gage staples, $^7/_{16}$ ″ crown; or	Toenail
	2-16d common ($3^1/_2$ ″ × 0.162″); or 3-10d box (3″ × 0.128″); or 3-3″ × 0.131″ nails; or	End nail

	3-3″ 14 gage staples, $^7/_{16}$″ crown	
17. Top or bottom plate to stud	2-16d common ($3^1/_2$″ × 0.162″); or 3-10d box (3″ × 0.128″); or 3-3″ × 0.131″ nails; or 3-3″ 14 gage staples, $^7/_{16}$″ crown	End nail
18. Top plates, laps at corners and intersections	2-16d common ($3^1/_2$″ × 0.162″); or 3-10d box (3″ × 0.128″); or 3-3″ × 0.131″ nails; or 3-3″ 14 gage staples, $^7/_{16}$″ crown	Face nail

DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION
Wall		
19. 1″ brace to each stud and plate	2-8d common ($2^1/_2$″ × 0.131″); or 2-10d box (3″ × 0.128″); or 2-3″ × 0.131″ nails; or 2-3″ 14 gage staples, $^7/_{16}$″ crown	Face nail
20. 1″ × 6″ sheathing to each bearing	2-8d common ($2^1/_2$″ × 0.131″); or 2-10d box (3″ × 0.128″)	Face nail
21. 1″ × 8″ and wider sheathing to each bearing	3-8d common ($2^1/_2$″ × 0.131″); or 3-10d box (3″ × 0.128″)	Face nail
Floor		
22. Joist to sill, top plate, or girder	3-8d common ($2^1/_2$″ × 0.131″); or floor 3-10d box (3″ × 0.128″); or 3-3″ × 0.131″ nails; or 3-3″ 14 gage staples, $^7/_{16}$″ crown	Toenail
23. Rim joist, band joist, or blocking to top plate, sill or other framing below	8d common ($2^1/_2$″ × 0.131″); or 10d box (3″ × 0.128″); or 3″ × 0.131″ nails; or 3″ 14 gage staples, $^7/_{16}$″ crown	6" o.c., toenail

24. 1″ × 6″ subfloor or less to each joist	2-8d common ($2^1/_2$ ″ × 0.131″); or 2-10d box (3″ × 0.128″)	Face nail
25. 2″ subfloor to joist or girder	2-16d common ($3^1/_2$ ″ × 0.162″)	Face nail
26. 2″ planks (plank & beam – floor & roof)	2-16d common ($3^1/_2$ ″ × 0.162″)	Each bearing, face nail
27. Built-up girders and beams, 2″ lumber layers	20d common (4″ × 0.192″)	32" o.c., face nail at top and bottom staggered on opposite sides
	10d box (3″ × 0.128″); or 3″ × 0.131″ nails; or 3″ 14 gage staples, $^7/_{16}$ ″ crown	24" o.c. face nail at top and bottom staggered on opposite sides
	And: 2-20d common (4″ × 0.192″); or 3-10d box (3″ × 0.128″); or 3-3″ × 0.131″ nails; or 3-3″ 14 gage staples, $^7/_{16}$ ″ crown	Ends and at each splice, face nail
28. Ledger strip supporting joists or rafters	3-16d common ($3^1/_2$ ″ × 0.162″); or 4-10d box (3″ × 0.128″); or 4-3″ × 0.131″ nails; or 4-3″ 14 gage staples, $^7/_{16}$ ″ crown	Each joist or rafter, face nail
29. Joist to band joist or rim joist	3-16d common ($3^1/_2$ ″ × 0.162″); or 4-10d box (3″ × 0.128″); or 4-3″ × 0.131″ nails; or 4-3″ 14 gage staples, $^7/_{16}$ ″ crown	End nail
30. Bridging or blocking to joist, rafter or truss	2-8d common ($2^1/_2$ ″ × 0.131″); or 2-10d box (3″ × 0.128″); or 2-3″ × 0.131″ nails; or 2-3″ 14 gage staples, $^7/_{16}$ ″ crown	Each end, toenail

DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION
Wood structural panels (WSP), subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framinga		

		Edges (inches)	Intermediate supports (inches)
31. $\frac{3}{8}$″ – $\frac{1}{2}$″	6d common or deformed (2″ × 0.113″) (subfloor and wall)	6	12
	8d ~~box~~common or deformed ($2\frac{1}{2}$″ × ~~0.113~~0.131″) (roof), or RSRS-01 (2-3/8" x 0.113") nail (roof)[d]	6	12
	$2\frac{3}{8}$″ × 0.113″ nail (subfloor and wall)	6	12
	$1\frac{3}{4}$″ 16 gage staple, $\frac{7}{16}$″ crown (subfloor and wall)	4	8
	$2\frac{3}{8}$″ × 0.113″ nail (roof)	4	8
	$1\frac{3}{4}$″ 16 gage staple, $\frac{7}{16}$″ crown (roof)	3	6
32. $\frac{19}{32}$″ – $\frac{3}{4}$″	8d common ($2\frac{1}{2}$″ × 0.131″); or 6d deformed (2″ × 0.113″) (subfloor and wall)	6	12
	8d common or deformed (2-1/2" x 0.131") (roof), or RSRS-01 (2-3/8" x 0.113") nail (roof)[d]	6	12
	$2\frac{3}{8}$″ × 0.113″ nail; or 2″ 16 gage staple, $\frac{7}{16}$″ crown	4	8
33. $\frac{7}{8}$″ – $1\frac{1}{4}$″	10d common (3″ × 0.148″); or 8d deformed ($2\frac{1}{2}$″ × 0.131″)	6	12

Other exterior wall sheathing			
34. $^1/_2$″ fiberboard sheathing[b]	$1^1/_2$″ galvanized roofing nail ($^7/_{16}$″ head diameter); or $1^1/_4$″ 16 gage staple with $^7/_{16}$″ or 1″ crown	3	6
35. $^{25}/_{32}$″ fiberboard sheathing[b]	$1^3/_4$″ galvanized roofing nail ($^7/_{16}$″ diameter head); or $1^1/_2$″ 16 gage staple with $^7/_{16}$″ or 1″ crown	3	6
Wood structural panels, combination subfloor underlayment to framing			
36. $^3/_4$″ and less	8d common ($2^1/_2$″ × 0.131″); or 6d deformed (2″ × 0.113″)	6	12
37. $^7/_8$″ – 1″	8d common ($2^1/_2$″ × 0.131″); or 8d deformed ($2^1/_2$″ × 0.131″)	6	12
38. $1^1/_8$″ – $1^1/_4$″	10d common (3″ × 0.148″); or 8d deformed ($2^1/_2$″ × 0.131″)	6	12
Panel siding to framing			
39. $^1/_2$″ or less	6d corrosion-resistant siding ($1^7/_8$″ × 0.106″); or 6d corrosion-resistant casing (2″ × 0.099″)	6	12
40. $^5/_8$″	8d corrosion-resistant siding ($2^3/_8$″ × 0.128″); or 8d corrosion-resistant casing ($2^1/_2$″ × 0.113″)	6	12

DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF	SPACING AND LOCATION

	FASTENER		
Wood structural panels (WSP), subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framinga			
		Edges (inches)	**Intermediate supports (inches)**
Interior paneling			
41. $^{1}/_{4}$ ″	4d casing ($1^{1}/_{2}$ ″ × 0.080″); or 4d finish ($1^{1}/_{2}$ ″ × 0.072″)	6	12
42. $^{3}/_{8}$ ″	6d casing (2″ × 0.099″); or 6d finish (Panel supports at 24 inches)	6	12

For SI: 1 inch = 25.4 mm.

a. Nails spaced at 6 inches at intermediate supports where spans are 48 inches or more. For nailing of wood structural panel and particleboard diaphragms and shear walls, refer to Section 2305. Nails for wall sheathing are permitted to be common, box or casing.

b. Spacing shall be 6 inches on center on the edges and 12 inches on center at intermediate supports for nonstructural applications. Panel supports at 16 inches (20 inches if strength axis in the long direction of the panel, unless otherwise marked).

c. Where a rafter is fastened to an adjacent parallel ceiling joist in accordance with this schedule and the ceiling joist is fastened to the top plate in accordance with this schedule, the number of toenails in the rafter shall be permitted to be reduced by one nail.

d. RSRS-01 is a Roof Sheathing Ring Shank nail meeting the specifications in ASTM F1667.

Reason: This change brings consistency with the IRC for minimum nail size for roof sheathing attachment which is an 8d common nail (2-1/2" x 0.131"). The deformed nail option (2-1/2" x 0.131") is based on the assumption that the deformed nail, which has non-standard deformations, has at least the same withdrawal capacity and head pull through performance as the 8d common smooth shank nail.
This change also adds a new standardized Roof Sheathing Ring Shank (RSRS) nail for roof sheathing applications. The RSRS nail has been standardized in ASTM F1667 and added in this proposal as equivalent to the 8d common nail to resist uplift of roof sheathing. This standard ring shank nail provides improved withdrawal resistance relative to the 8d common smooth shank nail. A head size of 0.281" diameter is specified for the RSRS-01 nail in ASTM F1667 which is equivalent to the head diameter of the 8d common nail. The slightly larger net area under the head (i.e. area of head minus area of shank) is considered to provide slightly improved head pull through performance.

Cost Impact: Will not increase the cost of construction
Although there are technical changes, existing alternatives for attachment remain unchanged and a new ring shank nail option is added; therefore, there is no cost increase.

S272-16 : TABLE 2304.10.1-COATS11400

S273-16

IBC: 2304.10.1.

Proponent : Matthew Hunter, representing American Wood Council (mhunter@awc.org)

2015 International Building Code

Revise as follows:

TABLE 2304.10.1
FASTENING SCHEDULE

DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION
Roof		
1. Blocking between ceiling joists, rafters or trusses to top plate or other framing below	3-8d common ($2^1/_2$″ × 0.131″); or 3-10d box (3″ × 0.128″); or 3-3″ × 0.131″ nails; or 3-3″14 gage staples, $^7/_{16}$″ crown	Each end, toenail
Blocking between rafters or truss not at the wall top plate, to rafter or truss	2-8d common ($21/_2$″ × 0.131″) 2-3″ × 0.131″ nails 2-3″ 14 gage staples	Each end, toenail
	2-16 d common ($31/_2$″ × 0.162″) 3-3″ × 0.131″ nails 3-3″ 14 gage staples	End nail
Flat blocking to truss and web filler	16d common ($3^1/_2$″ × 0.162″) @ 6″ o.c. 3″ × 0.131″ nails @ 6″ o.c. 3″ × 14 gage staples @ 6″ o.c	Face nail
2. Ceiling joists to top plate	3-8d common ($2^1/_2$″ × 0.131″); or 3-10d box (3″ × 0.128″); or 3-3″ × 0.131″ nails; or 3-3″ 14 gage staples, $^7/_{16}$″ crown	Each joist, toenail
3. Ceiling joist not attached to parallel rafter, laps over partitions (no thrust) (see Section 2308.7.3.1, Table 2308.7.3.1)	3-16d common ($3^1/_2$″ × 0.162″); or 4-10d box (3″ × 0.128″); or 4-3″ × 0.131″ nails; or 4-3″ 14 gage staples, $^7/_{16}$″ crown	Face nail
4. Ceiling joist attached to parallel rafter		

(heel joint) (see Section 2308.7.3.1, Table 2308.7.3.1)	Per Table 2308.7.3.1	Face nail
5. Collar tie to rafter	3-10d common (3″ × 0.148″); or 4-10d box (3″ × 0.128″); or 4-3″ × 0.131″ nails; or 4-3″ 14 gage staples, $^{7}/_{16}$ ″ crown	Face nail
6. Rafter or roof truss to top plate (See Section 2308.7.5, Table 2308.7.5)	3-10 common (3″ × 0.148″); or 3-16d box ($3^{1}/_{2}$ ″ × 0.135″); or 4-10d box (3″ × 0.128″); or 4-3″ × 0.131 nails; or 4-3″ 14 gage staples, $^{7}/_{16}$ ″ crown	Toenail[c]
7. Roof rafters to ridge valley or hip rafters; or roof rafter to 2-inch ridge beam	2-16d common (31 /$_{2}$ ″ × 0.162″); or 3-10d box (3″ × 0.128″); or 3-3″ × 0.131″ nails; or 3-3″ 14 gage staples, 7 /$_{16}$ ″ crown; or	End nail
	3-10d common (3~~1 /2~~ ″ × 0.148″); or 4-16d box (31 /$_{2}$ ″ × 0.135″); or 4-10d box (3″ × 0.128″); or 4-3″ × 0.131″ nails; or 4-3″ 14 gage staples, 7 /$_{16}$ ″ crown	Toenail

DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION
Wall		
8. Stud to stud (not at braced wall panels)	16d common ($3^{1}/_{2}$ ″ × 0.162″);	24″ o.c. face nail
	10d box (3″ × 0.128″); or 3″ × 0.131″ nails; or 3-3″ 14 gage staples, $^{7}/_{16}$ ″ crown	16″ o.c. face nail
9. Stud to stud and abutting studs at intersecting wall corners (at braced wall panels)	16d common ($3^{1}/_{2}$ ″ × 0.162″); or	16″ o.c. face nail
	16d box ($3^{1}/_{2}$ ″ × 0.135″); or	12″ o.c. face nail
	3″ × 0.131″ nails; or 3-3″ 14 gage staples,	12″ o.c. face nail

	$^7/_{16}$″ crown	
10. Built-up header (2″ to 2″ header)	16d common ($3^1/_2$″ × 0.162″); or	16″ o.c. each edge, face nail
	16d box ($3^1/_2$″ × 0.135″)	12″ o.c. each edge, face nail
11. Continuous header to stud	4-8d common ($2^1/_2$″ × 0.131″); or 4-10d box (3″ × 0.128″)	Toenail
12. Top plate to top plate	16d common ($3^1/_2$″ × 0.162″); or	16″ o.c. face nail
	10d box (3″ × 0.128″); or 3″ × 0.131″ nails; or 3″ 14 gage staples, $^7/_{16}$″ crown	12″ o.c. face nail
13. Top plate to top plate, at end joints	8-16d common ($3^1/_2$″ × 0.162″); or 12-10d box (3″ × 0.128″); or 12-3″ × 0.131″ nails; or 12-3″ 14 gage staples, $^7/_{16}$″ crown	Each side of end joint, face nail (minimum 24" lap splice length each side of end joint)
14. Bottom plate to joist, rim joist, band joist or blocking (not at braced wall panels)	16d common ($3^1/_2$″ × 0.162″); or	16" o.c. face nail
	16d box ($3^1/_2$″ × 0.135″); or 3″ × 0.131″ nails; or 3″ 14 gage staples, $^7/_{16}$″ crown	12" o.c. face nail
15. Bottom plate to joist, rim joist, band joist or blocking at braced wall panels	2-16d common ($3^1/_2$″ × 0.162″); or 3-16d box ($3^1/_2$″ × 0.135″); or 4-3″ × 0.131″ nails; or 4-3″ 14 gage staples, $^7/_{16}$″ crown	16" o.c. face nail
16. Stud to top or bottom plate	4-8d common ($2^1/_2$″ × 0.131″); or 4-10d box (3″ × 0.128″); or 4-3″ × 0.131″ nails; or 4-3″ 14 gage staples, $^7/_{16}$″ crown; or	Toenail
	2-16d common ($3^1/_2$″ × 0.162″); or 3-10d box (3″ × 0.128″); or 3-3″ × 0.131″ nails; or	End nail

	3-3″ 14 gage staples, $^7/_{16}$ ″ crown	
~~17. Top or bottom plate to stud~~	~~2-16d common ($3^1/_2$ ″ × 0.162″); or 3-10d box (3″ × 0.128″); or 3-3″ × 0.131″ nails; or 3-3″ 14 gage staples, $^7/_{16}$ ″ crown~~	~~End nail~~
18. Top plates, laps at corners and intersections	2-16d common ($3^1/_2$ ″ × 0.162″); or 3-10d box (3″ × 0.128″); or 3-3″ × 0.131″ nails; or 3-3″ 14 gage staples, $^7/_{16}$ ″ crown	Face nail

DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION
Wall		
19. 1″ brace to each stud and plate	2-8d common ($2^1/_2$ ″ × 0.131″); or 2-10d box (3″ × 0.128″); or 2-3″ × 0.131″ nails; or 2-3″ 14 gage staples, $^7/_{16}$ ″ crown	Face nail
20. 1″ × 6″ sheathing to each bearing	2-8d common ($2^1/_2$ ″ × 0.131″); or 2-10d box (3″ × 0.128″)	Face nail
21. 1″ × 8″ and wider sheathing to each bearing	3-8d common ($2^1/_2$ ″ × 0.131″); or 3-10d box (3″ × 0.128″)	Face nail
Floor		
22. Joist to sill, top plate, or girder	3-8d common ($2^1/_2$ ″ × 0.131″); or floor 3-10d box (3″ × 0.128″); or 3-3″ × 0.131″ nails; or 3-3″ 14 gage staples, $^7/_{16}$ ″ crown	Toenail
23. Rim joist, band joist, or blocking to top plate, sill or other framing below	8d common ($2^1/_2$ ″ × 0.131″); or 10d box (3″ × 0.128″); or 3″ × 0.131″ nails; or 3″ 14 gage staples, $^7/_{16}$ ″ crown	6" o.c., toenail

24. 1″ × 6″ subfloor or less to each joist	2-8d common ($2^1/_2$″ × 0.131″); or 2-10d box (3″ × 0.128″)	Face nail
25. 2″ subfloor to joist or girder	2-16d common ($3^1/_2$″ × 0.162″)	Face nail
26. 2″ planks (plank & beam – floor & roof)	2-16d common ($3^1/_2$″ × 0.162″)	Each bearing, face nail
27. Built-up girders and beams, 2″ lumber layers	20d common (4″ × 0.192″)	32" o.c., face nail at top and bottom staggered on opposite sides
	10d box (3″ × 0.128″); or 3″ × 0.131″ nails; or 3″ 14 gage staples, $^7/_{16}$″ crown	24" o.c. face nail at top and bottom staggered on opposite sides
	And: 2-20d common (4″ × 0.192″); or 3-10d box (3″ × 0.128″); or 3-3″ × 0.131″ nails; or 3-3″ 14 gage staples, $^7/_{16}$″ crown	Ends and at each splice, face nail
28. Ledger strip supporting joists or rafters	3-16d common ($3^1/_2$″ × 0.162″); or 4-10d box (3″ × 0.128″); or 4-3″ × 0.131″ nails; or 4-3″ 14 gage staples, $^7/_{16}$″ crown	Each joist or rafter, face nail
29. Joist to band joist or rim joist	3-16d common ($3^1/_2$″ × 0.162″); or 4-10d box (3″ × 0.128″); or 4-3″ × 0.131″ nails; or 4-3″ 14 gage staples, $^7/_{16}$″ crown	End nail
30. Bridging or blocking to joist, rafter or truss	2-8d common ($2^1/_2$″ × 0.131″); or 2-10d box (3″ × 0.128″); or 2-3″ × 0.131″ nails; or 2-3″ 14 gage staples, $^7/_{16}$″ crown	Each end, toenail

DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION
Wood structural panels (WSP), subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framinga		

		Edges (inches)	Intermediate supports (inches)
31. 3/8″ – 1/2″	6d common or deformed (2″ × 0.113″) (subfloor and wall)	6	12
	8d ~~box~~common or deformed (2 1/2″ × 0.~~113~~ 0.131″) (roof)	6	12
	2 3/8″ × 0.113″ nail (subfloor and wall)	6	12
	1 3/4″ 16 gage staple, 7/16″ crown (subfloor and wall)	4	8
	2 3/8″ × 0.113″ nail (roof)	4	8
	1 3/4″ 16 gage staple, 7/16″ crown (roof)	3	6
32. 19/32″ – 3/4″	8d common (2 1/2″ × 0.131″)~~;~~ or 6d deformed (2″ × 0.113″) (subfloor and wall)	6	12
	8d common or deformed (2 1/2" × 0.131") (roof)	6	12
	2 3/8″ × 0.113″ nail; or 2″ 16 gage staple, 7/16″ crown	4	8
33. 7/8″ – 1 1/4″	10d common (3″ × 0.148″); or 8d deformed (2 1/2″ × 0.131″)	6	12
Other exterior wall sheathing			
	1 1/2″ galvanized roofing nail (7		

34. 1/2″ fiberboard sheathing[b]	/16″ head diameter); or 1 1/4″ 16 gage staple with 7/16″ or 1″ crown	3	6
35. 25/32″ fiberboard sheathing[b]	1 3/4″ galvanized roofing nail (7/16″ diameter head); or 1 1/2″ 16 gage staple with 7/16″ or 1″ crown	3	6
Wood structural panels, combination subfloor underlayment to framing			
36. 3/4″ and less	8d common (2 1/2″ × 0.131″); or 6d deformed (2″ × 0.113″)	6	12
37. 7/8″ – 1″	8d common (2 1/2″ × 0.131″); or 8d deformed (2 1/2″ × 0.131″)	6	12
38. 1 1/8″ – 1 1/4″	10d common (3″ × 0.148″); or 8d deformed (2 1/2″ × 0.131″)	6	12
Panel siding to framing			
39. 1/2″ or less	6d corrosion-resistant siding (1 7/8″ × 0.106″); or 6d corrosion-resistant casing (2″ × 0.099″)	6	12
40. 5/8″	8d corrosion-resistant siding (2 3/8″ × 0.128″); or 8d corrosion-resistant casing (2 1/2″ × 0.113″)	6	12

DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION
Wood structural panels (WSP), subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framinga		

		Edges (inches)	Intermediate supports (inches)
Interior paneling			
41. 1/4″	4d casing (1 1/2″ × 0.080″); or 4d finish (1 1/2″ × 0.072″)	6	12
42. 3/8″	6d casing (2″ × 0.099″); or 6d finish (Panel supports at 24 inches)	6	12

For SI: 1 inch = 25.4 mm.

a. Nails spaced at 6 inches at intermediate supports where spans are 48 inches or more. For nailing of wood structural panel and particleboard diaphragms and shear walls, refer to Section 2305. Nails for wall sheathing are permitted to be common, box or casing.

b. Spacing shall be 6 inches on center on the edges and 12 inches on center at intermediate supports for nonstructural applications. Panel supports at 16 inches (20 inches if strength axis in the long direction of the panel, unless otherwise marked).

c. Where a rafter is fastened to an adjacent parallel ceiling joist in accordance with this schedule and the ceiling joist is fastened to the top plate in accordance with this schedule, the number of toenails in the rafter shall be permitted to be reduced by one nail.

Reason: Item 7. The correct length of the 10d common nail is 3" not 3-1/2". 10d common is correctly shown as 3" long elsewhere in the table. The equivalent number of 16d box nails to the common nail reference is 4. This change makes the specified nailing consistent with IRC Table R602.3(1).
Item 17. Top or bottom plate to stud nailing is redundant with nailing in Item 16. Item 16 includes both toenail and end nail option. Item 16 end nail option is identical to the end nail option described in item 17.

Item 31. This change brings consistency with the IRC for minimum nail size for roof sheathing attachment which is an 8d common nail (2-1/2" x 0.131"). The 8d common is a smooth shank nail.

Item 32. The deformed nail option (2-1/2" x 0.131") is based on the assumption that the deformed nail has at least the same withdrawal capacity and head pull through performance of the equivalent diameter 8d common smooth shank nail.

Cost Impact: Will not increase the cost of construction
Nail sizes are editorially fixed, redundancy removed, and with size consistent with recognized sizes in IRC, therefore increased costs are not anticipated.

S273-16 : TABLE 2304.10.1-HUNTER11289

S274-16

IBC: 2304.10.5.

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

2015 International Building Code

Revise as follows:

2304.10.5 Fasteners and connectors in contact with preservative-treated and fire-retardant-treated wood. Fasteners, including nuts and washers, and connectors in contact with *preservative-treated* and *fire-retardant-treated wood* shall be in accordance with Sections 2304.10.5.1 through 2304.10.5.4. The coating weights for zinc-coated fasteners shall be in accordance with ASTM A 153. Stainless steel driven fasteners shall be in accordance with the material requirements of ASTM F 1667

Reason: In the last code cycle the requirements for stainless steel complying to the requirements of ASTM F 1667 w ere included in the IRC (Group B), how ever the proposal w as crafted after the IBC (Group A) cycle had completed. This proposal is intended to bring over the approved language from the IRC into the IBC so that the tw o documents contain the same requirements for the permissible type of stainless steel fastener used in treated w ood.

The follow ing is the reason statement used in the approval of RB176-13:

ASTM F 1667 reads as follows:

6. *Material Requirements*

6.1 *Steel wire used in the manufacture of driven fasteners shall be of low carbon, medium-low carbon, or medium-high carbon.*

6.2 *Stainless steel wire used in the manufacture of driven fasteners shall be of Types 302, 304, 305, or 316.*

So the intent here is to require fasteners used with treated wood be manufactured from Types 302, 304, 305, or 316 stainless steel.

There has been a lot of work done on fasteners and connectors in contact with treated wood in the last 8-10 years. All of the testing and historical performance of stainless steel were based on the traditional use of 300 series of stainless steel. Tet there are many types of stainless steel, and some are much less corrosion resistant than others. By limiting the types of stainless steel to these specific series, it ensures that the stainless steel fasteners will be corrosion resistant when exposed to treated wood.

There is precedent for this. Section 402.1.1 specifies that for wood foundation s stainless steel fasteners must be "of Type 304 or 316 stainless steel". Section R905.10.4 states "Copper, brass, copper alloy and 300-series stainless steel fasteners shall be used for copper roofs". Further, ASTM F 1667 is already specified for several different types of fasteners in the IRC. The result of this proposal is not to require the exclusive use of 300-series stainless steel fasteners. This section permits hot-dipped, zinc-coated galvanized steel, stainless steel, silicon bronze, or copper fasteners. The existing sentence before the added one is meant to specify a minimum coating weight for the galvanized fasteners so they perform as expected. The new proposed sentence does the same thing for stainless steel fasteners.

Cost Impact: Will increase the cost of construction
The majority of driven stainless steel fasteners are already manufactured from 300 series stainless steel. If a manufacturer w ere supplying the lesser performing (and low er cost) stainless steel types then there w ould be a

cost increase in going to the 300 series stainless steel. The increase in performance w ould justify the additional cost. How ever, the use of stainless steel fasteners is not required, and w here stainless steel is not used no increase w ould be incurred.

S274-16 : 2304.10.5-HUSTON13406

S275-16

Part I:

IBC: 2304.10.5.1, 2304.10.5.3.

Part II:

IRC: R317.3.1, R317.3.3.

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC-BUILDING COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

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

Part I

2015 International Building Code

Revise as follows:

2304.10.5.1 Fasteners and connectors for preservative-treated wood. Fasteners, including nuts and washers, in contact with *preservative-treated wood* shall be of hot-dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper. Staples shall be of stainless steel. Fasteners other than nails, staples, timber rivets, wood screws and lag screws shall be permitted to be of mechanically deposited zinc-coated steel with coating weights in accordance with ASTM B 695, Class 55 minimum. Connectors that are used in exterior applications and in contact with *preservative-treated wood*shall have coating types and weights in accordance with the treated wood or connector manufacturer's recommendations. In the absence of manufacturer's recommendations, a minimum of ASTM A 653, Type G185 zinc-coated galvanized steel, or equivalent, shall be used.

> **Exception:** Plain carbon steel fasteners, including nuts and washers, in SBX/DOT and zinc borate *preservative-treated wood* in an interior, dry environment shall be permitted.

2304.10.5.3 Fasteners for fire-retardant-treated wood used in exterior applications or wet or damp locations. Fasteners, including nuts and washers, for *fire-retardant-treated wood* used in exterior applications or wet or damp locations shall be of hot-dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper. Staples shall be of stainless steel. Fasteners other than nails, staples, timber rivets, wood screws and lag screws shall be permitted to be of mechanically deposited zinc-coated steel with coating weights in accordance with ASTM B 695, Class 55 minimum.

Part II

2015 International Residential Code

Revise as follows:

R317.3.1 Fasteners for preservative-treated wood. Fasteners, including nuts and washers, for preservative-treated wood shall be of hot-dipped, zinc-coated galvanized steel, stainless steel, silicon bronze or copper. Staples shall be of stainless steel. Coating types and weights for connectors in contact with preservative-treated wood shall be in accordance with the connector manufacturer's recommendations. In the absence of manufacturer's recommendations, a minimum of ASTM A 653 type G185 zinc-coated galvanized steel, or equivalent, shall be used.

Exceptions:

1. one/two ($^1/_2$)-inch-diameter (12.7 mm) or greater steel bolts.
2. Fasteners other than nails<u>, staples,</u> and timber rivets shall be permitted to be of mechanically deposited zinc-coated steel with coating weights in accordance with ASTM B 695, Class 55 minimum.
3. Plain carbon steel fasteners in SBX/DOT and zinc borate preservative-treated wood in an interior, dry environment shall be permitted.

R317.3.3 Fasteners for fire-retardant-treated wood used in exterior applications or wet or damp locations. Fasteners, including nuts and washers, for fire-retardant-treated wood used in exterior applications or wet or damp locations shall be of hot-dipped, zinc-coated galvanized steel, stainless steel, silicon bronze or copper. Fasteners other than nails<u>, staples,</u> and timber rivets shall be permitted to be of mechanically deposited zinc-coated steel with coating weights in accordance with ASTM B 695, Class 55 minimum.

Reason: Over the past several code cycles staples have been included as another type of fastener used in various types of w ood-to-w ood connections. The intention of this proposal is to better integrate staples into the code so that the provisions for small diameter fasteners (nail and timber rivets) also are explicitly extended to staples w here applicable. This repeatedly occurs in the limitations for fasteners in treated w ood. The phrase "other than nails and timber rivets" is being rew ritten to include staples as w ell. This occurs in both the IBC and IRC in sections: 2304.10.5.1, 2304.10.5.3. R317.3.1 and R317.3.3.

The second part of this proposal is to specifically limit staples to stainless steel w here exposed to high corrosion environments. The thin w ire gages used in staple fasteners (16ga – 14ga) are much thinner than those used in nails, and are consequentially more susceptible to corrosion. Also, according to ICC ESR-1539 report for pow er-drive staples and nails, currently stainless steel staples are the only available option for staples to meet the increased corrosion resistance requirements of sections 2304.10.5.1 and R317.3.1. By specifically specifying staples as requiring stainless steel this avoids confusion and possible misuse of other types of staples in increased corrosion risk applications.

Cost Impact: Will increase the cost of construction
Currently w hen staples are used in treated w ood the only know n available option is to use stainless steel staples. In this case there w ould be no cost increase in construction.
For use in treated w ood if staples are not presently stainless then there w ould be a slight cost increase, how ever w e do not feel that these staples w ould be code conforming. In this case any increase in performance w ould justify the additional cost

S275-16 : 2304.10.5.1-HUSTON13410

S276-16

IBC: 2304.11.

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

2015 International Building Code

Revise as follows:

2304.11 Heavy timber construction. Where a structure or portion thereof is required to be of Type IV construction by other provisions of this code, the building elements therein shall comply with the applicable provisions of Sections 2304.11.1 through 2304.11.5. Lumber decking shall also be in accordance with Section 2304.9.

Reason: The intent of this change is to help the user be aw are of Section 2304.9 applicable to heavy timber for the detailing and fastening of lumber decking. this section w as revised in G 179 of the Group A cycle. There is no intent to modify changes already made to this section in G 179. The intent of this section is to add the w ords "Lumber decking shall also be in accordance w ith Section 2304.9." at the end of the final language approved in to 2304.11 in G 179 as a pointer to Section 2304.9.

Cost Impact: Will not increase the cost of construction
This code change correlates existing section to assist users of the code.

S276-16 : 2304.11-RICHARDSON13296

S277-16

IBC: 2304.12.1, 2304.12.2, 2304.12.2.4.

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~~ preservative treatment 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.

~~2304.12.2.4~~ 2304.12.1.6 ~~Laminated timbers~~ Exposed wood members. The portions of ~~glued-laminated timbers~~ wood members 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 or be manufactured from naturally durable or *preservative-treated wood*.

2304.12.2 Other locations. Wood used in the locations specified in Sections 2304.12.2.1 through 2304.12.2.5 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~~ 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 w ater-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-w aterborne 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 w ood against decay or termites. How ever, 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, w here sleepers and sills, and w ood siding are located. In fact, the requirements in the existing Section 2304.12.2.4 should be applied to all exposed w ood 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 w ill not increase the cost of construction as it simply recognizes the general intent of using preservative-treated w ood under exposed conditions.

S277-16 : 2304.12.1-KEITH11102

S278-16

IBC: 2304.12.2.2.

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 ~~projected~~ 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, Suporting member for permanent appurtanences**. 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 proposeal attempts to better define exposed to the weather by referencing the clearer description in 2304.12.2.3.

A second modificaiton 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 w ood and not use the base w ith 1" pedestal.

S278-16 : 2304.12.2.2-SHACKELFORD13246

S279-16

IBC: 2304.12.2.5.

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. <u>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.</u>

Reason: A key functional requirement of impervious moisture barrier systems installed under a permeable floor system exposed to w ater are elements that provide for drainage of any w ater making it's w ay through the permeable floor system. Without a properly functioning method to transport this w ater 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 w ater to drain out.

Cost Impact: Will increase the cost of construction
Drainage elements betw een the permeable floor slab and impervious barrier are commonly called for and installed by many practitioners and w ill not change the cost of construction in those cases. How ever in cases w here no method to provide positive drainage is currently provided, this proposal w ill increase the cost of construction.

S279-16 : 2304.12.2.5-RICHARDSON12652

S280-16

IBC: 2304.8.1, 2304.8.2.

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

2015 International Building Code

Revise as follows:

2304.8.1 Structural floor sheathing. Structural floor sheathing shall be designed in accordance with the general provisions of this code~~ and the special provisions in this section~~.

Floor sheathing conforming to the provisions of Table 2304.8(1), 2304.8(2), 2304.8(3) or 2304.8(4) shall be deemed to meet the requirements of this section.

2304.8.2 Structural roof sheathing. Structural roof sheathing shall be designed in accordance with the general provisions of this code and the special provisions in this section.

Roof sheathing conforming to the provisions of Table 2304.8(1), 2304.8(2), 2304.8(3) or 2304.8(5) shall be deemed to meet the requirements of this section. Wood structural panel roof sheathing shall be ~~bonded by~~ <u>of a type manufactured with</u> exterior glue<u> (Exposure 1 or Exterior)</u>.

Reason: This proposal is two-fold with the attempt to clarify and harmonize the code.

1. To clean-up the code and remove redundant language, the wording "and the special provisions in this section" is being removed from Section 2304.8.1 for structural floor sheathing. There are currently no provisions contained in this section, so the wording is meaningless. Leaving this phrase in this section only creates confusion and thus the wording should be removed.
2. In section 2304.8.2 the reference to exterior glue is changed to reflect the wording contained in section 2304.6.1 for exterior sheathing. As it stands the reference to "bonded by exterior glue" is ambiguous, and can be mistaken to mean the bond classification of the wood structural panel as defined in DOC PS1 or PS2. Identical wording contained in 2304.6.1 is used here to better reflect the intention of the code.

Cost Impact: Will not increase the cost of construction
This proposal is intended to clarify the code and does not contain any new requirements nor is it removing any requirements for construction

S280-16 : 2304.8.1-HUSTON13400

S281-16

IBC: 2304.9.3.2, 2304.9.3.2 (New).

Proponent : David Tyree, representing American Wood Council (dtyree@awc.org)

2015 International Building Code

Revise as follows:

2304.9.3.2 Nailing. The length of nails connecting laminations shall be not less than two and one-half times the net thickness of each lamination. Where decking supports are 48 inches (1219 mm) on center or less, side nails shall be installed not more than 30 inches (762 mm) on center alternating between top and bottom edges, and staggered one-third of the spacing in adjacent laminations. Where supports are spaced more than 48 inches (1219 mm) on center, side nails shall be installed not more than 18 inches (457 mm) on center alternating between top and bottom edges and staggered one-third of the spacing in adjacent laminations. For mechanically laminated decking constructed with laminations of 2-inch (51 mm) nominal thickness, nailing in accordance with Table 2304.9.3.2 shall be permitted. Two side nails shall be installed at each end of butt-jointed pieces.

Laminations shall be toenailed to supports with 20d or larger common nails. Where the supports are 48 inches (1219 mm) on center or less, alternate laminations shall be toenailed to alternate supports; where supports are spaced more than 48 inches (1219 mm) on center, alternate laminations shall be toenailed to every support. For mechanically laminated decking constructed with laminations of 2-inch (51 mm) nominal thickness, toenailing at supports in accordance with Table 2304.9.3.2 shall be permitted.

Add new text as follows:

TABLE 2304.9.3.2
FASTENING SCHEDULE FOR MECHANICALLY LAMINATED DECKING USING LAMINATIONS OF 2-INCH NOMINAL THICKNESS

MINIMUM NAIL SIZE (Length x Diameter)	MAXIMUM SPACING BETWEEN FACE NAILS [a,b] (inches)		NUMBER OF TOENAILS INTO SUPPORTS [c]
	Decking Supports ≤ 48 inches o.c.	Decking Supports > 48 inches o.c.	
4" x 0.192"	30	18	1
4" x 0.162"	24	14	2
4" x 0.148"	22	13	2
3$^{1}/_{2}$" x 0.162"	20	12	2

3 1/2" x 0.148"	19	11	2
3 1/2" x 0.135"	17	10	2
3" x 0.148"	11	7	2
3" x 0.128"	9	5	2
2 3/4" x 0.148"	10	6	2
2 3/4" x 0.131"	9	6	3
2 3/4" x 0.120"	8	5	3

For SI: 1 inch = 25.4 mm

a. Nails shall be driven perpendicular to the lamination face, alternating between top and bottom edges.

b. Where nails penetrate through two laminations and into the third, they shall be staggered one-third of the spacing in adjacent laminations. Otherwise, nails shall be staggered one-half of the spacing in adjacent laminations.

c. Where supports are 48 inches (1219 mm) on center or less, alternate laminations shall be toenailed to alternate supports; where supports are spaced more than 48 inches (1219 mm) on center, alternate laminations shall be toenailed to every support.

Reason: This proposal adds alternative fastener schedules for construction of mechanically laminated decking, providing specific guidance for use of mechanically-driven nails which are typically used in construction. The alternative fastening schedules are based on equivalency to the reference 20d common nail currently required in 2304.9.3.2 for laminations with a 2-inch nominal thickness, and provide equivalent lateral strength, shear stiffness and withdrawal capacity, as calculated in accordance with the AWC NDS.

Cost Impact: Will not increase the cost of construction
This change does not add additional requirements. It provides equivalent alternative options for construction of mechanically laminated decking.

S281-16 : 2304.9.3.2-TYREE12558

S282-16

IBC: 2305.2.

Proponent : Stephen Kerr, representing Self (skerr@jwa-se.com)

2015 International Building Code

Revise as follows:

2305.2 Diaphragm deflection. The deflection of wood-frame diaphragms shall be determined in accordance with AWC SDPWS. The deflection (~~?~~ Δ_{dia}) of a blocked wood structural panel diaphragm uniformly fastened throughout with staples is permitted to be calculated in accordance with Equation 23-1. If not uniformly fastened, the constant 0.188 (For SI: 1/1627) in the third term shall be modified by an approved method.

$$\cancel{\phi M_n = 3\sqrt{f'_c}S_m} \quad \text{~~(Equation 23-1)~~}$$

Δ_{dia} = 5vL³/8EAW + vL/4Gt + 0.188Le$_n$ + ∑(xΔ$_c$)/2W **(Equation 23-1)**

~~For SI: Δ~~ = $$\cancel{\frac{0.052vL^3}{EAb} + \frac{vL}{4Gt} + \frac{Le_n}{1627} + \frac{\Sigma(\Delta_c X)}{2b}}$$

For SI: Δ_{dia} = 0.052vL³/EAW + vL/4Gt + Le$_n$/1627 + ∑(xΔ$_c$)/2W

where:

~~*A*~~	~~=~~	~~Area of chord cross section, in square inches (mm²).~~
~~*b*~~	~~=~~	~~Diaphragm width, in feet (mm).~~
~~*E*~~	~~=~~	~~Elastic modulus of chords, in pounds per square inch (N/ mm²).~~
~~*e$_n$*~~	~~=~~	~~Staple deformation, in inches (mm) [see Table 2305.2(1)].~~
~~*Gt*~~	~~=~~	~~Panel rigidity through the thickness, in pounds per inch (N/mm) of panel width or depth [see Table 2305.2(2)].~~
~~*L*~~	~~=~~	~~Diaphragm length, in feet (mm).~~
~~*v*~~	~~=~~	~~Maximum shear due to design loads in the direction under consideration, in pounds per linear foot (plf) (N/mm).~~
~~Δ~~	~~=~~	~~The calculated deflection, in inches (mm).~~
~~Σ(Δ$_c$X)~~	~~=~~	~~Sum of individual chord-splice slip values on both sides of the diaphragm, each multiplied by its distance to the nearest support.~~
A	=	Area of chord cross section, in square inches (mm²).
W	=	Diaphragm width (In the direction of applied force, in feet (mm).
E	=	Modulus of elasticity of diaprhagm chords, in pounds per square inch (N/ mm²).
e$_n$	=	Staple slip, in inches (mm) [see Table 2305.2(1)].
Gt	=	Panel rigidity through the thickness, in pounds per inch (N/mm) of panel width or depth [see Table 2305.2(2)].
L	=	Diaphragm length (dimension perpendicular to the direction of the applied load), in feet (mm).
v	=	Induced unit shear, in pounds per linear foot (plf) (N/mm).
Δ_{dia}	=	Maximum mid-span diaphragm deflection determined by elastic analysis, in inches (mm).

x = Distance from chord splice to nearest support, ft. (mm)

Δ_c = Diaprhagm chord splice slip at the induced unit shear, in. (mm)

Reason: Currently for horizontal diaphragms and vertical shear walls staples are not included within the AWC SDPWS referenced standard and are only contained within chapter 23 of the IBC. Section 2305.2 is essentially a modification of the AWC SDPWS requirements for nails modified for staples. Since the inclusion of staples in

chapter 23 the SDPWS terminology has been modified and any similar changes have not been included into the IBC. The intent of this proposal is to bring parity betw een the language of SDPWS and the IBC. No technical change to the deflection formulas is considered.

Cost Impact: Will not increase the cost of construction
The proposed change w ill not impact the cost of construction. The change is editorial and no technical changes are considered.

S282-16 : 2305.2-KERR13165

S283-16

IBC: 2305.2.

Proponent : Stephen Kerr, representing Self (skerr@jwa-se.com)

2015 International Building Code

TABLE 2305.2 (2)
VALUES OF *Gt* FOR USE IN CALCULATING DEFLECTION OF WOOD STRUCTURAL PANEL SHEAR WALLS AND DIAPHRAGMS

PANEL TYPE	SPAN RATING	VALUES OF *Gt* (lb/in. panel depth or width)							
		Structural Sheathing ~~other~~				Structural I			
		Plywood			OSB	Plywood			OSB
		3-ply ~~plywood~~	4- ply ~~plywood~~	5-ply ~~plywood~~[a]		3-ply ~~plywood~~	4- ply ~~plywood~~	5-ply ~~plywood~~[a]	
Sheathing	24/0	25,000	32,500	37,500	77,500	32,500	42,500	41,500	77,500
	24/16	27,000	35,000	40,500	83,500	35,000	45,500	44,500	83,500
	32/16	27,000	35,000	40,500	83,500	35,000	45,500	44,500	83,500
	40/20	28,500	37,000	43,000	88,500	37,000	48,000	47,500	88,500
	48/24	31,000	40,500	46,500	96,000	40,500	52,500	51,000	96,000
Single Floor	16 o.c.	27,000	35,000	40,500	83,500	35,000	45,500	44,500	83,500
	20 o.c.	28,000	36,500	42,000	87,000	36,500	47,500	46,000	87,000
	24 o.c.	30,000	39,000	45,000	93,000	39,000	50,500	49,500	93,000
	32 o.c.	36,000	47,000	54,000	110,000	47,000	61,000	59,500	110,000
	48 o.c.	50,500	65,500	76,000	155,000	65,500	85,000	83,500	155,000

		~~Other~~ Structural Sheathing			Structural I		
–	Thickness (in.)	A-A, A-C	Marine	All Other Grades	A-A, A-C	Marine	All Other Grades
	1/4	24,000	31,000	24,000	31,000	31,000	31,000
	11/32	25,500	33,000	25,500	33,000	33,000	33,000
	3/8	26,000	34,000	26,000	34,000	34,000	34,000
	15/32	38,000	49,500	38,000	49,500	49,500	49,500
	1/2	38,500	50,000	38,500	50,000	50,000	50,000
Sanded Plywood	19/32	49,000	63,500	49,000	63,500	63,500	63,500
	5/8	49,500	64,500	49,500	64,500	64,500	64,500
	23/32	50,500	65,500	50,500	65,500	65,500	65,500
	3/4	51,000	66,500	51,000	66,500	66,500	66,500

7/8	52,500	68,500	52,500	68,500	68,500	68,500
1	73,500	95,500	73,500	95,500	95,500	95,500
1 1/8	75,000	97,500	75,000	97,500	97,500	97,500

For SI: 1 inch = 25.4 mm, 1 pound/inch = 0.1751 N/mm.

a. 5-ply applies to plywood with five or more layers. For 5-ply plywood with three layers, use values for 4- ply panels.

~~For SI: 1 inch = 25.4 mm, 1 pound/inch = 0.1751 N/mm.~~

~~a. Applies to plywood with five or more layers; for five-ply/three-layer plywood, use values for four-ply.~~

Reason: The proposed change is editorial and is intending to clarify the table and bring it more in line with the AWC SDPWS referenced standard. First, the proposal will remove some confusion in the table and footnote regarding the difference between the number of layers of plywood and the number of ply's. The wording for the proposed change is taken from 2008 AWC SDPWS Table C4.2.2A footnote 3. Second, the category "other" is removed and replaced with the term "structural sheathing". This is to recognize that the wood structural panel sheathing used in shear walls are structural panels and in accordance with section 2303.1.5 shall conform to the requirements of either DOC PS1, DOC PS2, or ANSI/APA PRP210.

Cost Impact: Will not increase the cost of construction
The proposed change will not impact the cost of construction. The change is editorial and no technical changes are considered.

S283-16 : TABLE 2305.2-KERR13231

S284-16

IBC: 2305.3.

Proponent : Stephen Kerr, representing Self (skerr@jwa-se.com)

2015 International Building Code

Revise as follows:

2305.3 Shear wall deflection. The deflection of wood-frame shear walls shall be determined in accordance with AWC SDPWS. The deflection (Δ) of a blocked wood structural panel shear wall uniformly fastened throughout with staples is permitted to be calculated in accordance with Equation 23-2.

$$\frac{vh^3}{3EAb} + \frac{vh}{Gt} + \frac{he_n}{407.6} + d_a\frac{h}{b} \quad \Delta sw = 8vh^3/EAb + vh/Gt + 0.75he_n + d_a h/b$$ **(Equation 23-2)**

For SI: Δsw = $$\frac{vh^3}{3EAb} + \frac{vh}{Gt} + \frac{he_n}{407.6} + d_a\frac{h}{b} \quad vh^3/3EAb + vh/Gt + he_n/407.6 + d_a h/b$$

where:

A	=	Area of ~~boundary element cross section~~ end post cross-section in square inches (mm^2) ~~(vertical member at shear wall boundary)~~.
b	=	~~Wall width~~ Shear wall length, in feet (mm).
~~d~~ Δ_a	=	~~Vertical~~ Total vertical elongation of ~~overturning~~ wall anchorage system (including fastener slip, device elongation, ~~anchor~~ rod elongation, etc.) at the ~~design~~ induced unit shear ~~load~~ in the shear wall (v).
E	=	~~Elastic modulus~~ Modulus of ~~boundary element (vertical member at shear wall boundary)~~ elasticity of end posts, in pounds per square inch (N/mm^2).
e_n	=	Staple ~~deformation~~ slip, in inches (mm) [see Table 2305.2(1)].
G_t	=	Panel rigidity through the thickness, in pounds per inch (N/mm) of panel width or depth [see Table 2305.2(2)].
h	=	~~Wall~~ Shear wall height, in feet (mm).
v	=	~~Maximum~~ Induced unit shear ~~due to design loads at the top of the wall~~, in pounds per linear foot (N/mm).
Δsw	=	~~The calculated~~ Maximum shear wall deflection determined by elatic analysis, in inches (mm).

Reason: Currently for horizontal diaphragms and vertical shear walls staples are not included within the AWC SDPWS referenced standard and are only contained within chapter 23 of the IBC. Section 2305.2 is essentially a modification of the AWC SDPWS requirements for nails modified for staples. Since the inclusion of staples in chapter 23 the SDPWS terminology has been modified and any similar changes have not been included into the IBC. The intent of this proposal is to bring parity between the language of SDPWS and the IBC. No technical change to the deflection formulas is considered.

Cost Impact: Will not increase the cost of construction
The proposed change will not impact the cost of construction. The change is editorial and no technical changes are considered.

S284-16 : 2305.3-KERR13218

S285-16

IBC: 2306.3.

Proponent : Stephen Kerr, representing Self (skerr@jwa-se.com)

2015 International Building Code

Revise as follows:

TABLE 2306.3 (1)
ALLOWABLE SHEAR VALUES (POUNDS PER FOOT) FOR WOOD STRUCTURAL PANEL SHEAR WALLS UTILIZING STAPLES WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE[a] FOR WIND OR SEISMIC LOADING[b, f, g, i]

PANEL GRADE	MINIMUM NOMINAL PANEL THICKNESS (inch)	MINIMUM FASTENER PENETRATION IN FRAMING (inches)	PANELS APPLIED DIRECT TO FRAMING					PANELS APPLIED OVER 1/2" OR 5/8" GYPSUM SHEATHING				
			Staple length and gage~~size~~[h]	Fastener spacing at panel edges (inches)				Staple length and gage~~size~~[h]	Fastener spacing at panel edges (inches)			
				6	4	3	2[d]		6	4	3	2[d]
Structural I sheathing	3/8	1	1 1/2 16 Gage	155	235	315	400	2 16 Gage	155	235	310	400
	7/16			170	260	345	440		155	235	310	400
	15/32			185	280	375	475		155	235	300	400
Sheathing, plywood siding[e] except Group 5 Species, ANSI/APA PRP 210 siding[e]	5/16[c] or 1/4[c]	1	1 1/2 16 Gage	145	220	295	375	2 16 Gage	110	165	220	285
	3/8			140	210	280	360		140	210	280	360
	7/16			155	230	310	395		140	210	280	360
	15/32			170	255	335	430		140	210	280	360
	19/32		1 3/4 16 Gage	185	280	375	475	—	—	—	—	—

For SI: 1 inch = 25.4 mm, 1 pound per foot = 14.5939 N/m.

a. For framing of other species: (1) Find specific gravity for species of lumber in ANSI/AWC NDS. (2) For staples find shear value from table above for Structural I panels (regardless of actual grade) and multiply value by 0.82 for species with specific gravity of 0.42 or greater, or 0.65 for all other species.

b. Panel edges backed with 2-inch nominal or wider framing. Install panels either horizontally or vertically. Space fasteners maximum 6 inches on center along intermediate framing members for 3/8-inch and 7/16-inch panels installed on studs spaced 24 inches on center. For other conditions and panel thickness, space fasteners maximum 12 inches on center on intermediate supports.

c. 3/8-inch panel thickness or siding with a span rating of 16 inches on center is the minimum recommended where applied directly to framing as exterior siding. For grooved panel siding, the nominal panel thickness is the thickness of the panel measured at the point of fastening.

d. Framing at adjoining panel edges shall be 3 inches nominal or wider.

e. Values apply to all-veneer plywood. Thickness at point of fastening on panel edges governs shear values.

f. Where panels are applied on both faces of a wall and fastener spacing is less than 6 inches on center on either side, panel joints shall be offset to fall on different framing members, or framing shall be 3 inches nominal or thicker at adjoining panel edges.

g. In Seismic Design Category D, E or F, where shear design values exceed 350 pounds per linear foot, all framing members receiving edge fastening from abutting panels shall be not less than a single 3-inch nominal member, or two 2-inch nominal members fastened together in accordance with Section 2306.1 to transfer the design shear value between framing members. Wood structural panel joint and sill plate nailing shall be staggered at all panel edges. See AWC SDPWS for sill plate size and anchorage requirements.

h. Staples shall have a minimum crown width of $^{7}/_{16}$ inch and shall be installed with their crowns parallel to the long dimension of the framing members.

i. For shear loads of normal or permanent load duration as defined by the ANSI/AWC NDS, the values in the table above shall be multiplied by 0.63 or 0.56, respectively.

Reason: The proposal will change the heading of "Staple Size" to "Staple Length and Gage" to reflect that the column in the table also contains the required length of the staple. This change is consistent with the wording currently found in IBC Table 2306.2(1).
In addition, footnote e was added to ANSI APA PRP 210 siding to clarify that the siding shall be all-veneer, similar to the requirements for sheathing classified under DOC PS1 as well.

The intention is to clarify the code.

Cost Impact: Will not increase the cost of construction
The proposed change will not change the staple requirements for shear wall, and consequentially there should be no change in the cost of construction.

S285-16 : TABLE 2306.3-KERR13402

S286-16

IBC: 2306.3.

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

2015 International Building Code

Revise as follows:

TABLE 2306.3 (2)
ALLOWABLE SHEAR VALUES (plf) FOR WIND OR SEISMIC LOADING ON SHEAR WALLS OF FIBERBOARD SHEATHING BOARD CONSTRUCTION UTILIZING STAPLES FOR TYPE V CONSTRUCTION ONLY[a, b, c, d, e]

THICKNESS AND GRADE	FASTENER SIZE	ALLOWABLE SHEAR VALUE (pounds per linear foot) STAPLE SPACING AT PANEL EDGES (inches)[a]		
		4	3	2
$^1/_2$ ″ or $^{25}/_{32}$ ″ Structural	~~No.~~ 16 gage galvanized staple, $^7/_{16}$ ″ crown~~[f]~~ 1-3/4 inch long	150	200	225
	~~No.~~ 16 gage galvanized staple, 1″ crown~~[f]~~ 1-3/4 inch long	220	290	325

For SI: 1 inch = 25.4 mm, 1 pound per foot = 14.5939 N/m.

a. Fiberboard sheathing shall not be used to brace concrete or masonry walls.

b. Panel edges shall be backed with 2-inch or wider framing of Douglas Fir-larch or Southern Pine. For framing of other species: (1) Find specific gravity for species of framing lumber in ANSI/AWC NDS. (2) For staples, multiply the shear value from the table above by 0.82 for species with specific gravity of 0.42 or greater, or 0.65 for all other species.

c. Values shown are for fiberboard sheathing on one side only with long panel dimension either parallel or perpendicular to studs.

d. Fastener shall be spaced 6 inches on center along intermediate framing members.

e. Values are not permitted in Seismic Design Category D, E or F.

~~f. Staple length shall be not less than 1 $^1/_2$ inches for $^{25}/_{32}$ inch sheathing or 1 $^1/_4$ inches for $^1/_2$ inch sheathing.~~

Reason: A review of the test report referenced at the time staples were added to this table shows that 16 gage staples were used in testing and also that staple length for both sheathing thicknesses was 1-3/4". The 1-3/4" staple length is incorporated directly into the table in lieu of reference to footnote f.

Cost Impact: Will not increase the cost of construction
This proposal does not increase the cost of construction as it merely correlates and clarifies various requirements from standards.

S287-16

IBC: 2308.2.3.

Proponent : Edward Kulik, representing Building Code Action Committee (bcac@iccsafe.org)

2015 International Building Code

Revise as follows:

2308.2.3 Allowable loads. Loads shall be in accordance with Chapter 16 and shall not exceed the following:

1. Average dead loads shall not exceed 15 psf (718 N/m^2) for combined roof and ceiling, exterior walls, floors and partitions.

   Exceptions:

   1. Subject to the limitations of Section 2308.6.10, stone or masonry veneer up to the lesser of 5 inches (127 mm) thick or 50 psf (2395 N/m^2) and installed in accordance with Chapter 14 is permitted to a height of 30 feet (9144 mm) above a noncombustible foundation, with an additional 8 feet (2438 mm) permitted for gable ends.
   2. Concrete or masonry fireplaces, heaters and chimneys shall be permitted in accordance with the provisions of this code.

4. Live loads shall not exceed 40 psf (1916 N/m^2) for floors.

   <u>**Exception:** Live loads for concrete slab-on-ground floors in Risk Category I and II occupancies are not limited.</u>

5. Ground snow loads shall not exceed 50 psf (2395 N/m^2).

Reason: Conventional light-frame construction is often desirable to use for small slab-on-ground commercial structures. The restriction to a 40 pound per square foot live load is currently interpreted to apply to all levels of the structure, even at a ground floor space located on a concrete slab-on-ground. This proposal clarifies that live loads of more than 40 pounds per square foot are permitted at ground floors of Risk Category I and II buildings having a concrete slab-on-ground. This clarification is consistent with the very specific scope identified for the conventional light-frame construction in Section 2320.1 that go back to the 1997 UBC. Concrete slabs-on-ground design will be governed by applicable portions of Chapter 18 and Section 1907.
This proposal is submitted by the ICC Building Code Action Committee (BCAC). 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**

Cost Impact: Will not increase the cost of construction
This proposal will not increase the cost of construction as it simply allows a higher live load to be used where a concrete slab on grade is used at the ground floor level.

S287-16 : 2308.2-KULIK10976

S288-16

IBC: 2308.4.1.1, 2308.4.1.1(2) (New).

Proponent : David Tyree, representing American Wood Council (dtyree@awc.org)

2015 International Building Code

Revise as follows:

~~2308.4.1.1~~ (2)

~~HEADER AND GIRDER SPANS[a, b] FOR INTERIOR BEARING WALLS (Maximum spans for Douglas Fir-Larch, Hem-Fir, Southern Pine and Spruce-Pine-Fir[b] and required number of jack studs)~~

~~HEADERS AND GIRDERS SUPPORTING~~	~~SIZE~~	~~BUILDING WIDTH[c] (feet)~~					
		~~20~~		~~28~~		~~36~~	
		~~Span~~	~~NJ[d]~~	~~Span~~	~~NJ[d]~~	~~Span~~	~~NJ[d]~~
~~One floor only~~	~~2-2 × 4~~	~~3-1~~	~~1~~	~~2-8~~	~~1~~	~~2-5~~	~~1~~
	~~2-2 × 6~~	~~4-6~~	~~1~~	~~3-11~~	~~1~~	~~3-6~~	~~1~~
	~~2-2 × 8~~	~~5-9~~	~~1~~	~~5-0~~	~~2~~	~~4-5~~	~~2~~
	~~2-2 × 10~~	~~7-0~~	~~2~~	~~6-1~~	~~2~~	~~5-5~~	~~2~~
	~~2-2 × 12~~	~~8-1~~	~~2~~	~~7-0~~	~~2~~	~~6-3~~	~~2~~
	~~3-2 × 8~~	~~7-2~~	~~1~~	~~6-3~~	~~1~~	~~5-7~~	~~2~~
	~~3-2 × 10~~	~~8-9~~	~~1~~	~~7-7~~	~~2~~	~~6-9~~	~~2~~
	~~3-2 × 12~~	~~10-2~~	~~2~~	~~8-10~~	~~2~~	~~7-10~~	~~2~~
	~~4-2 × 8~~	~~9-0~~	~~1~~	~~7-8~~	~~1~~	~~6-9~~	~~1~~
	~~4-2 × 10~~	~~10-1~~	~~1~~	~~8-9~~	~~1~~	~~7-10~~	~~2~~
	~~4-2 × 12~~	~~11-9~~	~~1~~	~~10-2~~	~~2~~	~~9-1~~	~~2~~
	~~2-2 × 4~~	~~2-2~~	~~1~~	~~1-10~~	~~1~~	~~1-7~~	~~1~~

	~~2-2 × 6~~	~~3-2~~	~~2~~	~~2-9~~	~~2~~	~~2-5~~	~~2~~
	~~2-2 × 8~~	~~4-1~~	~~2~~	~~3-6~~	~~2~~	~~3-2~~	~~2~~
	~~2-2 × 10~~	~~4-11~~	~~2~~	~~4-3~~	~~2~~	~~3-10~~	~~3~~
	~~2-2 × 12~~	~~5-9~~	~~2~~	~~5-0~~	~~3~~	~~4-5~~	~~3~~
~~Two floors~~	~~3-2 × 8~~	~~5-1~~	~~2~~	~~4-5~~	~~2~~	~~3-11~~	~~2~~
	~~3-2 × 10~~	~~6-2~~	~~2~~	~~5-4~~	~~2~~	~~4-10~~	~~2~~
	~~3-2 × 12~~	~~7-2~~	~~2~~	~~6-3~~	~~2~~	~~5-7~~	~~3~~
	~~4-2 × 8~~	~~6-1~~	~~1~~	~~5-3~~	~~2~~	~~4-8~~	~~2~~
	~~4-2 × 10~~	~~7-2~~	~~2~~	~~6-2~~	~~2~~	~~5-6~~	~~2~~
	~~4-2 × 12~~	~~8-4~~	~~2~~	~~7-2~~	~~2~~	~~6-5~~	~~2~~

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

~~a. Spans are given in feet and inches.~~

~~b. Spans are based on minimum design properties for No. 2 grade lumber of Douglas Fir-Larch, Hem-Fir and Spruce-Pine-Fir. No. 1 or better grade lumber shall be used for Southern Pine.~~

~~c. Building width is measured perpendicular to the ridge. For widths between those shown, spans are permitted to be interpolated.~~

~~d. NJ - Number of jack studs required to support each end. Where the number of required jack studs equals one, the header is permitted to be supported by an approved framing anchor attached to the full-height wall stud and to the header.~~

TABLE 2308.4.1.1(2)

HEADER AND GIRDER SPANS[a,b] FOR INTERIOR BEARING WALLS (Maximum spans for Douglas Fir-Larch, Hem-Fir, Southern Pine, and Spruce-Pine-Fir and required number of jack studs)

HEADERS AND GIRDERS SUPPORTING	SIZE	BUILDING Width[c] (feet)					
		12		24		36	
		Span[e]	NJ[d]	Span[e]	NJ[d]	Span[e]	NJ[d]
One floor only	2-2x4	4 - 1	1	2 - 10	1	2 - 4	1
	2-2x6	6 - 1	1	4 - 4	1	3 - 6	1
	2-2x8	7 - 9	1	5 - 5	1	4 - 5	2

	2-2x10	9 - 2	1	6 - 6	2	5 - 3	2
	2-2x12	10 - 9	1	7 - 7	2	6 - 3	2
	3-2x8	9 - 8	1	6 - 10	1	5 - 7	1
	3-2x10	11 - 5	1	8 - 1	1	6 - 7	2
	3-2x12	13 - 6	1	9 - 6	2	7 - 9	2
	4-2x8	11 - 2	1	7 - 11	1	6 - 5	1
	4-2x10	13 - 3	1	9 - 4	1	7 - 8	1
	4-2x12	15 - 7	1	11 - 0	1	9 - 0	2
Two floors	2-2x4	2 - 7	1	1 - 11	1	1 - 7	1
	2-2x6	3 - 11	1	2 - 11	2	2 - 5	2
	2-2x8	5 - 0	1	3 - 8	2	3 - 1	2
	2-2x10	5 - 11	2	4 - 4	2	3 - 7	2
	2-2x12	6 - 11	2	5 - 2	2	4 - 3	3
	3-2x8	6 - 3	1	4 - 7	2	3 - 10	2
	3-2x10	7 - 5	1	5 - 6	2	4 - 6	2
	3-2x12	8 - 8	2	6 - 5	2	5 - 4	2
	4-2x8	7 - 2	1	5 - 4	1	4 - 5	2
	4-2x10	8 - 6	1	6 - 4	2	5 - 3	2
	4-2x12	10 - 1	1	7 - 5	2	6 - 2	2

For SI: 1 inch = 25.4 mm, 1 pound per square foot = 0.0479 kPa.

a. Spans are given in feet and inches.

b. Spans are based on minimum design properties for No. 2 grade lumber of Douglas Fir-Larch, Hem-Fir, Southern Pine, and Spruce-Pine-Fir.

c. Building width is measured perpendicular to the ridge. For widths between those shown, spans are permitted to be interpolated.

d. NJ - Number of jack studs required to support each end. Where the number of required jack studs equals one, the header is permitted to be supported by an approved framing anchor attached to the full-height wall stud and to the header.

e. Spans are calculated assuming the top of the header or girder is laterally braced by perpendicular framing. Where the top of the header or girder is not laterally braced (e.g. cripple studs bearing on the header), tabulated spans for headers consisting of 2x8, 2x10, or 2x12 sizes shall be multiplied by 0.70 or the header or girder shall be designed.

Reason: The update of Table 2308.4.1.1(2) Header and Girder Spans for Interior Bearing Walls is proposed. Updated spans address use of Southern Pine No. 2 in lieu of Southern Pine No. 1. Footnote "e" is added to clarify that header spans are based on laterally braced assumption such as w hen the header is raised. For dropped headers consisting of 2x8, 2x10, or 2x12 size framing and not laterally braced, a factor of 0.7 can be applied to determine the spans or alternatively the header or girder can be designed to include any adjustment for potential buckling. Laterally braced (raised) and not laterally braced (dropped) header conditions and building w idths for w hich header spans are tabulated represent the same conditions used to develop header span tables in the Wood Frame Construction Manual (WFCM).

Cost Impact: Will increase the cost of construction
Increased cost may be associated w ith reduced spans that result from the not laterally braced condition and application of footnote e. Due to smaller building w idth column (12'), permissible use of Southern Pine No. 2, and the laterally braced assumption for tabulated spans, there are also cases w here this change w ill not increase the cost of construction and may reduce cost of construction.

S288-16 : TABLE 2308.4.1.1-TYREE11413

S289-16

IBC: 2308.4.1.1, 2308.4.1.1(1) (New).

Proponent : David Tyree, representing American Wood Council (dtyree@awc.org)

2015 International Building Code

Revise as follows:

~~2308.4.1.1~~ (1)

~~HEADER AND GIRDER SPANS[a, b] FOR EXTERIOR BEARING WALLS (Maximum spans for Douglas Fir-Larch, Hem-Fir, Southern Pine and Spruce-Pine-Fir[b] and required number of jack studs)~~

~~GIRDERS AND HEADERS SUPPORTING~~	~~SIZE~~	~~GROUND SNOW LOAD (psf)[e]~~											
		~~30~~						~~50~~					
		~~Building width[c] (feet)~~											
		~~20~~		~~28~~		~~36~~		~~20~~		~~28~~		~~36~~	
		~~Span~~	~~NJ[d]~~	~~Span~~	~~NJ[d]~~	~~Span~~	~~NJ[d]~~	~~Span~~	~~NJ[d]~~	~~Span~~	~~NJ[d]~~	~~Span~~	~~NJ[d]~~
	~~2-2 × 4~~	~~3-6~~	~~1~~	~~3-2~~	~~1~~	~~2-10~~	~~1~~	~~3-2~~	~~1~~	~~2-9~~	~~1~~	~~2-6~~	~~1~~
	~~2-2 × 6~~	~~5-5~~	~~1~~	~~4-8~~	~~1~~	~~4-2~~	~~1~~	~~4-8~~	~~1~~	~~4-1~~	~~1~~	~~3-8~~	~~2~~
	~~2-2 × 8~~	~~6-10~~	~~1~~	~~5-11~~	~~2~~	~~5-4~~	~~2~~	~~5-11~~	~~2~~	~~5-2~~	~~2~~	~~4-7~~	~~2~~
	~~2-2 × 10~~	~~8-5~~	~~2~~	~~7-3~~	~~2~~	~~6-6~~	~~2~~	~~7-3~~	~~2~~	~~6-3~~	~~2~~	~~5-7~~	~~2~~
	~~2-2 × 12~~	~~9-9~~	~~2~~	~~8-5~~	~~2~~	~~7-6~~	~~2~~	~~8-5~~	~~2~~	~~7-3~~	~~2~~	~~6-6~~	~~2~~
~~Roof and ceiling~~	~~3-2 × 8~~	~~8-4~~	~~1~~	~~7-5~~	~~1~~	~~6-8~~	~~1~~	~~7-5~~	~~1~~	~~6-5~~	~~2~~	~~5-9~~	~~2~~
	~~3-2 × 10~~	~~10-6~~	~~1~~	~~9-1~~	~~2~~	~~8-2~~	~~2~~	~~9-1~~	~~2~~	~~7-10~~	~~2~~	~~7-0~~	~~2~~
	~~3-2 × 12~~	~~12-2~~	~~2~~	~~10-7~~	~~2~~	~~9-5~~	~~2~~	~~10-7~~	~~2~~	~~9-2~~	~~2~~	~~8-2~~	~~2~~
	~~4-2 × 8~~	~~9-2~~	~~1~~	~~8-4~~	~~1~~	~~7-8~~	~~1~~	~~8-4~~	~~1~~	~~7-5~~	~~1~~	~~6-8~~	~~1~~
	~~4-2 × 10~~	~~11-8~~	~~1~~	~~10-6~~	~~1~~	~~9-5~~	~~2~~	~~10-6~~	~~1~~	~~9-1~~	~~2~~	~~8-2~~	~~2~~
	~~4-2 × 12~~	~~14-1~~	~~1~~	~~12-2~~	~~2~~	~~10-11~~	~~2~~	~~12-2~~	~~2~~	~~10-7~~	~~2~~	~~9-5~~	~~2~~
	~~2-2 × 4~~	~~3-1~~	~~1~~	~~2-9~~	~~1~~	~~2-5~~	~~1~~	~~2-9~~	~~1~~	~~2-5~~	~~1~~	~~2-2~~	~~1~~
	~~2-2 × 6~~	~~4-6~~	~~1~~	~~4-0~~	~~1~~	~~3-7~~	~~2~~	~~4-1~~	~~1~~	~~3-7~~	~~2~~	~~3-3~~	~~2~~
	~~2-2 × 8~~	~~5-9~~	~~2~~	~~5-0~~	~~2~~	~~4-6~~	~~2~~	~~5-2~~	~~2~~	~~4-6~~	~~2~~	~~4-1~~	~~2~~
	~~2-2 × 10~~	~~7-0~~	~~2~~	~~6-2~~	~~2~~	~~5-6~~	~~2~~	~~6-4~~	~~2~~	~~5-6~~	~~2~~	~~5-0~~	~~2~~
~~Roof, ceiling and one center-bearing floor~~	~~2-2 × 12~~	~~8-1~~	~~2~~	~~7-1~~	~~2~~	~~6-5~~	~~2~~	~~7-4~~	~~2~~	~~6-5~~	~~2~~	~~5-9~~	~~3~~
	~~3-2 × 8~~	~~7-2~~	~~1~~	~~6-3~~	~~2~~	~~5-8~~	~~2~~	~~6-5~~	~~2~~	~~5-8~~	~~2~~	~~5-1~~	~~2~~

	~~3-2 × 10~~	~~8-9~~	~~2~~	~~7-8~~	~~2~~	~~6-11~~	~~2~~	~~7-11~~	~~2~~	~~6-11~~	~~2~~	~~6-3~~	~~2~~
	~~3-2 × 12~~	~~10-2~~	~~2~~	~~8-11~~	~~2~~	~~8-0~~	~~2~~	~~9-2~~	~~2~~	~~8-0~~	~~2~~	~~7-3~~	~~2~~
	~~4-2 × 8~~	~~8-1~~	~~1~~	~~7-3~~	~~1~~	~~6-7~~	~~1~~	~~7-5~~	~~1~~	~~6-6~~	~~1~~	~~5-11~~	~~2~~
	~~4-2 × 10~~	~~10-1~~	~~1~~	~~8-10~~	~~2~~	~~8-0~~	~~2~~	~~9-1~~	~~2~~	~~8-0~~	~~2~~	~~7-2~~	~~2~~
	~~4-2 × 12~~	~~11-9~~	~~2~~	~~10-3~~	~~2~~	~~9-3~~	~~2~~	~~10-7~~	~~2~~	~~9-3~~	~~2~~	~~8-4~~	~~2~~
~~Roof, ceiling and one clear span floor~~	~~2-2 × 4~~	~~2-8~~	~~1~~	~~2-4~~	~~1~~	~~2-1~~	~~1~~	~~2-7~~	~~1~~	~~2-3~~	~~1~~	~~2-0~~	~~1~~
	~~2-2 × 6~~	~~3-11~~	~~1~~	~~3-5~~	~~2~~	~~3-0~~	~~2~~	~~3-10~~	~~2~~	~~3-4~~	~~2~~	~~3-0~~	~~2~~
	~~2-2 × 8~~	~~5-0~~	~~2~~	~~4-4~~	~~2~~	~~3-10~~	~~2~~	~~4-10~~	~~2~~	~~4-2~~	~~2~~	~~3-9~~	~~2~~
	~~2-2 × 10~~	~~6-1~~	~~2~~	~~5-3~~	~~2~~	~~4-8~~	~~2~~	~~5-11~~	~~2~~	~~5-1~~	~~2~~	~~4-7~~	~~3~~
	~~2-2 × 12~~	~~7-1~~	~~2~~	~~6-1~~	~~3~~	~~5-5~~	~~3~~	~~6-10~~	~~2~~	~~5-11~~	~~3~~	~~5-4~~	~~3~~
	~~3-2 × 8~~	~~6-3~~	~~2~~	~~5-5~~	~~2~~	~~4-10~~	~~2~~	~~6-1~~	~~2~~	~~5-3~~	~~2~~	~~4-8~~	~~2~~
	~~3-2 × 10~~	~~7-7~~	~~2~~	~~6-7~~	~~2~~	~~5-11~~	~~2~~	~~7-5~~	~~2~~	~~6-5~~	~~2~~	~~5-9~~	~~2~~
	~~3-2 × 12~~	~~8-10~~	~~2~~	~~7-8~~	~~2~~	~~6-10~~	~~2~~	~~8-7~~	~~2~~	~~7-5~~	~~2~~	~~6-8~~	~~2~~
	~~4-2 × 8~~	~~7-2~~	~~1~~	~~6-3~~	~~2~~	~~5-7~~	~~2~~	~~7-0~~	~~1~~	~~6-1~~	~~2~~	~~5-5~~	~~2~~
	~~4-2 × 10~~	~~8-9~~	~~2~~	~~7-7~~	~~2~~	~~6-10~~	~~2~~	~~8-7~~	~~2~~	~~7-5~~	~~2~~	~~6-7~~	~~2~~
	~~4-2 × 12~~	~~10-2~~	~~2~~	~~8-10~~	~~2~~	~~7-11~~	~~2~~	~~9-11~~	~~2~~	~~8-7~~	~~2~~	~~7-8~~	~~2~~
~~Roof, ceiling and two center bearing floors~~	~~2-2 × 4~~	~~2-7~~	~~1~~	~~2-3~~	~~1~~	~~2-0~~	~~1~~	~~2-6~~	~~1~~	~~2-2~~	~~1~~	~~1-11~~	~~1~~
	~~2-2 × 6~~	~~3-9~~	~~2~~	~~3-3~~	~~2~~	~~2-11~~	~~2~~	~~3-8~~	~~2~~	~~3-2~~	~~2~~	~~2-10~~	~~2~~
	~~2-2 × 8~~	~~4-9~~	~~2~~	~~4-2~~	~~2~~	~~3-9~~	~~2~~	~~4-7~~	~~2~~	~~4-0~~	~~2~~	~~3-8~~	~~2~~
	~~2-2 × 10~~	~~5-9~~	~~2~~	~~5-1~~	~~2~~	~~4-7~~	~~3~~	~~5-8~~	~~2~~	~~4-11~~	~~2~~	~~4-5~~	~~3~~
	~~2-2 × 12~~	~~6-8~~	~~2~~	~~5-10~~	~~3~~	~~5-3~~	~~3~~	~~6-6~~	~~2~~	~~5-9~~	~~3~~	~~5-2~~	~~3~~
	~~3-2 × 8~~	~~5-11~~	~~2~~	~~5-2~~	~~2~~	~~4-8~~	~~2~~	~~5-9~~	~~2~~	~~5-1~~	~~2~~	~~4-7~~	~~2~~
	~~3-2 × 10~~	~~7-3~~	~~2~~	~~6-4~~	~~2~~	~~5-8~~	~~2~~	~~7-1~~	~~2~~	~~6-2~~	~~2~~	~~5-7~~	~~2~~
	~~3-2 × 12~~	~~8-5~~	~~2~~	~~7-4~~	~~2~~	~~6-7~~	~~2~~	~~8-2~~	~~2~~	~~7-2~~	~~2~~	~~6-5~~	~~3~~
	~~4-2 × 8~~	~~6-10~~	~~1~~	~~6-0~~	~~2~~	~~5-5~~	~~2~~	~~6-8~~	~~1~~	~~5-10~~	~~2~~	~~5-3~~	~~2~~
	~~4-2 × 10~~	~~8-4~~	~~2~~	~~7-4~~	~~2~~	~~6-7~~	~~2~~	~~8-2~~	~~2~~	~~7-2~~	~~2~~	~~6-5~~	~~2~~
	~~4-2 × 12~~	~~9-8~~	~~2~~	~~8-6~~	~~2~~	~~7-8~~	~~2~~	~~9-5~~	~~2~~	~~8-3~~	~~2~~	~~7-5~~	~~2~~
~~Roof, ceiling, and two clear span floors~~	~~2-2 × 4~~	~~2-1~~	~~1~~	~~1-8~~	~~1~~	~~1-6~~	~~2~~	~~2-0~~	~~1~~	~~1-8~~	~~1~~	~~1-5~~	~~2~~
	~~2-2 × 6~~	~~3-1~~	~~2~~	~~2-8~~	~~2~~	~~2-4~~	~~2~~	~~3-0~~	~~2~~	~~2-7~~	~~2~~	~~2-3~~	~~2~~
	~~2-2 × 8~~	~~3-10~~	~~2~~	~~3-4~~	~~2~~	~~3-0~~	~~3~~	~~3-10~~	~~2~~	~~3-4~~	~~2~~	~~2-11~~	~~3~~

~~GIRDERS AND HEADERS SUPPORTING~~	~~SIZE~~	~~GROUND SNOW LOAD (psf)~~[e]											
		~~30~~						~~50~~					
		~~Building width~~[e] ~~(feet)~~											
		~~20~~		~~28~~		~~36~~		~~20~~		~~28~~		~~36~~	
		~~Span~~	~~NJ~~[d]	~~Span~~	~~NJ~~[d]	~~Span~~	~~NJ~~[d]	~~Span~~	~~NJ~~[d]	~~Span~~	~~NJ~~[d]	~~Span~~	~~NJ~~[d]
~~Roof, ceiling, and two clear span floors~~	~~2-2 × 10~~	~~4-9~~	~~2~~	~~4-1~~	~~3~~	~~3-8~~	~~3~~	~~4-8~~	~~2~~	~~4-0~~	~~3~~	~~3-7~~	~~3~~
	~~2-2 × 12~~	~~5-6~~	~~3~~	~~4-9~~	~~3~~	~~4-3~~	~~3~~	~~5-5~~	~~3~~	~~4-8~~	~~3~~	~~4-2~~	~~3~~
	~~3-2 × 8~~	~~4-10~~	~~2~~	~~4-2~~	~~2~~	~~3-9~~	~~2~~	~~4-9~~	~~2~~	~~4-1~~	~~2~~	~~3-8~~	~~2~~
	~~3-2 × 10~~	~~5-11~~	~~2~~	~~5-1~~	~~2~~	~~4-7~~	~~3~~	~~5-10~~	~~2~~	~~5-0~~	~~2~~	~~4-6~~	~~3~~
	~~3-2 × 12~~	~~6-10~~	~~2~~	~~5-11~~	~~3~~	~~5-4~~	~~3~~	~~6-9~~	~~2~~	~~5-10~~	~~3~~	~~5-3~~	~~3~~
	~~4-2 × 8~~	~~5-7~~	~~2~~	~~4-10~~	~~2~~	~~4-4~~	~~2~~	~~5-6~~	~~2~~	~~4-9~~	~~2~~	~~4-3~~	~~2~~
	~~4-2 × 10~~	~~6-10~~	~~2~~	~~5-11~~	~~2~~	~~5-3~~	~~2~~	~~6-9~~	~~2~~	~~5-10~~	~~2~~	~~5-2~~	~~2~~
	~~4-2 × 12~~	~~7-11~~	~~2~~	~~6-10~~	~~2~~	~~6-2~~	~~3~~	~~7-9~~	~~2~~	~~6-9~~	~~2~~	~~6-0~~	~~3~~

~~For SI: 1 inch = 25.4 mm, 1 pound per square foot = 0.0479 kPa.~~

~~a. Spans are given in feet and inches.~~

~~b. Spans are based on minimum design properties for No. 2 grade lumber of Douglas Fir-Larch, Hem-Fir and Spruce-Pine-Fir. No. 1 or better grade lumber shall be used for Southern Pine.~~

~~c. Building width is measured perpendicular to the ridge. For widths between those shown, spans are permitted to be interpolated.~~

~~d. NJ - Number of jack studs required to support each end. Where the number of required jack studs equals one, the header is permitted to be supported by an approved framing anchor attached to the full-height wall stud and to the header.~~

~~e. Use 30 psf ground snow load for cases in which ground snow load is less than 30 psf and the roof live load is equal to or less than 20 psf.~~

TABLE 2308.4.1.1(1)

HEADER AND GIRDER SPANS[a,b] FOR EXTERIOR BEARING WALLS (Maximum spans for Douglas Fir-Larch, Hem-Fir, Southern Pine, and Spruce-Pine-Fir and required number of jack studs)

GIRDERS AND HEADERS SUPPORTING	SIZE	GROUND SNOW LOAD (psf)[e]																	
		30						50						70					
		Building width[c] (feet)																	
		12		24		36		12		24		36		12		24		36	
		Span[f]	NJ[d]	Span[f]	NJ[d]	Span[f]	NJ[d]	Span[f]	NJ[d]	Span[f]	NJ[d]	Span[f]	NJ[d]	Span[f]	NJ[d]	Span[f]	NJ[d]	Span[f]	NJ[d]
Roof and ceiling	1-2x6	4 - 0	1	3 - 1	2	2 - 7	2	3 - 5	1	2 - 8	2	2 - 3	2	3 - 0	2	2 - 4	2	2 - 0	2
	1-2x8	5 - 1	2	3 - 11	2	3 - 3	2	4 - 4	2	3 - 4	2	2 - 10	2	3 - 10	2	3 - 0	2	2 - 6	3
	1-2x10	6 - 0	2	4 - 8	2	3 - 11	2	5 - 2	2	4 - 0	2	3 - 4	3	4 - 7	2	3 - 6	3	3 - 0	3
	1-	7 - 1	2	5 - 5	2	4 - 7	3	6 - 1	2	4 - 8	3	3 - 11	3	5 - 5	2	4 - 2	3	3 - 6	3

	2x12																		
	2-2x4	4 - 0	1	3 - 1	1	2 - 7	1	3 - 5	1	2 - 7	1	2 - 2	1	3 - 0	1	2 - 4	1	2 - 0	1
	2-2x6	6 - 0	1	4 - 7	1	3 - 10	1	5 - 1	1	3 - 11	1	3 - 3	2	4 - 6	1	3 - 6	2	2 - 11	2
	2-2x8	7 - 7	1	5 - 9	1	4 - 10	2	6 - 5	1	5 - 0	2	4 - 2	2	5 - 9	1	4 - 5	2	3 - 9	2
	2-2x10	9 - 0	1	6 - 10	2	5 - 9	2	7 - 8	2	5 - 11	2	4 - 11	2	6 - 9	2	5 - 3	2	4 - 5	2
	2-2x12	10 - 7	2	8 - 1	2	6 - 10	2	9 - 0	2	6 - 11	2	5 - 10	2	8 - 0	2	6 - 2	2	5 - 2	3
	3-2x8	9 - 5	1	7 - 3	1	6 - 1	1	8 - 1	1	6 - 3	1	5 - 3	2	7 - 2	1	5 - 6	2	4 - 8	2
	3-2x10	11 - 3	1	8 - 7	1	7 - 3	2	9 - 7	1	7 - 4	2	6 - 2	2	8 - 6	1	6 - 7	2	5 - 6	2
	3-2x12	13 - 2	1	10 - 1	2	8 - 6	2	11 - 3	2	8 - 8	2	7 - 4	2	10 - 0	2	7 - 9	2	6 - 6	2
	4-2x8	10 - 11	1	8 - 4	1	7 - 0	1	9 - 4	1	7 - 2	1	6 - 0	1	8 - 3	1	6 - 4	1	5 - 4	2
	4-2x10	12 - 11	1	9 - 11	1	8 - 4	1	11 - 1	1	8 - 6	1	7 - 2	2	9 - 10	1	7 - 7	2	6 - 4	2
	4-2x12	15 - 3	1	11 - 8	1	9 - 10	2	13 - 0	1	10 - 0	2	8 - 5	2	11 - 7	1	8 - 11	2	7 - 6	2
Roof, ceiling and one center-bearing floor	1-2x6	3 - 3	1	2 - 7	2	2 - 2	2	3 - 0	2	2 - 4	2	2 - 0	2	2 - 9	2	2 - 2	2	1 - 10	2
	1-2x8	4 - 1	2	3 - 3	2	2 - 9	2	3 - 9	2	3 - 0	2	2 - 6	3	3 - 6	2	2 - 9	2	2 - 4	3
	1-2x10	4 - 11	2	3 - 10	2	3 - 3	3	4 - 6	2	3 - 6	3	3 - 0	3	4 - 1	2	3 - 3	3	2 - 9	3
	1-2x12	5 - 9	2	4 - 6	3	3 - 10	3	5 - 3	2	4 - 2	3	3 - 6	3	4 - 10	3	3 - 10	3	3 - 3	4
	2-2x4	3 - 3	1	2 - 6	1	2 - 2	1	3 - 0	1	2 - 4	1	2 - 0	1	2 - 8	1	2 - 2	1	1 - 10	1
	2-2x6	4 - 10	1	3 - 9	1	3 - 3	2	4 - 5	1	3 - 6	2	3 - 0	2	4 - 1	1	3 - 3	2	2 - 9	2
	2-2x8	6 - 1	1	4 - 10	2	4 - 1	2	5 - 7	2	4 - 5	2	3 - 9	2	5 - 2	2	4 - 1	2	3 - 6	2
	2-2x10	7 - 3	2	5 - 8	2	4 - 10	2	6 - 8	2	5 - 3	2	4 - 5	2	6 - 1	2	4 - 10	2	4 - 1	2
	2-2x12	8 - 6	2	6 - 8	2	5 - 8	2	7 - 10	2	6 - 2	2	5 - 3	3	7 - 2	2	5 - 8	2	4 - 10	3
	3-2x8	7 - 8	1	6 - 0	1	5 - 1	2	7 - 0	1	5 - 6	2	4 - 8	2	6 - 5	1	5 - 1	2	4 - 4	2

	3-2x10	9 - 1	1	7 - 2	2	6 - 1	2	8 - 4	1	6 - 7	2	5 - 7	2	7 - 8	2	6 - 1	2	5 - 2	2
	3-2x12	10 - 8	2	8 - 5	2	7 - 2	2	9 - 10	2	7 - 8	2	6 - 7	2	9 - 0	2	7 - 1	2	6 - 1	2
	4-2x8	8 - 10	1	6 - 11	1	5 - 11	1	8 - 1	1	6 - 4	1	5 - 5	2	7 - 5	1	5 - 11	1	5 - 0	2
	4-2x10	10 - 6	1	8 - 3	2	7 - 0	2	9 - 8	1	7 - 7	2	6 - 5	2	8 - 10	1	7 - 0	2	6 - 0	2
	4-2x12	12 - 4	1	9 - 8	2	8 - 3	2	11 - 4	2	8 - 11	2	7 - 7	2	10 - 4	2	8 - 3	2	7 - 0	2
Roof, ceiling and one clear span floor	1-2x6	2 - 11	2	2 - 3	2	1 - 11	2	2 - 9	2	2 - 1	2	1 - 9	2	2 - 7	2	2 - 0	2	1 - 8	2
	1-2x8	3 - 9	2	2 - 10	2	2 - 5	3	3 - 6	2	2 - 8	2	2 - 3	3	3 - 3	2	2 - 6	3	2 - 2	3
	1-2x10	4 - 5	2	3 - 5	3	2 - 10	3	4 - 2	2	3 - 2	3	2 - 8	3	3 - 11	2	3 - 0	3	2 - 6	3
	1-2x12	5 - 2	2	4 - 0	3	3 - 4	3	4 - 10	3	3 - 9	3	3 - 2	4	4 - 7	3	3 - 6	3	3 - 0	4
	2-2x4	2 - 11	1	2 - 3	1	1 - 10	1	2 - 9	1	2 - 1	1	1 - 9	1	2 - 7	1	2 - 0	1	1 - 8	1
	2-2x6	4 - 4	1	3 - 4	2	2 - 10	2	4 - 1	1	3 - 2	2	2 - 8	2	3 - 10	1	3 - 0	2	2 - 6	2
	2-2x8	5 - 6	2	4 - 3	2	3 - 7	2	5 - 2	2	4 - 0	2	3 - 4	2	4 - 10	2	3 - 9	2	3 - 2	2
	2-2x10	6 - 7	2	5 - 0	2	4 - 2	2	6 - 1	2	4 - 9	2	4 - 0	2	5 - 9	2	4 - 5	2	3 - 9	3
	2-2x12	7 - 9	2	5 - 11	2	4 - 11	3	7 - 2	2	5 - 7	2	4 - 8	3	6 - 9	2	5 - 3	3	4 - 5	3
	3-2x8	6 - 11	1	5 - 3	2	4 - 5	2	6 - 5	1	5 - 0	2	4 - 2	2	6 - 1	1	4 - 8	2	4 - 0	2
	3-2x10	8 - 3	2	6 - 3	2	5 - 3	2	7 - 8	2	5 - 11	2	5 - 0	2	7 - 3	2	5 - 7	2	4 - 8	2
	3-2x12	9 - 8	2	7 - 5	2	6 - 2	2	9 - 0	2	7 - 0	2	5 - 10	2	8 - 6	2	6 - 7	2	5 - 6	3
	4-2x8	8 - 0	1	6 - 1	1	5 - 1	2	7 - 5	1	5 - 9	2	4 - 10	2	7 - 0	1	5 - 5	2	4 - 7	2
	4-2x10	9 - 6	1	7 - 3	2	6 - 1	2	8 - 10	1	6 - 10	2	5 - 9	2	8 - 4	1	6 - 5	2	5 - 5	2
	4-2x12	11 - 2	2	8 - 6	2	7 - 2	2	10 - 5	2	8 - 0	2	6 - 9	2	9 - 10	2	7 - 7	2	6 - 5	2
Roof, ceiling and two center-	1-2x6	2 - 8	2	2 - 1	2	1 - 10	2	2 - 7	2	2 - 0	2	1 - 9	2	2 - 5	2	1 - 11	2	1 - 8	2

bearing floors	1-2x8	3 - 5	2	2 - 8	2	2 - 4	3	3 - 3	2	2 - 7	2	2 - 2	3	3 - 1	2	2 - 5	3	2 - 1	3
	1-2x10	4 - 0	2	3 - 2	3	2 - 9	3	3 - 10	2	3 - 1	3	2 - 7	3	3 - 8	2	2 - 11	3	2 - 5	3
	1-2x12	4 - 9	3	3 - 9	3	3 - 2	4	4 - 6	3	3 - 7	3	3 - 1	4	4 - 3	3	3 - 5	3	2 - 11	4
	2-2x4	2 - 8	1	2 - 1	1	1 - 9	1	2 - 6	1	2 - 0	1	1 - 8	1	2 - 5	1	1 - 11	1	1 - 7	1
	2-2x6	4 - 0	1	3 - 2	2	2 - 8	2	3 - 9	1	3 - 0	2	2 - 7	2	3 - 7	1	2 - 10	2	2 - 5	2
	2-2x8	5 - 0	2	4 - 0	2	3 - 5	2	4 - 10	2	3 - 10	2	3 - 3	2	4 - 7	2	3 - 7	2	3 - 1	2
	2-2x10	6 - 0	2	4 - 9	2	4 - 0	2	5 - 8	2	4 - 6	2	3 - 10	3	5 - 5	2	4 - 3	2	3 - 8	3
	2-2x12	7 - 0	2	5 - 7	2	4 - 9	3	6 - 8	2	5 - 4	3	4 - 6	3	6 - 4	2	5 - 0	3	4 - 3	3
	3-2x8	6 - 4	1	5 - 0	2	4 - 3	2	6 - 0	1	4 - 9	2	4 - 1	2	5 - 8	2	4 - 6	2	3 - 10	2
	3-2x10	7 - 6	2	5 - 11	2	5 - 1	2	7 - 1	2	5 - 8	2	4 - 10	2	6 - 9	2	5 - 4	2	4 - 7	2
	3-2x12	8 - 10	2	7 - 0	2	5 - 11	2	8 - 5	2	6 - 8	2	5 - 8	3	8 - 0	2	6 - 4	2	5 - 4	3
	4-2x8	7 - 3	1	5 - 9	1	4 - 11	2	6 - 11	1	5 - 6	2	4 - 8	2	6 - 7	1	5 - 2	2	4 - 5	2
	4-2x10	8 - 8	1	6 - 10	2	5 - 10	2	8 - 3	2	6 - 6	2	5 - 7	2	7 - 10	2	6 - 2	2	5 - 3	2
	4-2x12	10 - 2	2	8 - 1	2	6 - 10	2	9 - 8	2	7 - 8	2	6 - 7	2	9 - 2	2	7 - 3	2	6 - 2	2
Roof, ceiling and two clear span floors	1-2x6	2 - 3	2	1 - 9	2	1 - 5	2	2 - 3	2	1 - 9	2	1 - 5	3	2 - 2	2	1 - 8	2	1 - 5	3
	1-2x8	2 - 10	2	2 - 2	3	1 - 10	3	2 - 10	2	2 - 2	3	1 - 10	3	2 - 9	2	2 - 1	3	1 - 10	3
	1-2x10	3 - 4	2	2 - 7	3	2 - 2	3	3 - 4	3	2 - 7	3	2 - 2	4	3 - 3	3	2 - 6	3	2 - 2	4
	1-2x12	4 - 0	3	3 - 0	3	2 - 7	4	4 - 0	3	3 - 0	4	2 - 7	4	3 - 10	3	3 - 0	4	2 - 6	4
	2-2x4	2 - 3	1	1 - 8	1	1 - 4	1	2 - 3	1	1 - 8	1	1 - 4	1	2 - 2	1	1 - 8	1	1 - 4	2
	2-2x6	3 - 4	1	2 - 6	2	2 - 2	2	3 - 4	2	2 - 6	2	2 - 2	2	3 - 3	2	2 - 6	2	2 - 1	2
	2-2x8	4 - 3	2	3 - 3	2	2 - 8	2	4 - 3	2	3 - 3	2	2 - 8	2	4 - 1	2	3 - 2	2	2 - 8	3
	2-2x10	5 - 0	2	3 - 10	2	3 - 2	3	5 - 0	2	3 - 10	2	3 - 2	3	4 - 10	2	3 - 9	3	3 - 2	3

	2-2x12	5 - 11	2	4 - 6	3	3 - 9	3	5 - 11	2	4 - 6	3	3 - 9	3	5 - 8	2	4 - 5	3	3 - 9	3
	3-2x8	5 - 3	1	4 - 0	2	3 - 5	2	5 - 3	2	4 - 0	2	3 - 5	2	5 - 1	2	3 - 11	2	3 - 4	2
	3-2x10	6 - 3	2	4 - 9	2	4 - 0	2	6 - 3	2	4 - 9	2	4 - 0	2	6 - 1	2	4 - 8	2	4 - 0	3
	3-2x12	7 - 5	2	5 - 8	2	4 - 9	3	7 - 5	2	5 - 8	2	4 - 9	3	7 - 2	2	5 - 6	3	4 - 8	3
	4-2x8	6 - 1	1	4 - 8	2	3 - 11	2	6 - 1	1	4 - 8	2	3 - 11	2	5 - 11	1	4 - 7	2	3 - 10	2
	4-2x10	7 - 3	2	5 - 6	2	4 - 8	2	7 - 3	2	5 - 6	2	4 - 8	2	7 - 0	2	5 - 5	2	4 - 7	2
	4-2x12	8 - 6	2	6 - 6	2	5 - 6	2	8 - 6	2	6 - 6	2	5 - 6	2	8 - 3	2	6 - 4	2	5 - 4	3

For SI: 1 inch = 25.4 mm, 1 pound per square foot = 0.0479 kPa.

a. Spans are given in feet and inches.

b. Spans are based on minimum design properties for No. 2 grade lumber of Douglas Fir-Larch, Hem-Fir, Southern Pine, and Spruce-Pine-Fir.

c. Building width is measured perpendicular to the ridge. For widths between those shown, spans are permitted to be interpolated.

d. NJ - Number of jack studs required to support each end. Where the number of required jack studs equals one, the header is permitted to be supported by an approved framing anchor attached to the full-height wall stud and to the header.

e. Use 30 psf ground snow load for cases in which ground snow load is less than 30 psf and the roof live load is equal to or less than 20 psf.

f. Spans are calculated assuming the top of the header or girder is laterally braced by perpendicular framing. Where the top of the header or girder is not laterally braced (e.g. cripple studs bearing on the header), tabulated spans for headers consisting of 2x8, 2x10, or 2x12 sizes shall be multiplied by 0.70 or the header or girder shall be designed.

Reason: The update of Table 2308.4.1.1(1) Header and Girder Spans for Exterior Bearing Walls is proposed. Updated spans address use of Southern Pine No. 2 in lieu of Southern Pine No. 1. Footnote "f" is added to clarify that header spans are based on laterally braced assumption such as when the header is raised. For dropped headers consisting of 2x8, 2x10, or 2x12 sizes that are not laterally braced, a factor of 0.7 can be applied to determine the spans or alternatively the header or girder can be designed to include any adjustment for potential buckling. Laterally braced (raised) and not laterally braced (dropped) header conditions and building widths for which header spans are tabulated represent the same condtions used to develop header span tables in the Wood Frame Construction Manual (WFCM).

Cost Impact: Will increase the cost of construction
Increased cost may be associated with reduced spans that result from the not laterally braced condition and application of footnote f. Due to smaller building width column (12'), permissible use of Southern Pine No. 2, and the laterally braced assumption for tabulated spans, there are also cases where this change will not increase the cost of construction and may reduce cost of construction.

S289-16 : TABLE 2308.4.1.1-TYREE12519

S290-16

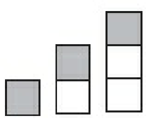
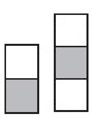
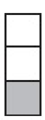
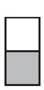
IBC: 2308.6.1.

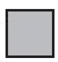
Proponent : Larry Wainright, representing the Structural Building Components Association, representing Structural Building Components Association (lwainright@qualtim.com)

2015 International Building Code

Revise as follows:

TABLE 2308.6.1
[a] WALL BRACING REQUIREMENTS

SEISMIC DESIGN CATEGORY	STORY CONDITION (SEE SECTION 2308.2)	MAXIMUM SPACING OF BRACED WALL LINES	BRACED PANEL LOCATION, SPACING (O.C.) AND MINIMUM PERCENTAGE (X)			MAXIMUM DISTANCE OF BRACED WALL PANELS FROM EACH END OF BRACED WALL LINE
			Bracing method[b]			
			LIB	DWB, WSP[c]	SFB, PBS, PCP, HPS, GB,~~c~~,d,e	
A and B		35′- 0″	Each end and ≤ 25′- 0″ o.c.	Each end and ≤ 25′- 0″ o.c.	Each end and ≤ 25′- 0″ o.c.	12′- 6″
		35′- 0″	Each end and ≤ 25′- 0″ o.c.	Each end and ≤ 25′- 0″ o.c.	Each end and ≤ 25′- 0″ o.c.	12′- 6″
		35′- 0″	NP	Each end and ≤ 25′- 0″ o.c.	Each end and ≤ 25′- 0″ o.c.	12′- 6″
C		35′- 0″	NP	Each end and ≤ 25′- 0″ o.c.	Each end and ≤ 25′- 0″ o.c.	12′- 6″
		35′- 0″	NP	Each end and ≤ 25′- 0″ o.c. (minimum 25% of wall	Each end and ≤ 25′- 0″ o.c. (minimum 25% of wall	12′- 6″

				length)[~~e~~f]	length)[~~e~~f]	
D and E		25′- 0″	NP	S_{DS} of wall length)[~~e~~f]	S_{DS} 25′- 0″ o.c. (minimum 43% of wall length)[~~e~~f]	8′- 0″
				0.5 ≤ S_{DS} and ≤ 25′- 0″ o.c. (minimum 32% of wall length)[~~e~~f]	0.5 ≤ S_{DS} and ≤ 25′- 0″ o.c. (minimum 59% of wall length)[~~e~~f]	
				0.75 ≤ S_{DS} ≤ 1.00: Each end and ≤ 25′- 0″ o.c. (minimum 37% of wall length)[~~e~~f]	0.75 ≤ S_{DS} ≤ 1.00: Each end and ≤ 25′- 0″ o.c. (minimum 75% of wall length)	
				S_{DS} > 1.00: Each end and ≤25′- 0″ o.c. (minimum 48% of wall length)[~~e~~f]	S_{DS} > 1.00: Each end and ≤ 25′- 0″ o.c. (minimum 100% of wall length)[~~e~~f]	

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

NP = Not Permitted.

a. This table specifies minimum requirements for braced wall panels along interior or exterior braced wall lines.

b. See Section 2308.6.3 for full description of bracing methods.

c. For Method WSP, braced wall panel lengths were developed using the following: 3/8" WSP using a 6d galvanized box nail applied to Douglas Fir framing with a nail spacing of 6 inches along edges and 12 inches in the field and having a nominal unit shear capacity for seismic zones of 400 plf.

~~c~~ d. — For Method GB, gypsum wallboard applied to framing supports that are spaced at 16 inches on center.

~~d~~ e. The required lengths shall be doubled for gypsum board applied to only one face of a braced wall panel.

~~e~~ f. Percentage shown represents the minimum amount of bracing required along the building length ~~(~~or wall length ~~if~~ where the structure has an irregular shape~~)~~.

Reason: To clarify the basis of the design values that were used for the predominate braced wall panel material in the market, which is WSP.

Table 2308.6.1 shows the required braced panel location, spacing and minimum percentages. This engineering foundation for this information is not transparent but can be found via the design values published in AWC, *Wind*

and Seismic, Special Design Provisions for Wind and Seismic (SPDWS). Rather than having to go to another document to get this fundamental design value information, this footnote is proposed to enhance the usefulness of the IBC to those not intimately familiar w ith SPDWS.

This code change is intended simply to alert users of the code to the basis of Table 2308.6.1 for WSP allow ing them to make an informed decision w ith respect to WSP braced w all panel design in the context of lateral design of a building.

Cost Impact: Will not increase the cost of construction
This is simply a footnote added for transparency and clarity as to the basis of the Table.

S290-16 : TABLE 2308.6.1-WAINRIGHT13327

S291-16

IBC: 2308.3.1, 2308.3.1.2 (New), 2308.3.2.

Proponent : Edward Kulik, representing Building Code Action Committee (bcac@iccsafe.org)

2015 International Building Code

Revise as follows:

2308.3.1 Foundation plates or sills. Foundation plates or sills resting on concrete or masonry foundations shall comply with Section 2304.3.1. Foundation plates or sills shall be bolted or anchored to the foundation with not less than $^{1}/_{2}$-inch-diameter (12.7 mm) steel bolts or *approved* anchors spaced to provide equivalent anchorage as the steel bolts. Bolts shall be embedded at least 7 inches (178 mm) into concrete or masonry. The bolts shall be located in the middle third of the width of the plate. Bolts shall be spaced not more than 6 feet (1829 mm) on center and there shall be not less than two bolts or anchor straps per piece with one bolt or anchor strap located not more than 12 inches (305 mm) or less than 4 inches (102 mm) from each end of each piece. Bolts in sill plates of braced wall lines in structures over two stories above grade shall be spaced not more than 4 feet (1219 mm) on center. A properly sized nut and washer shall be tightened on each bolt to the plate.

> ~~**Exceptions:**~~
>
> ~~1. Along *braced wall lines* in structures assigned to *Seismic Design Category* E, steel bolts with a minimum nominal diameter of $^{5}/_{8}$ inch (15.9 mm) or approved anchor straps load-rated in accordance with Section 2304.10.3 and spaced to provide equivalent anchorage shall be used.~~
>
> ~~2. Bolts in *braced wall lines* in structures over two stories above grade shall be spaced not more than 4 feet (1219 mm) on center.~~

~~2308.3.2~~ 2308.3.1.1 Braced wall line sill plate anchorage in Seismic Design ~~Categories~~ Category D ~~and E~~. Sill plates along *braced wall lines* in buildings assigned to *Seismic Design Category* D ~~or E~~ shall be anchored with not less than 1/2 inch diameter (12.7 mm) anchor bolts with steel plate washers between the foundation sill plate and the nut, or approved anchor straps load-rated in accordance with Section 2304.10.3 and spaced to provide equivalent anghorage. ~~Such~~ Plate washers shall be a minimum of 0.229 inch by 3 inches by 3 inches (5.82 mm by 76 mm by 76 mm) in size. The hole in the plate washer is permitted to be diagonally slotted with a width of up to $^{3}/_{16}$ inch (4.76 mm) larger than the bolt diameter and a slot length not to exceed $1^{3}/_{4}$ inches (44 mm), provided a standard cut washer is placed between the plate washer and the nut.

Add new text as follows:

2308.3.1.2 Braced wall line sill plate anchorage in Seismic Design Category E. Sill plates along braced wall lines in buildings assigned to Seismic Design Category E shall be anchored with not less than 5/8 inch diameter (15.9 mm) anchor bolts with steel plate washers between the foundation sill plate and the nut, or approved anchor straps load-rated in accordance with Section 2304.10.3 and spaced to provide equivalent anchorage. Plate washers shall be a minimum of 0.229 inch by 3 inches by 3 inches (5.82 mm by 76 mm by 76 mm) in size. The hole in the plate washer is permitted to be diagonally slotted with a width of up to 3/16-inch (4.76 mm) larger than the bolt diameter and a slot length not to exceed 1-3/4 inches (44 mm), provided a standard cut washer is placed between the plate washer and the nut.

Reason: In the course of reviewing S273-12 during the previous code cycle, the editorial reorganization of Section 2308, two inconsistencies in the foundation anchorage section were identified, however at the time the BCAC did not want to make any changes that could be perceived as technical changes. The primary purpose of this proposal is to bring these changes forward.
The first change relocates Exception #2 under Section 2308.3.1, the 4 foot anchor bolt spacing requirement for three-story structures, into the base provision moves from. This move is in keeping with a general philosophy that exceptions should be relaxations to a base provision, not more stringent. The actual implementation of the bolt spacing requirement is not changed.

The second change creates a new subsection specific to anchorage requirements for Seismic Design Category E. The requirements from the existing paragraph on Seismic Design Category D and E are copied, and the language from Exception #1 under Section 2308.3.1 is moved to the new paragraph. At the same time, the existing seismic section is changed so that it only applies to Seismic Design Category D. Both sections will now appear as subsections of the basic foundation sill anchorage requirements.

In addition, the language successfully added last cycle by proposal RB219-13 to Section R403.1.6 of the IRC is brought over to this corresponding section in the IBC. The intent of this language is to clarify the location of the anchor bolts relative to the middle third of the plate. The requirement insures there is adequate distance from the bolts to the edge of the plate such that the anchor bolts can achieve their anticipated capacity without causing the plate to fail in shear parallel to the grain. It is noted that buildings constructed under the conventional construction provisions of Section 2308 do not rely on the use of high-load shear walls and do not typically use wall plates larger than 2x6. Thus, it is not necessary to locate the bolts such that the edge of the plate washers in high-seismic categories is within 1/2 inch of the sheathed edge(s) nor is it necessary to stagger the bolts in such plates.

This proposal is submitted by the ICC Building Code Action Committee (BCAC). 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**

Cost Impact: Will not increase the cost of construction
This proposal will not increase the cost of construction as it is intended to be an editorial reorganization and needed clarification on the anchor bolt placement.

S291-16 : 2308.3.1-KULIK10975

S292-16

IBC: 2308.5.5.1, 2308.5.5.1(1) (New), 2308.5.5.1(2) (New).

Proponent : Paul Coats, PE CBO, representing American Wood Council (pcoats@awc.org)

2015 International Building Code

Revise as follows:

2308.5.5.1 Openings in exterior bearing walls. Headers shall be provided over each opening in exterior bearing walls. The size and spans in Table 2308.4.1.1(1) are permitted to be used for one- and two-family *dwellings*. Headers for other buildings shall be designed in accordance with Section 2301.2, Item 1 or 2. Headers ~~shall be~~ of two or more pieces of nominal 2-inch (51 mm) framing lumber set on edge ~~as~~ shall be permitted ~~by~~ in accordance with Table 2308.4.1.1(1) and nailed together in accordance with Table 2304.10.1 or of solid lumber of equivalent size.

Single member headers of nominal 2-inch thickness shall be framed with a single flat 2-inch-nominal (51 mm) member or wall plate not less in width than the wall studs on the top and bottom of the header in accordance with Figures 2308.5.5.1(1) and 2308.5.5.1(2) and face nailed to the top and bottom of the header with 10d box nails (3 inches × 0.128 inches) spaced 12 inches on center.

Wall studs shall support the ends of the header in accordance with Table 2308.4.1.1(1). Each end of a lintel or header shall have a bearing length of not less than $1^1/_2$ inches (38 mm) for the full width of the lintel.

Add new text as follows:

FIGURE 2308.5.5.1(1)
Single Member Header in Exterior Bearing Wall

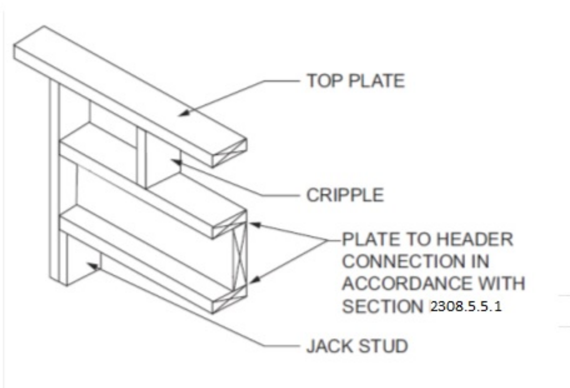
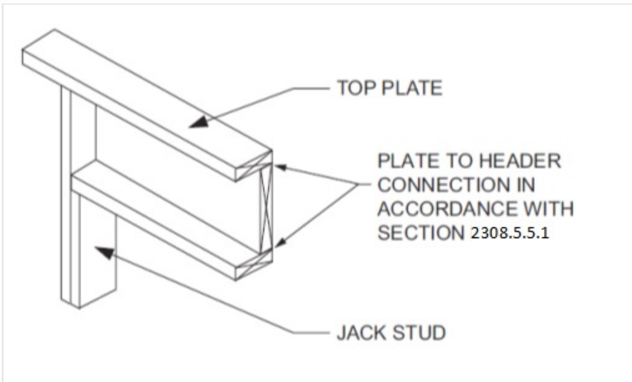


FIGURE 2308.5.5.1(2)

Alternative Single Member Header Without Cripple



Reason: This propsoal adds prescriptive framing and connection requirements for single member (single ply) headers consistent with the IRC. Additionally, provisions of 2308.5.5.1 are revised to coordinate with tabulated header sizes consisting of 2, 3, or 4 member headers.

Cost Impact: Will not increase the cost of construction
This change adds a more efficient single member header option in some cases and will not raise the cost of construction.

S292-16 : 2308.5.5.1-COATS13384

S293-16

Part I:
IBC: 2308.6.5.1, 2308.6.5.2.
Part II:
IRC: R602.10.3, R602.10.6.1.

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC-STRUCTURAL CODE COMMITTEE. PART II WILL BE HEARD BY THE IRC-BUILDING CODE COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

Proponent : Larry Wainright, representing the Structural Building Components Association, representing Structural Building Components Association (lwainright@qualtim.com)

Part I

2015 International Building Code

Revise as follows:

2308.6.5.1 Alternate braced wall (ABW). An ABW shall be constructed in accordance with this section and Figure 2308.6.5.1. In one-story buildings, each panel shall have a length of not less than 2 feet 8 inches (813 mm) and a height of not more than 10 feet (3048 mm). Each panel shall be sheathed on one face with $^3/_8$-inch (3.2 mm) minimum-thickness wood structural panel sheathing nailed with 8d common or galvanized box nails in accordance with Table 2304.10.1 and blocked at wood structural panel edges. Two anchor bolts installed in accordance with Section 2308.3.1 shall be provided in each panel. Anchor bolts shall be placed at each panel outside quarter points. Each panel end stud shall have a hold-down device fastened to the foundation, capable of providing an *approved* uplift capacity of not less than 1,800 pounds (180 plf times a 10 foot wall height) (8006 N). The hold-down device shall be installed in accordance with the manufacturer's recommendations. The ABW shall be supported directly on a foundation or on floor framing supported directly on a foundation that is continuous across the entire length of the *braced wall line*. This foundation shall be reinforced with not less than one No. 4 bar top and bottom. Where the continuous foundation is required to have a depth greater than 12 inches (305 mm), a minimum 12-inch by 12-inch (305 mm by 305 mm) continuous footing or turned-down slab edge is permitted at door openings in the *braced wall line*. This continuous footing or turned-down slab edge shall be reinforced with not less than one No. 4 bar top and bottom. This reinforcement shall be lapped 15 inches (381 mm) with the reinforcement required in the continuous foundation located directly under the *braced wall line*.

Where the ABW is installed at the first *story* of two-story buildings, the wood structural panel sheathing shall be provided on both faces, three anchor bolts shall be placed at one-quarter points and tie-down device uplift capacity shall be not less than 3,000 pounds (300 plf times a 10 foot wall height) (13 344 N).

Add new text as follows:

2308.6.5.2 Portal frame with hold-downs (PFH). A PFH shall be constructed in accordance with this section and Figure 2308.6.5.2. The adjacent door or window opening shall have a full-length header.

In one-story buildings, each panel shall have a length of not less than 16 inches (406 mm) and a height of not more than 10 feet (3048 mm). Each panel shall be sheathed on one face with a

single layer of $^3/_8$-inch (9.5 mm) minimum-thickness wood structural panel sheathing nailed with 8d common or galvanized box nails in accordance with Figure 2308.6.5.2. The wood structural panel sheathing shall extend up over the solid sawn or glued-laminated header and shall be nailed in accordance with Figure 2308.6.5.2. A built-up header consisting of at least two 2-inch by 12-inch (51 mm by 305 mm) boards, fastened in accordance with Item 24 of Table 2304.10.1 shall be permitted to be used. A spacer, if used, shall be placed on the side of the built-up beam opposite the wood structural panel sheathing. The header shall extend between the inside faces of the first full-length outer studs of each panel. The clear span of the header between the inner studs of each panel shall be not less than 6 feet (1829 mm) and not more than 18 feet (5486 mm) in length. A strap with an uplift capacity of not less than 1,000 pounds (4,400 N) shall fasten the header to the inner studs opposite the sheathing. One anchor bolt not less than $^5/_8$ inch (15.9 mm) diameter and installed in accordance with Section 2308.3.1 shall be provided in the center of each sill plate. The studs at each end of the panel shall have a hold-down device fastened to the foundation with an uplift capacity of not less than 3,500 pounds (350 plf times a 10 foot wall height) (15 570 N).

Where a panel is located on one side of the opening, the header shall extend between the inside face of the first full-length stud of the panel and the bearing studs at the other end of the opening. A strap with an uplift capacity of not less than 1,000 pounds (4400 N) shall fasten the header to the bearing studs. The bearing studs shall also have a hold-down device fastened to the foundation with an uplift capacity of not less than 1,000 pounds (4400 N). The hold-down devices shall be an embedded strap type, installed in accordance with the manufacturer's recommendations. The PFH panels shall be supported directly on a foundation that is continuous across the entire length of the braced wall line. This foundation shall be reinforced with not less than one No. 4 bar top and bottom. Where the continuous foundation is required to have a depth greater than 12 inches (305 mm), a minimum 12-inch by 12-inch (305 mm by 305 mm) continuous footing or turned-down slab edge is permitted at door openings in the braced wall line. This continuous footing or turned-down slab edge shall be reinforced with not less than one No. 4 bar top and bottom. This reinforcement shall be lapped not less than 15 inches (381 mm) with the reinforcement required in the continuous foundation located directly under the braced wall line.

Where a PFH is installed at the first ~~story~~*story* of two-story buildings, each panel shall have a length of not less than 24 inches (610 mm).

Part II

2015 International Residential Code

Revise as follows:

TABLE R602.10.6.1
MINIMUM HOLD-DOWN FORCES FOR METHOD ABW BRACED WALL PANELS

SEISMIC DESIGN CATEGORY AND WIND SPEED	SUPPORTING/STORY	HOLD-DOWN FORCE (pounds)				
		Height of Braced Wall Panel				
		8 feet	9 feet	10 feet	11 feet	12 feet

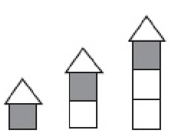
SDC A, B and C Ultimate design wind speed	One story	1,800	1,800	1,800	2,000	2,200
	First of two stories	3,000	3,000	3,000	3,300	3,600
SDC D_0 , D_1 and D_2Ultimate design wind speed	One story	1,800	1,800	1,800	NP	NP
	First of two stories	3,000	3,000	3,000	NP	NP

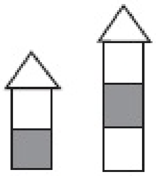
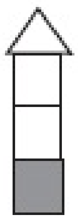
For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound = 4.45 N, 1 mile per hour = 0.447 m/s.

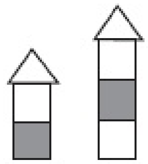
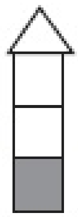
NP = Not Permitted.

1. Hold-down force is determined as the plf capacity of the braced wall panel times a minimum 10 ft wall height.

TABLE R602.10.3 (1)
BRACING REQUIREMENTS BASED ON WIND SPEED

· EXPOSURE CATEGORY B · 30-FOOT MEAN ROOF HEIGHT · 10-FOOT WALL HEIGHT · 2 BRACED WALL LINES			MINIMUM TOTAL LENGTH (FEET) OF BRACED WALL PANELS REQUIRED ALONG EACH BRACED WALL LINE[a]			
Ultimate Design Wind Speed (mph)	**Story Location**	**Braced Wall Line Spacing (feet)**	**Method LIB[b]**	**Method GB**	**Methods DWB, WSP, SFB, PBS, PCP, HPS, BV-WSP, ABW, PFH, PFC, CS-SFB[c]**	**Methods CS-WSP, CS-G, CS-PF**
		10	3.5	3.5	2.0	1.5
		20	6.0	6.0	3.5	3.0
		30	8.5	8.5	5.0	4.5
		40	11.5	11.5	6.5	5.5
		50	14.0	14.0	8.0	7.0
		60	16.5	16.5	9.5	8.0
		10	6.5	6.5	3.5	3.0

≤110		20	11.5	11.5	6.5	5.5
		30	16.5	16.5	9.5	8.0
		40	21.5	21.5	12.5	10.5
		50	26.5	26.5	15.5	13.0
		60	31.5	31.5	18.0	15.5
		10	NP	9.5	5.5	4.5
		20	NP	17.0	10.0	8.5
		30	NP	24.5	14.0	12.0
		40	NP	32.0	18.5	15.5
		50	NP	39.5	22.5	19.0
		60	NP	46.5	26.5	23.0
≤ 115		10	3.5	3.5	2.0	2.0
		20	6.5	6.5	3.5	3.5
		30	9.5	9.5	5.5	4.5
		40	12.5	12.5	7.0	6.0
		50	15.0	15.0	9.0	7.5
		60	18.0	18.0	10.5	9.0
		10	7.0	7.0	4.0	3.5
		20	12.5	12.5	7.5	6.5
		30	18.0	18.0	10.5	9.0
		40	23.5	23.5	13.5	11.5

		50	29.0	29.0	16.5	14.0
		60	34.5	34.5	20.0	17.0
		10	NP	10.0	6.0	5.0
		20	NP	18.5	11.0	9.0
		30	NP	27.0	15.5	13.0
		40	NP	35.0	20.0	17.0
		50	NP	43.0	24.5	21.0
		60	NP	51.0	29.0	25.0

· EXPOSURE CATEGORY B · 30-FOOT MEAN ROOF HEIGHT · 10-FOOT WALL HEIGHT · 2 BRACED WALL LINES			MINIMUM TOTAL LENGTH (FEET) OF BRACED WALL PANELS REQUIRED ALONG EACH BRACED WALL LINE[a]			
Ultimate Design Wind Speed (mph)	**Story Location**	**Braced Wall Line Spacing (feet)**	**Method LIB[b]**	**Method GB**	**Methods DWB, WSP, SFB, PBS, PCP, HPS, BV-WSP, ABW, PFH, PFG, CS-SFB[c]**	**Methods CS-WSP, CS-G, CS-PF**
		10	4.0	4.0	2.5	2.0
		20	7.0	7.0	4.0	3.5
		30	10.5	10.5	6.0	5.0
		40	13.5	13.5	8.0	6.5
		50	16.5	16.5	9.5	8.0

		60	19.5	19.5	11.5	9.5
≤ 120	✗	10	7.5	7.5	4.5	3.5
		20	14.0	14.0	8.0	7.0
		30	20.0	20.0	11.5	9.5
		40	25.5	25.5	15.0	12.5
		50	31.5	31.5	18.0	15.5
		60	37.5	37.5	21.5	18.5
	✗	10	NP	11.0	6.5	5.5
		20	NP	20.5	11.5	10.0
		30	NP	29.0	17.0	14.5
		40	NP	38.0	22.0	18.5
		50	NP	47.0	27.0	23.0
		60	NP	55.5	32.0	27.0
	✗	10	4.5	4.5	2.5	2.5
		20	8.5	8.5	5.0	4.0
		30	12.0	12.0	7.0	6.0
		40	15.5	15.5	9.0	7.5
		50	19.5	19.5	11.0	9.5
		60	23.0	23.0	13.0	11.0
		10	8.5	8.5	5.0	4.5
		20	16.0	16.0	9.5	8.0

≤130		30	23.0	23.0	13.5	11.5
		40	30.0	30.0	17.5	15.0
		50	37.0	37.0	21.5	18.0
		60	44.0	44.0	25.0	21.5
		10	NP	13.0	7.5	6.5
		20	NP	24.0	13.5	11.5
		30	NP	34.5	19.5	17.0
		40	NP	44.5	25.5	22.0
		50	NP	55.0	31.5	26.5
		60	NP	65.0	37.5	31.5

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 mile per hour = 0.447 m/s.

a. Linear interpolation shall be permitted.

b. Method LIB shall have gypsum board fastened to not less than one side with nails or screws in accordance with Table R602.3(1) for exterior sheathing or Table R702.3.5 for interior gypsum board. Spacing of fasteners at panel edges shall not exceed 8 inches.

c. Where a braced wall line has parallel braced wall lines on one or both sides of differing dimensions, the average dimension shall be permitted to be used for braced wall line spacing.

d. Braced wall length requirements for intermittently braced Method WSP walls are determined using 500 PLF for the wood structural panels plus 200 PLF for the gypsum sheathing times a net adjustment factor of 1.2.

Reason: To clarify the basis of hold-down requirements and bracing length requirements for those not intimately familiar with lateral load and uplift calculation concepts. This code change is intended simply to provide users of the code with information that is generally used to make uplift calculations from lateral loads and provide the basis of calculation used to develop the wall bracing lengths provided in the Table .

Cost Impact: Will not increase the cost of construction
This proposal changes not technical requirements and thus does not increase the cost of construction.

S293-16 : 2308.6.5.1-WAINRIGHT13368

S294-16

IBC: 1406.3, 1410.1, 2407.1; IFC: 804.1.

Proponent : Maureen Traxler, City of Seattle Dept of Construction & Inspections, representing City of Seattle Dept of Construction & Inspections (maureen.traxler@seattle.gov)

2015 International Building Code

Revise as follows:

2407.1 Materials. Glass used in a handrail~~, guardrail~~ or~~ a~~ *guard*~~ section~~ shall be laminated glass constructed of fully tempered or heat-strengthened glass and shall comply with Category II or CPSC 16 CFR Part 1201 or Class A of ANSI Z97.1. Glazing in railing in-fill panels shall be of an *approved* safety glazing material that conforms to the provisions of Section 2406.1.1. For all glazing types, the minimum nominal thickness shall be $^1/_4$ inch (6.4 mm).

> **Exception:** Single fully tempered glass complying with Category II of CPSC 16 CFR Part 1201 or Class A of ANSI Z97.1 shall be permitted to be used in handrails and ~~guardrails~~ guards where there is no walking surface beneath them or the walking surface is permanently protected from the risk of falling glass.

1406.3 Balconies and similar projections. Balconies and similar projections of combustible construction other than fire-retardant-treated wood shall be fire-resistance rated where required by Table 601 for floor construction or shall be of Type IV construction in accordance with Section 602.4. The aggregate length of the projections shall not exceed 50 percent of the building's perimeter on each floor.

Exceptions:

1. On buildings of Type I and II construction, three stories or less above *grade plane*, *fire-retardant-treated wood* shall be permitted for balconies, porches, decks and exterior stairways not used as required exits.
2. Untreated wood is permitted for pickets, rails and ~~rails or~~ similar ~~guardrail devices~~ guard components that are limited to 42 inches (1067 mm) in height.
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.
4. Where sprinkler protection is extended to the balcony areas, the aggregate length of the balcony on each floor shall not be limited.

1410.1 Plastic composite decking. Exterior deck boards, stair treads, handrails and ~~guardrail systems~~ guards constructed of plastic composites, including plastic lumber, shall comply with Section 2612.

2015 International Fire Code

Revise as follows:

804.1 Interior trim. Material, other than foam plastic, used as interior trim in new and existing buildings shall have minimum Class C flame spread and smoke-developed indices, when tested in accordance with ASTM E 84 or UL 723, as described in Section 803.1.1. Combustible trim, excluding handrails and ~~guardrails~~ guards, shall not exceed 10 percent of the specific wall or ceiling areas to which it is attached.

Reason: This proposal changes the term "guardrail" to "guard" in several code sections. "Guard" is defined in the IBC and should be used consistently throughout the codes. The term is defined as "a building component or a system of building components located at or near the open sides of elevated walking surfaces that minimizes the possibility of a fall from the walking surface to a lower level." This definition is appropriate for each of the code sections addressed in this proposal.
There is one other use of the term "guardrail," in IFC Section 2306.7.9.2.2.2, regarding physical protection for vapor-processing equipment. Because the requirement is not related to fall protection, we determined that "guard" as defined in the IBC is not the appropriate term, and we did not include that section in this proposal.

Cost Impact: Will not increase the cost of construction
This proposal will have no effect on the cost of construction. It changes the term "guardrail" to "guard" in several code sections.

S294-16 : 2407.1-TRAXLER7936

S295-16

IBC: 2407.1.1, 2407.1.2.

Proponent : Lee Kranz, City of Bellevue, WA, representing Washington Association of Building Officials Technical Code Development Committee (lkranz@bellevuewa.gov)

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 factor of safety of four~~ shall be used for safety~~.

2407.1.2 ~~Support~~ Structural glass baluster panels. ~~Each handrail~~
Guards with structural glass baluster panels shall be installed with an attached top rail or ~~*guard* section~~ handrail. The top rail or handrail shall be supported by a minimum of three glass ~~balusters~~ baluster panels, or shall be otherwise supported to remain in place should one glass baluster panel fail.~~ Glass balusters shall not be installed without an attached handrail or *guard*.~~

> **Exception:**~~A~~ An attached top rail ~~shall~~ or handrail is not~~ be~~ required where the glass ~~balusters~~ baluster panels are laminated glass with two or more glass plies of equal thickness and of the same glass type~~ when *approved* by the *building official*. The panels shall be designed to withstand the loads specified in Section 1607.8.~~

Reason: This proposal w ill clarify code requirements for glass panels that are used as a structural component in a guard. Imperfections in glass can cause the panel to fail at loads that are w ell below its nominal resistance value. We believe the intent of the IBC requirements is to have something (a top rail or a handrail at stairs) to provide some additional fall protection for a person leaning on the guard, should a glass panel fail. Having a handrail attached to at least 3 panels also provides some backup support if a panel fails w hile someone is grabbing the handrail to prevent a fall. How ever, there is an exception that allow s glass-only guards (w ithout an attached top rail or handrail) if the balusters are laminated glass. The laminated glass provides some backup against total panel failure, but note that the entire glass baluster still has to be designed to be able to support the full loads for guards, as specified in Section 2407.1.1, including using a factor of safety of 4.
The change in Section 2407.1.1 is proposed because "factor of safety" is a term that is understood by the engineers w ho w ill be doing the design. "Design factor...for safety" has no meaning.

The requirement regarding design loading in the original exception to Section 2407.1.2 has not been carried forw ard, because the requirement is covered in Section 2407.1.1.

The restriction on use of the exception to "w hen approved by the building official" has been deleted because there is no guidance or criteria as to w hen it w ould or w ould not be approved.

Cost Impact: Will not increase the cost of construction
This change creates consistency w ith the IRC for glass guards only and allow s for more safety and flexibility in design. There should be no increase in the cost.

S295-16 : 2407.1.1-KRANZ11181

S296-16

IBC: 2407.1.1.

Proponent : Jonathan Siu, City of Seattle Department of Construction & Inspections, representing Washington Association of Building Officials Technical Code Development Committee (jon.siu@seattle.gov)

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

S297-16

IBC: 2407.1.2.

Proponent : Tom Zaremba, Roetzel & Andress, representing Glazing Industry Code Commitee (tzaremba@ralaw.com)

2015 International Building Code

Revise as follows:

2407.1.2 Support. Each handrail or *guard* section shall be supported by a minimum of three glass balusters or shall be otherwise supported to remain in place should one baluster panel fail. Glass balusters shall not be installed without an attached handrail or *guard*.

> **Exception:**A top rail shall not be required where the glass balusters are laminated glass with two or more glass plies of equal thickness and the same glass type~~ when *approved* by the *building official*~~. The panels shall be designed to withstand the loads specified in Section 1607.8<u> and shall be tested to remain in place as a barrier following impact or glass breakage in accordance with ASTM E2353</u>.

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
<u>ASTM 2353-14 Standard Test Methods for Performance of Glazing in Permanent Railing Systems, Guards and Balustrades</u>

Reason: Currently, the Code exempts glass balusters from having a top rail, but only if approved by the building official. This proposal deletes the requirement that these assemblies be approved by the building official, but adds the requirement that they be tested to ASTM E2353-14. ASTM E2353-14 w as developed to test the ability of glazing materials in these types of assemblies to remain in place as a barrier after impact or glass breakage. Testing glass baluster systems that have no top rails in accordance w ith this standard w ill ensure that they stay in place as a barrier after impact or glass breakage w hile eliminating the need for the building code official to evaluate them on a case by case basis.

Cost Impact: Will increase the cost of construction
The additional testing to ASTM E2353-14 may increase the cost of construction, but w ill be mitigated by eliminating the case by case approval of the building code official that is now required.

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM E2353, w ith regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) w ill be posted on the ICC w ebsite on or before April 1, 2016.

S297-16 : 2407.1.2-ZAREMBA12666

S298-16

IBC: 2506.2.

Proponent : Mike Fischer, Kellen Company (mfischer@kellencompany.com)

2015 International Building Code

Revise as follows:

TABLE 2506.2 (2506.2)
GYPSUM BOARD AND GYPSUM PANEL PRODUCTS MATERIALS AND ACCESSORIES

MATERIAL	STANDARD
Accessories for gypsum board	ASTM C1047
Adhesives for fastening gypsum board	ASTM C557
Cold-formed steel studs and track, structural	AISI S200 and ASTM C 955, Section 8
Cold-formed steel studs and track, nonstructural	AISI S220 and ASTM C 645, Section 10
Elastomeric joint sealants	ASTM C 920
<u>Factory-laminated gypsum panel products</u>	<u>ASTM C 1766</u>
Fiber-reinforced gypsum panels	ASTM C 1278
Glass mat gypsum backing panel	ASTM C 1178
Glass mat gypsum panel 5	ASTM C 1658
Glass mat gypsum substrate	ASTM C 1177
Joint reinforcing tape and compound	ASTM C 474; C 475
Nails for gypsum boards	ASTM C 514, F 547, F 1667
Steel screws	ASTM C 954; C 1002
Standard specification for gypsum board	ASTM C 1396
Testing gypsum and gypsum products	ASTM C 22; C 472; C 473

Reference standards type:

Add new standard(s) as follows:

ASTM C 1766-13 Standard Specification for Factory-Laminated Gypsum Panel Products

Reason: ASTM C1766 was developed by ASTM subcommittee C11.01, assigned the responsibility for the development and maintenance of test methods and materials for gypsum products. Standard C 1766 addresses gypsum panel products, laminated in the facory, that are designed for use in sound control (in ceilings, walls, partitions etc.) or for gypsum studs or coreboards. Adding the standard to Table 2506.2 will help ensure that the latest available information and product standards for these panels are appropriately applied.

Cost Impact: Will not increase the cost of construction
The proposal adds in a product standard that extends performance requirements for factory-laminated products to meet the current intent of the code.

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM C1766, 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.

S298-16 : TABLE 2506.2-FISCHER5349

S299-16

IBC: 2506.2, 2507.2.

Proponent : Bonnie Manley, AISI, representing American Iron and Steel Institute (bmanley@steel.org)

2015 International Building Code

Revise as follows:

TABLE 2506.2
GYPSUM BOARD AND GYPSUM PANEL PRODUCTS MATERIALS AND ACCESSORIES

MATERIAL	STANDARD
Accessories for gypsum board	ASTM C1047
Adhesives for fastening gypsum board	ASTM C557
Cold-formed steel studs and track, structural	AISI S240~~S200 and ASTM C 955, Section 8~~
Cold-formed steel studs and track, nonstructural	AISI S220~~ and ASTM C 645, Section 10~~
Elastomeric joint sealants	ASTM C 920
Fiber-reinforced gypsum panels	ASTM C 1278
Glass mat gypsum backing panel	ASTM C 1178
Glass mat gypsum panel 5	ASTM C 1658
Glass mat gypsum substrate	ASTM C 1177
Joint reinforcing tape and compound	ASTM C 474; C 475
Nails for gypsum boards	ASTM C 514, F 547, F 1667
Steel screws	ASTM C 954; C 1002
Standard specification for gypsum board	ASTM C 1396
Testing gypsum and gypsum products	ASTM C 22; C 472; C 473

TABLE 2507.2
LATH, PLASTERING MATERIALS AND ACCESSORIES

MATERIAL	STANDARD

Accessories for gypsum veneer base	ASTM C1047
Blended cement	ASTM C595
Exterior plaster bonding compounds	ASTM C932
Cold-formed steel studs and track, structural	AISI S240~~S200 and ASTM C 955, Section 8~~
Cold-formed steel studs and track, nonstructural	AISI S220~~ and ASTM C 645, Section 10~~
Hydraulic cement	ASTM C 1157; C 1600
Gypsum casting and molding plaster	ASTM C 59
Gypsum Keene's cement	ASTM C 61
Gypsum plaster	ASTM C 28
Gypsum veneer plaster	ASTM C 587
Interior bonding compounds, gypsum	ASTM C 631
Lime plasters	ASTM C 5; C 206
Masonry cement	ASTM C 91
Metal lath	ASTM C 847
Plaster aggregates Sand Perlite Vermiculite	ASTM C 35; C 897 ASTM C 35 ASTM C 35
Plastic cement	ASTM C 1328
Portland cement	ASTM C 150
Steel screws	ASTM C 1002; C 954
Welded wire lath	ASTM C 933

Woven wire plaster base	ASTM C 1032

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-~~11~~15, *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 $\beta o = 1.6$" 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.

S299-16 : TABLE 2506.2-MANLEY12122

S300-16

Part I:

IBC: 2506.2, 2508.4 (New).

Part II:

IRC: R702.3.1, R702.3.1.1 (New).

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC-BUILDING & ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

Proponent : Mike Fischer, representing The Center for the Polyurethanes Industry of the American Chemistry Council (mfischer@kellencompany.com)

Part I

2015 International Building Code

Revise as follows:

TABLE 2506.2
GYPSUM BOARD AND GYPSUM PANEL PRODUCTS MATERIALS AND ACCESSORIES

MATERIAL	STANDARD
Accessories for gypsum board	ASTM C1047
Adhesives for fastening gypsum board	ASTM C557
Expandable foam adhesives for fastening gypsum wallboard	ASTM D6464
Cold-formed steel studs and track, structural	AISI S200 and ASTM C 955, Section 8
Cold-formed steel studs and track, nonstructural	AISI S220 and ASTM C 645, Section 10
Elastomeric joint sealants	ASTM C 920
Fiber-reinforced gypsum panels	ASTM C 1278
Glass mat gypsum backing panel	ASTM C 1178
Glass mat gypsum panel 5	ASTM C 1658
Glass mat gypsum substrate	ASTM C 1177
Joint reinforcing tape and compound	ASTM C 474; C 475
Nails for gypsum boards	ASTM C 514, F 547, F 1667

Steel screws	ASTM C 954; C 1002
Standard specification for gypsum board	ASTM C 1396
Testing gypsum and gypsum products	ASTM C 22; C 472; C 473

Add new text as follows:

2508.4 Adhesives Gypsum board and gypsum panel products secured to framing with adhesives in ceiling assemblies shall be attached using an *approved* fastening schedule. Expandable foam adhesives for fastening gypsum wallboard shall conform to ASTM D6464. All other adhesives for the installation of gypsum wallboard shall conform to ASTM C557.

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
ASTM D 6464-03a(2009)e1 Standard Specification for Expandable Foam Adhesives for Fastening Gypsum Wallboard to Wood Framing

Part II

2015 International Residential Code

Revise as follows:

R702.3.1 Materials. Gypsum board and gypsum panel product materials and accessories shall conform to ASTM C 22, C 475, C 514, C 1002, C 1047, C 1177, C 1178, C 1278, C 1396 or C 1658 and shall be installed in accordance with the provisions of this section.~~ Adhesives for the installation of gypsum board and gypsum panel products shall conform to ASTM C 557.~~

Add new text as follows:

R702.3.1.1 Adhesives Adhesives for the installation of gypsum board and gypsum panel products shall conform to ASTM C 557. All other adhesives for the installation of gypsum board and gypsum panel products shall conform to ASTM C557. Supports and fasteners used to attach gypsum board and gypsum panel products shall comply with Table R702.3.5 or other approved method.

Reference standards type: This reference standard is new to the ICC Code Books
Add new standard(s) as follows:
ASTM D 6464-03a(2009)e1 Standard Specification for Expandable Foam Adhesives for Fastening Gypsum Wallboard to Wood Framing

Reason:

Part I: This proposal adds ASTM D6464 to Table 2506.2, and adds a mandatory statement outlining the requirements for adhesives used to attach gypsum board products. The new referenced standard applies to expandable foam adhesives, which are currently accepted for use by product evaluation reports. Adding the mandatory scoping provision makes it clear that there will be two separate adhesive standards referenced, and that they apply to different types of products. More importantly, the proposal adds a requirement calling for approved fastening methods for gypsum products used in ceiling assemblies, which is consistent with the fastening requirements for gypsum products used in ceiling assemblies under the IRC.

Part II: This proposal adds a new referenced standard, ASTM D 6464, which applies to expandable foam adhesives used with gypsum products. The code today refers only to ASTM C 557 for adhesives used with gypsum board, but not all adhesives are included in the scope of ASTM C 557. The new referenced standard applies only to expandable foam adhesives, which are currently qualified for use by product evaluation reports. Adding the mandatory scoping provision makes it clear that there will be two separate adhesive standards referenced, and that they apply to different types of products, and adds a mandatory statement outlining the requirements for adhesives used to attach gypsum board products.
Additionally, the proposal adds a pointer to Table R702.3.5 or requires approved fastening methods for gypsum products using adhesives. Table R702.3.5 includes important provisions for attachment methods with and without adhesives; the pointer calls attention to the need to consider proper fastening.

Cost Impact: Will not increase the cost of construction
The proposal increases product selection options, but contains no mandatory requirements.

S300-16 : 2508.4 (New)-FISCHER5391

S301-16

IBC: 2510.6, 2510.6.1 (New).

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~~ comply with Section ~~1405.4) intended to drain to the *water-resistive barrier* is directed between the layers~~ 2510.6.1.

> ~~**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.~~

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 1, 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 secion 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 specificadtion 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 preferrable 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.

S301-16 : 2510.6-CRANDELL12539

S302-16

IBC: 2510.6.

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:**
> ~~**Exception:**~~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, 3A, 4A, 5A, and 4C in accordance with Chapter 3 of the *International Energy Conseration 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.

Bibliography: ABTG (2015). "Assessment of Water Vapor Control Methods for Modern Insulated Light-Frame Wall Assemblies", Research Report No. ABTG-1410-03, Applied Building Technology Group, LLC, http://www.appliedbuildingtech.com/rr/1410-03
BSC (2005). Healthy and Affordable Housing: Practical Recommendations for Building, Renovating and Maintaining Housing. Prepared for the U.S. Department of Housing and Urban Development and Building America Program of the U.S. Department of Energy, Building Science Corporation, www.buildingscience.com

Derome, D. (2010). The nature, significance and control of solar-driven water vapor diffusion in wall systems -- synthesis of Research Project RP-1235, ASHRAE Transactions, January 2010. www.ashrae.org

Lepage, R. and Lstiburek, J. (2013). Moisture Durability ith Vapor-Permeable Insulating Sheathing, U.S. DOE, Building Technologies Office, www.osti.gov/bridge

Wilkinson, J. Ueno, K., DeRose, D., Straube, J.F. and Fugler, D. (2007). Understanding Vapour Permeance and Condensation in Wall Assemblies, *11th Canadian Conference on Building Science and Technology*, Banff, Alberta

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.

S302-16 : 2510.6-CRANDELL12547

S303-16

IBC: 2510.6.

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 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 w ith 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 costruction w ill not increase.

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM E2925, w ith regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) w ill be posted on the ICC w ebsite on or before April 1, 2016.

S303-16 : 2510.6-DALGLEISH13099

S304-16

IBC: 2510.6.

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 an intervening, substantially nonwater-absorbing layer or drainage space.~~ :
> a. ASTM E 2556, Type II and is separated from the stucco by an intervening, substantially nonwater-absorbing layer or drainage space, or
> b. ASTM E2556, Type I and is seperated 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 Manufcatured Polymeric Drainage and Ventilation Materials used to Provide a Rainscreen Function

Reason: Materials complying w ith 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 w ill not increase. Using materials that comply w ith ASTM E2925 allow s you to use a Type I w ater resistive barrier w hich is not as costly as a Type II.

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM E2925, w ith regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) w ill be posted on the ICC w ebsite on or before April 1, 2016.

S304-16 : 2510.6-DALGLEISH13115

S305-16

IBC: 2510.6.

Proponent : Theresa Weston, representing DuPont Building Innovations (theresa.a.weston@dupont.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~~ one 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.~~

following,

1. 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 is directed between the layers
2. 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. Flashing installed in accordance with Section 1405.4 is directed between the intervening layer and water-resistive barrier or into the drainage space.
3. Other approved methods

Reason: This proposal provides clarity by reorganizing this section to show the w ater-resistive barrier requirements as options rather than as a single requirement and an exception. There is no change in the requirements. The reorganization is useful because a larger portion of the industry is opting for systems that currently fall under the exception.

Cost Impact: Will not increase the cost of construction
This proposal does not change the requirements in the code.

S305-16 : 2510.6-WESTON13013

S306-16

IBC: G101.5 (New), G103.1, G103.2, G103.3, G103.4, G103.5, G103.6, G103.6.1, G103.7, G103.8, G103.9, G104.1, G104.2, G104.3, G104.4, G104.5, G105.2, G105.7.

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 hazardareas* established in Section G102.2.
2. Where a proposed development site is in a *flood hazardarea*, 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 r*egistered 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*~~*building official's*~~ *'s* designee shall make, or cause to be made, inspections of all development in *flood hazardareas* 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 m^2), 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 responsibile 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," w hich 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, w hich may or may not be the builidng 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.

S306-16 : G101.5 (NEW)-WILSON12067

S307-16

IBC: G103.6.

Proponent : Gregory Wilson, representing Federal Emergency Management Agency (gregory.wilson2@fema.dhs.gov); Rebecca Quinn, representing Federal Emergency Management Agency (rcquinn@earthlink.net)

2015 International Building Code

Revise as follows:

G103.6 Watercourse alteration. Prior to issuing a *permit* for any alteration or relocation of any watercourse, the *building official* 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.

Reason: The National Flood Insurance Program regulations specify that communities notify adjacent communities w hen a proposal to alter or relocate a w atercourse is received. When a local jurisdiction uses IBC Appendix G, the current phrasing in Section G103.6 requires judgment to determine w hether an adjacent jurisdiction is or is not affected by a proposed w atercourse alteration. Only w ith engineering analyses is it feasible to determine w hether adjacent communities are affected. Instead, this proposal requires notification of all adjacent communities.

Cost Impact: Will not increase the cost of construction
There is no cost impact to construction because this proposal is administrative.

S307-16 : G103.6-WILSON12069

S308-16

IBC: G103.10 (New).

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 aboundaries 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 w ith 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 ow n 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 w ith 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 show n on the FIRM, decisions regarding future permit requirements and NFIP flood insurance policies w ould not be based on the best available information. The requirement to submit new studies w ithin 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.

S308-16 : G103.10 (NEW)-WILSON12071

S309-16

IBC: G103.8.

Proponent : Gregory Wilson (gregory.wilson2@fema.dhs.gov); Rebecca Quinn, representing Federal Emergency Management Agency (rcquinn@earthlink.net)

2015 International Building Code

Revise as follows:

G103.8 Records. The *building official* shall maintain a permanent record of all *permits* issued in *flood hazard areas*, including supporting certifications and documentation required by this appendix and copies of inspection reports, design certifications and ~~certifications~~ documentation of elevations required in Section 1612 of the I*nternational Building Code* and Section R322 of the I*nternational Residential Code*.

Reason: Communities that participate in the National Flood Insurance Program agree to obtain and maintain certain certifications and documentation in their permanent records and make them available for inspection. Required certifications and documentation for buildings and structures are identified in the IBC and IRC. When a community uses IBC Appendix G, Appendix G also identifies documentation to support permit decisions that should be maintained, including floodw ay encroachment analyses and analyses of the flood-carrying capacity of altered or relocated w atercourses.

Cost Impact: Will not increase the cost of construction
NFIP communities are already required to maintain these documents.

S309-16 : G103.8-WILSON12072

S310-16

IBC: G601.2, G601.3.

Proponent : Gregory Wilson, Federal Emergency Management Agency (gregory.wilson2@fema.dhs.gov)

2015 International Building Code

Revise as follows:

G601.2 Temporary placement. Recreational vehicles in *flood hazard areas* shall be fully licensed and ready for highway use, ~~and~~ or shall be placed on a site for less than 180 consecutive days.

G601.3 Permanent placement. Recreational vehicles that are not fully licensed and ready for highway use, or that are to be placed on a site for more than 180 consecutive days, shall meet the requirements of Section G501 for manufactured homes.

Reason: This proposal is editorial for internal consistency. The NFIP regulations for placement of recreational vehicles in flood hazard areas [44 Code of Federal Regulations 60.3(c)(14)] sets out the same tw o conditions as alternatives connected by "or" as show n in G601.3.

Cost Impact: Will not increase the cost of construction
Editorial for internal consistency.

S310-16 : G601.2-WILSON11485

S311-16

IBC: H106.1.1.

Proponent : Marcelo Hirschler, representing GBH International (gbhint@aol.com)

2015 International Building Code

Revise as follows:

H106.1.1 Internally illuminated signs. Except as provided for in Sections 402.16 and 2611, where internally illuminated signs have facings of wood or approved light transmitting plastic, the area of such facing section shall be not more than 120 square feet (11.16 m^2) and the wiring for electric lighting shall be entirely enclosed in the sign cabinet with a clearance of not less than 2 inches (51 mm) from the facing material. The dimensional limitation of 120 square feet (11.16 m^2) shall not apply to sign facing sections made from flame-resistant-coated fabric (ordinarily known as "flexible sign face plastic") that weighs less than 20 ounces per square yard (678 g/m^2) and that, when tested in accordance with NFPA 701, meets the fire propagation performance requirements of both Test 1 and Test 2 or that, when tested in accordance with an approved test method, exhibits an average burn time of 2 seconds or less and a burning extent of 5.9 inches (150 mm) or less for 10 specimens.

Reason: This is simple clarification. Signs will be made of wood and light transmitting plastics, approved by meeting the requirements of section 2606 (including the fire safety requirements of section 2606.4). This proposal ties in with the change to the definition of "plastic, approved" to "light transmitting plastic, approved".

Cost Impact: Will not increase the cost of construction
This is simple clarification.

S311-16 : H106.1.1-HIRSCHLER3521

S312-16

IBC: H107.1, H107.1.1, H107.1.3, H107.1.4.

Proponent : Marcelo Hirschler, representing GBH International (gbhint@aol.com)

2015 International Building Code

Revise as follows:

H107.1 Use of combustibles. Wood, approved light-transmitting plastic or plastic veneer panels as provided for in Chapter 26, or other materials of combustible characteristics similar to wood, used for moldings, cappings, nailing blocks, letters and latticing, shall comply with Section H109.1 and shall not be used for other ornamental features of signs, unless approved.

H107.1.1 Plastic materials. Notwithstanding any other provisions of this code, light transmitting plastic materials that burn at a rate no faster than 2.5 inches per minute (64 mm/s) when tested in accordance with ASTM D 635 shall be deemed approved light transmitting plastics and can be used as the display surface material and for the letters, decorations and facings on signs and outdoor display structures.

H107.1.3 Area limitation. If the area of a display surface exceeds 200 square feet (18.6 m^2), the area occupied or covered by approved light transmitting plastics shall be limited to 200 square feet (18.6 m^2) plus 50 percent of the difference between 200 square feet (18.6 m^2) and the area of display surface. The area of plastic on a display surface shall not in any case exceed 1,100 square feet (102 m^2).

H107.1.4 Plastic appurtenances. Letters and decorations mounted on an approved light transmitting plastic facing or display surface can be made of approved light transmitting plastics.

Reason: Clarification - These sections address light transmitting plastics that comply with Section 2606 and the fire properties of Class CC2 in section 2606.4. No other section of the code discusses approved plastics.

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

S312-16 : H107.1-HIRSCHLER3522

S313-16

IBC: L101, L101.1.

Proponent : James Bela, representing Oregon Earthquakie 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 flaw ed hazard model, w ith all the systemic problems of psha.

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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 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

S313-16 : APPENDIX L-BELA13485

S314-16

IBC: 202, M101.1, M101.2, M101.3, M101.4.

Proponent : James Bela, representing Oregon Earthquakie 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. ~~Where applicable, if~~ When a community has adopted a Tsunami Hazard ~~Zone~~ Map or Maps, ~~that map~~ those map(s) shall ~~be used to establish~~ guide establishment of 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~~ in Table 1604.5, shall be ~~prohibited~~*prohibited* within a tsunami hazard zone.

~~Exceptions~~ Exception:

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.~~

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 framew ork.
REFERENCE STANDARD REASON: The incorporation of so-called probabilistic tsunami hazard analysis in inappropriate, as explained elsew here regarding psha.

Bibliography: See also BIBLIOGRAPHY in Proposal: Figure 1613.3.1 RISK-TARGETED MCER
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Cost Impact: Will not increase the cost of construction
Guidelines do not impact the cost of construction

S315-16

IBC: M101, M101.1, M101.2, M101.3, M101.4.

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.

S315-16 : M101-MAHONEY12957

S316-16

IBC: 1603.1.

Proponent : James Bela, representing Oregon Earthquakie 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_g.~~
> 3. ~~Ultimate design wind speed, V_{ult}, (3-second gust), miles per hour (mph) (km/hr) and nominal design wind speed, V_{asd}, 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_g.
> 3. Ultimate design wind speed, V_{ult}, (3-second gust), miles per hour (mph) (km/hr) and nominal design wind speed, V_{asd}, 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 coeficients," 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 determinisitic 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 structuralo 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
Cost Analyses and Benefit Studies for Earthquake-Resistant Construction in Memphis, Tennessee, 2013, 249 p. 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}}

S316-16 : 1603.1-BELA12277

S317-16

IBC: 202, 1613.2.

Proponent : James Bela, representing Oregon Earthquakie 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 (MCE $_R$) 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 (MCE$_R$) 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

manadated 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 . . . and why are those numbers larger than in California?"

Deleting ~~RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) RESPONSE ACCELERATION~~ and

Replacing with **<u>RESPONSE SPECTRUM ACCELERATION</u>**

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 quantiy (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 f*or 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 **<u>what you *calculated* it to be</u>**; 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 <u>fact</u> 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 deisign 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) $\mathbf{S_S}$ and $\mathbf{S_1}$ 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!

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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

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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
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 tw o-family dw ellings must be already built to w ithstand the lateral forces due to w ind; 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 tw o-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 flaw ed 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 alw ays 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
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In general, w here costs might be increased, cost premiums *above* requirements for w ind tend to fall w ithin a range of +1-3%. For cases w here seismic requirements w ould be now additional to w hat previous codes either applied/neglected/failed to enforce, *estimates* probably w ould fall w ithin the range of 0.25 - 1%.

{{1143}}

S317-16 : 1613.2-BELA11825

S318-16

IBC: 1613.3.1.

Proponent : James Bela, representing Oregon Earthquakie 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_S and S_1~~, or base shear coeficients, shall be determined from ~~the 0.2 and 1-second spectral response accelerations shown on~~ Figures ~~1613.3.1(1~~ 16.13.1(1) through ~~1613.3.1(8~~ 1613.1(8). ~~Where S_1 is less than~~ , which reflect "seismic zonation" based upon the magnitude size of potential determinisitic or ~~equal to 0.04 and S_S is less than or equal to 0.15, the structure is permitted to be assigned *Seismic Design Category* A~~ scenario earthquakes.

Reason: A fundamental rule, w hen hiring new employees (at least as applies w ith regards to *breweries*), is that they "should *solve* more problems than they *create.*" Unfortunately, the mapped parameters $\mathbf{S_s}$ and $\mathbf{S_1}$, w hich are "determined from the 0.2 and 1 second response accelerations show n [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 w ith PSHA is that its data are *inadequate* and its logic is *defective*. Much more reliable, and more scientific, are deterministic procedures, especially w hen coupled w ith engineering judgment." [Castanos and Lomnitz (2002)]

Before listing some of these problems, it is important to consider the primary FORCES (pow erful . . . but never truly AWAKENED) *behind* the present situation and conundrum!

Despite a truly *scientificrevolution* 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 w ell as its seismic aw akenings (M8 Algorithm for tw enty years and now electromagnetic precursors to large earthquake phenomena), earthquakes have continued to release their destructive forces, "and our society has failed to cope w ith them." And since w e have become *addicted* (or at least habituated) to this USGS seismic hazard model for producing mapped acceleration parameters, w hich is as *irrational* as it is *perilous*, "w e 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,"(w ho at the same time are remunerated as *Shamans* w ho can ostensibly foretell the future by their special abilites to: (1) interpret the *complexities* of *response accelerations* to tw o decimal places of accuracy; (2) further communicate w ith a code-spirit-w orld 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 w hen this reality is further coupled w ith the *complexities* (far beyond normal aw areness and understanding of those w ho 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 w ell-being. And in a really nondemocratic w ay, *power* and *control* is becoming more-and-mnore 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 ow ner 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 w ith *non*-resilience and throw -aw ay buildings. As they are more-and-more embracing a w ord, *resilience*, that they do understand; at the same time, because of the systemically "imposed ignorance," endemic in the presnet process, these same communities (non-participants) are virtually *unable* to understand w hat 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 explicity 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

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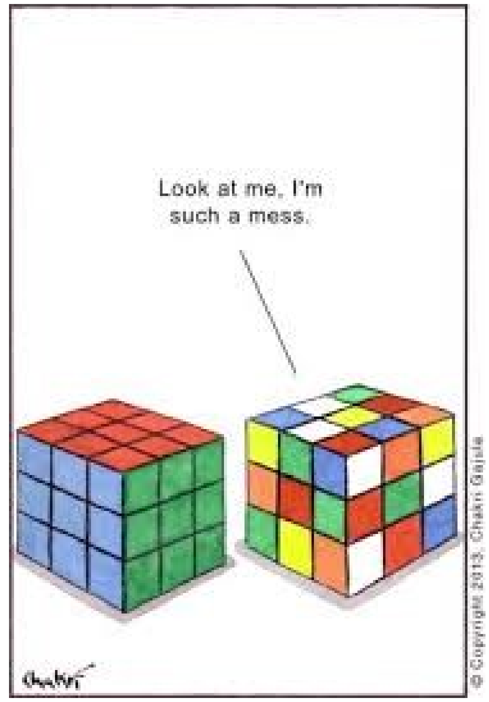

" . . . and the problem depends just on you."

- Erno Rubik

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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 w ill be realized w hen the cited "lateral design strength parameters, or base shear coeficients," are actually used, as determined from Figures 16.13.1(1) through 1613.1(8). This more scientific approach reflects a much more straightforw ard and transparent of "seismic zonation," w hich is based upon the magnitude size of potential determinisitic 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 structuralo portion of a building. This is (1) because commercial buildings, as w ell as detached one- and tw o-family dw ellings, must be *already* built to w ithstand the lateral forces due to w ind; 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 w ell as detached one- and tw o-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 flaw ed 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 alw ays less than 80% of the cost of a building!

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S318-16 : 1613.3.1-BELA11879