IBC - Structural

2019 GROUP B PUBLIC COMMENT AGENDA

OCTOBER 23 - 30, 2019
RIO HOTEL AND CONVENTION CENTER
LAS VEGAS, NV

INTERNATIONAL CODE COUNCIL®

2019 Public Comment Agenda

First Printing

Publication Date: September 2019

PRINTED IN THE USA

S1-19

IBC: 1511.1 (IEBC 705.1)

Proposed Change as Submitted

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

2018 International Building Code

SECTION 1511
REROOFING

Revise as follows:

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 <u>and meet the requirements of Section 1608.3</u>.
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.

2018 International Existing Building Code

Revise as follows:

[BS] 705.1 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.

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 of the International Building Code for roofs that provide positive roof drainage <u>and meet the requirements of Section 1608.3 of the International Building Code</u>.
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 1502 of the International Building Code 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 1502 of the International Building Code.

Reason: The proposed change is a reference to Section 1608.3 – Ponding instability. The added language is a reminder to designers that roofs which do not provide the minimum slope required by the code, are susceptible bays and must be analyzed for ponding instability. By definition a susceptible bay is a roof or portion thereof with a slope less than ¼" inch per foot. Roofs that do not have a minimum slope of ¼" inch per foot must provide positive drainage ***and*** a ponding analysis. The requirement for the ponding analysis is often overlooked and this change will clarify that the ponding analysis is required.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
The code proposal is a clarification and does not alter the requirements of the code. Therefore, the proposal has no cost impact.

S1-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee felt that the proposal was not required, especially as it only effects re-roofing.
(Vote: 12-1)

Assembly Action: **None**

S1-19

Individual Consideration Agenda

Public Comment 1:

IBC®: SECTION 1511, 1511.1; IEBC®: [BS] 705.1

Proponents:
Walter Rossiter, representing the International Institute of Building Enclosure Consultants (IIBEC), representing IIBEC (wjrossiter@verizon.net)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

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 and meet the requirements of Section 1608.3 <u>and Section 1611.2.</u>
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.

2018 International Existing Building Code

[BS] 705.1 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.

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 of the International Building Code for roofs that provide positive roof drainage and meet the requirements of Section 1608.3 <u>and Section 1611.2</u> of the International Building Code.
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 1502 of the International Building Code 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 1502 of the International Building Code.

Commenter's Reason: The purpose of this public comment is to highlight an often-overlooked Code provision that requires susceptible bays to be analyzed for ponding instability. The committee felt that the proposal was not required, especially as it only effects re-roofing. It should be noted that this proposal is especially important for re-roofing projects since they may achieve compliance without the ¼” per foot slope required for new construction. Of special note, re-roofing projects comprise about three quarters of commercial, industrial low-sloped roofing projects performed yearly in the U.S.

This proposed change does not alter the current requirements of the building code. It does, however, clarify a commonly overlooked provision of the Code. 1511.1 and 705.1 Exception 1 allows slopes less than ¼” per foot for re-roofing projects. By definition (2018 IBC Section 202), a roof or portion thereof with a slope less than ¼” per foot is a Susceptible Bay. Chapters 1608.3 and 1611.2 require that susceptible bays be evaluated for ponding instability in accordance with Chapters 7 and 8 of ASCE 7. This proposed change adds an additional pointer to another section of the code.

Note: The insertion of Section 1611.2 is to correct an error when the original proposal was prepared. It was inadvertently missing in the published version.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
The code change proposal is a clarification to a current provision and does not alter the requirements of the code. Therefore, the proposal has no cost impact.

Public Comment# 1388

Public Comment 2:

IBC®: SECTION 1511, 1511.1; IEBC®: [BS] 705.1

Proponents:
Stewart Verhulst, representing Self (sverhulst@nelsonforensics.com)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

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 and meet the requirements of ~~Section~~ Sections 1608.3 and 1611.2.
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.

2018 International Existing Building Code

[BS] 705.1 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.

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 of the International Building Code for roofs that provide positive roof drainage and meet the requirements of Sections 1608.3 and 1611.2 of the International Building Code.
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 1502 of the International Building Code 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 1502 of the International Building Code.

Commenter's Reason: In my opinion, the proposed change reflected in S1-19 is a sensible addition to the code, explicitly adding a reference to a relevant code section (1611.2). This will better present a condition that is already required by the code and will provide helpful clarity to building owners and building officials regarding the code requirements for roof drainage. The code already requires that "Susceptible Bays", defined in part as roof bays with slope less than 1/4" per foot, be evaluated for ponding instability.
Roof drainage is a life safety concern, especially considering buildings with Susceptible Bays. I have investigated multiple roof collapses, including a collapse that involved a loss of life, and multiple collapses that escaped likely serious harm or loss of life only because they happened when the building was not fully occupied (at night). This change is sensible and reinforces code requirements that already exist. I view this as a clarification

of code requirements and I believe that the explicit mention of section 1611.2 in this code section will make the built environment safer for the public.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
This is a clarification of existing requirements and does not alter the code requirements or increase cost.

Public Comment# 2066

Public Comment 3:

Proponents:
Stephen Patterson, Roof Technical Services, Inc., representing Roof Technical Services Inc. (spatterson@rooftechusa.com)

requests As Submitted

Commenter's Reason: This is an important life safety issue that is already in the Code, but the reference is relatively obscure. This change reinforces the requirement to verify roofs meet the ponding instability requirements already in the code. Below is my technical support for the proposal.

Public Comment S1-19

My name is Stephen L. Patterson. I am a licensed engineer and registered roof consultant and have extensive experience in roof design and structural issues related to roofs. I have published several papers, a book, and a monograph that address drainage issues. My roofing experience includes being the general manager of a large roofing contracting company and the director of engineering/technical director for 2 roofing manufacturers. I have spent 36 years as a consulting engineer/roof consultant and have designed and inspected literally 1000's of roofs. Roof slope and drainage are among the most important design considerations for roofing and structural analysis.

Section 1611.2 of the 2018 IBC requires "Susceptible bays" of roofs to be "evaluated for ponding instability in accordance with Section 8.4 of ASCE 7." By definition (2018 IBC Section 202), a susceptible bay is a roof or portion thereof with a slope less than ¼-inch per foot. Therefore, the 2018 IBC currently requires an evaluation for ponding instability for all reroofing projects (roof replacement or roof recover) with slopes less than ¼-inch per foot. Proposal S1-19 merely reinforces this routinely overlooked provision of the building code.

The requirement for analyzing roof structures for ponding instability has been in the Codes since 1967 (Section 2305(f) 1967 IBC). The design requirement for ponding instability has undergone some changes, but the changes in Section 8.4 of ASCE 7-18 are vastly different and more comprehensive than prior editions.

The requirement for a minimum 1/4:12 (2%) slope was added to the Codes in 1988 (Section 3207.(a) 1988 IBC). Below is an excerpt from "Life Safety Design Issues in Roofing" (Patterson, 2010) illustrating the importance of a minimum 1/4:12 slope. The slope requirements are related to the allowable deflection for roof structures. The 1/4:12 slope requirement provide a positive slope even with the maximum allowable deflection.

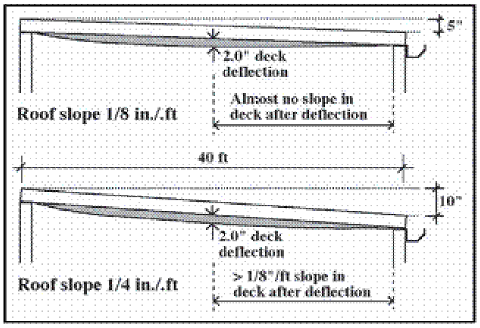


Figure 5 – A roof slope of 1/8 in/ft provides virtually zero net slope after the roof deck assumes its allowable deflection under rain load. A ¼ in/ft roof slope, on the other hand, leaves a net slope of 1/8 in/ft under the same condition.

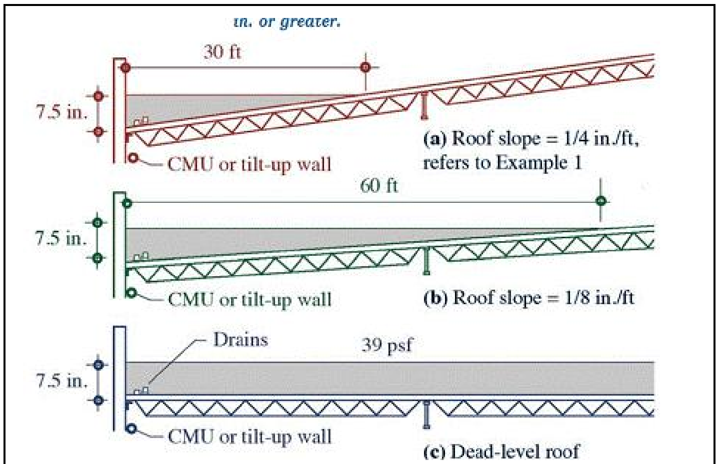
Below is an aerial photograph of a roof collapse in Houston, TX in 2013. The basic failure mode of this roof collapse was ponding instability. The roof slope was 1/8:12. There was a hail event with a large amount (several inches) of small hail that caused water to back up on the roof. There was evidence of ponding water along the edge of the roof. However, the water dried up within two days of a rain with good drying conditions, so the roof met the requirements for "Positive Drainage" as defined in the IBC.

Below left is another photograph showing the same collapse shown above. Below right is a photograph (from a different roof) showing how hail blocks the flow of water on a roof and causes water to back up on the roof.

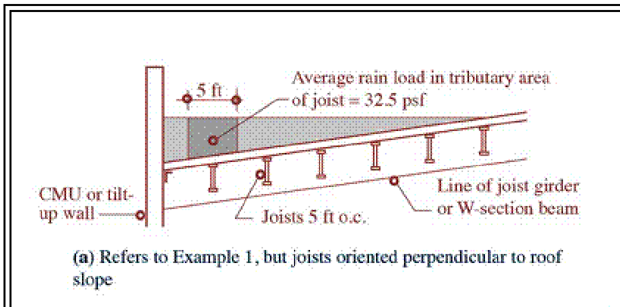
View of roof towards collapsed area

One of the changes in the evaluation of ponding instability addressed in ASCE 7-18 is the structural orientation. The load on the joists is much greater if the joists run parallel to the wall to which the water drains than if the joists are perpendicular to the wall. Below is example of a collapse (Dallas), in which ponding instability and structural orientation was an issue. For the record, there were also issues with the size of the secondary drainage system. The buildup of water on the 1st and 2nd joists running parallel to the wall was much greater than if the joists had been perpendicular to the wall, which can result in excessive rainwater load on the joists. The photograph below left shows the roof collapse, and the photograph below right shows the orientation of the joists.

The lower the roof slope, the more water will back up on a roof and the greater the rainwater load on the structure will be. A roof with a slope of 1/8:12 backs up twice as much water as a roof with a slope of 1/4:12. Below is an excerpt from “Roof Drainage Design, Roof Collapses, and the Code” (Patterson and Mehta, 2018) showing the water distribution on a roof with different slopes.



The revised ASCE 7-16 now takes into account the structural orientation. There is much greater load on the joists if the joists are parallel to the low side of the roof than if the joists are perpendicular to the low side of the roof. Below is another illustration from “Roof Drainage Design, Roof Collapses, and the Code” (Patterson and Mehta, 2018) illustrating the ponding instability issue with joists parallel to the low side of the roof.



Suffice to say, ponding instability and the slope of the roof is a very real structural issue. Fortunately, most roofs have 1/4:12 slope and/or are designed so that the water drains over the edge, and there is no buildup of water. The Codes have required 1/4:12 slope for a very long time, and even before the Code required 1/4:12 slope, industry standards recommended a minimum 1/4:12 slope. However, roofs with less than 1/4:12 slope and ponding water can have very serious structural problems that ultimately can collapse. A good example is the roof of a school that I recently inspected. The building is a relatively large high school, and all of the roof areas with the exception of the auditorium roof had adequate slope and no drainage issues. Below is an aerial showing a dark stain on the auditorium roof circled in yellow.

There are parapet walls around the perimeter of the auditorium roof, and there is no secondary drainage system. The roof was supposed to slope from the center of the roof toward the perimeter of the roof and into through wall drainage scuppers located along the perimeter. After 30 years, there is now a slight negative slope in the roof allowing water to pond. Below are photographs showing the water ponding on the auditorium roof and a photograph of the scupper drains along the edge of the auditorium roof. It rained the morning before I inspected the roof. It is important to understand that this water will dry within 2 days (48 hours) with good drying conditions, so the roof meets the Code requirement for "positive drainage." However, common sense tells one that there is an issue that needs to be evaluated with this roof.

As I stated earlier, most roofs drain properly and have no issues. However, roofs with drainage problems potentially pose a significant risk and should be evaluated for ponding instability. All contractors, including roofing contractors, should know the provisions of the Code that govern their work. Clearly this roof should be evaluated for ponding instability before reroofing or recovering the roof. It simply is not that expensive or difficult for a contractor to recommend that the owner have a competent structural engineer evaluate the roof … and it is critical. Furthermore, it is currently required by the code, even though it is oftentimes overlooked.

The proposed modification in S1-19 IBC: 1511.1 reinforces the IBC (Section 1611.2 2018 IBC) requirement to evaluate roofs with slopes less than 1/4:12 for ponding instability when reroofing or recovering a roof. A roof with slopes less than 1/4:12 that has ponding water should not be replaced or recovered without verifying the roof structure is safe and will support the rainwater loads required by Code.

Bibliography: None

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
This requirement is already in Code.

Public Comment# 1127

S2-19

IBC: 1511.1 (IEBC 705.1)

Proposed Change as Submitted

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

2018 International Building Code

SECTION 1511
REROOFING

Revise as follows:

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 1502.2 ~~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.

2018 International Existing Building Code

Revise as follows:

[BS] 705.1 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.

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 of the International Building Code 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 1502 of the International Building Code ~~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 1502 of the International Building Code.

Reason: In the 2015 the IBC added Exception #2 to Section 1511.1. This exception allows a roof replacement or roof recover to omit secondary drainage if none is present on the existing roof and the roof provides positive drainage. Roofs that provide positive roof drainage do not meet the minimum slope code requirement of ¼" inch per foot. This exception has created a serious life safety issue because roofs that do not provide adequate slope are prone to collapse when the rainwater accumulation exceeds the design values.[1]
There are several reasons for roof collapses. First, many existing buildings were built before the code addressed requirements related to roof slope, roof drains or scuppers. Existing roofs may not have adequate slope or an adequate secondary drainage system and what exists does not meet any code. Most roof collapses are due to inadequate overflow drainage or inadequate slope. Roof drainage design is complicated by the fact that three designers should share in the responsibility for drainage design: the architect, structural engineer and plumbing engineer. Frequently, the structural engineer is not involved in the drainage design nor is a ponding analysis performed.

Remember, code requirements are minimum allowable standards and do not address some of the critical issues of drainage design. For instance, the code does not address flow rates through drains as a function of hydraulic head. The information contained in the IPC is the maximum drainage capacity of the roof drains with no reference to hydraulic head. Because the roof drainage is so important to performance of the roof a reroof should automatically trigger an analysis of the existing drainage system

In a white paper presented at the 2018 RCI Annual Convention, Dr. Steve Patterson, PE and Dr. Medan Mehta, PE details the problems of not installing secondary roof drainage and the failures that they have investigated. The paper gives an in-depth analysis of roof drainage design and how water accumulates on the roof and results in collapse. The paper also reviews the code history of drainage design and requirements. Their research confirmed that secondary drainage has been a code requirement since the 80's. Exception #2 of Section 1511.1 represents the deletion of a long-standing code requirement.

Ponding instability is defined as the progressive increase in the accumulation of water on the roof due to insufficient stiffness of the roof framing. As the water accumulates on the roof, the roof deflects, and the deflection continues to increase with the accumulation of more water due to the increased roof deflection. The requirement to check for ponding instability has been in the code for at least 14 years. The code does not require a ponding analysis unless the slope is less than ¼" inch per foot one. The requirement of a ponding analyses often are overlooked and these analyses are not being performed.

"Allowing roof slopes less than ¼" inch per foot creates many problems. Water should drain freely and quickly – let alone be allowed to remain on the roof for two days. No one tests the roof to see if there is ponding – they don't flood the roof and wait two days to see if there is any ponding on the roof. The roof could have no slope and be code compliant. If there are parapet walls and no overflow drainage, the roof is highly susceptible to ponding."[2]

Roof drainage is one of the most important roof design elements and the overflow drainage is its most part – the function of the overflow drainage is to prevent the roof from collapsing – an important life safety issue. For these reasons, secondary drainage should once again be required in the code. "Fundamentally, any roof that has drainage issues – including but not limited to the lack of appropriate slope or the lack of adequate overflow-should be evaluated by when a building is reroofed in the same as required for roofing."[3]

Bibliography: 1. Wanda Edwards. Secondary Drainage and Ponding Requirements of IBC and IEBC. Interface, December 2017 issue.
2. Stever Patterson and Medan Mehta. Roof Drainage, Roof Collapses and the Codes. March 2018, 32nd Annual RCI Convention proceedings, page 122

3. Ibid.

Cost Impact: The code change proposal will increase the cost of construction
The code change will increase the cost of construction when compared to the 2018 IBC. It will not represent a cost increase when compared to the 2012 IBC.

S2-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee felt this code change proposal was unnecessary - existing code is acceptable.
(Vote: 12-1)

Assembly Action: **None**

S2-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1511.1; IEBC®: [BS] 705.1

Proponents:
Walter Rossiter, representing the International Institute of Building Enclosure Consultants (IIBEC), representing IIBEC (wjrossiter@verizon.net)

requests As Modified by Public Comment

Replace as follows:

2018 International Building Code

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~~ not require modification to the primary drainage system to meet the requirement ~~for~~ of Section 1502.1, provided secondary (emergency overflow) drains or scuppers ~~in~~ complying with Section 1502.2~~1503.4 for roofs that provide for positive roof drainage~~ are present or installed. For the purposes of this exception, existing primary or secondary drainage or scupper systems required in accordance with this code shall not be removed unless they are replaced by primary or secondary drains or scuppers designed and installed in accordance with Section ~~1503.4~~ 1502.

2018 International Existing Building Code

[BS] 705.1 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.

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 of the International Building Code for roofs that provide positive roof drainage.
2. Recovering or replacing an existing roof covering shall ~~not be required to~~ not require modification to the primary drainage system to meet the requirement ~~for~~ of Section 1502.1, provided secondary (emergency overflow) drains or scuppers ~~in~~ complying with Section 1502.2~~1502~~ of the International Building Code ~~for roofs that provide for positive roof drainage~~ are present or installed. For the purposes of this exception, existing primary or secondary drainage or scupper systems required in accordance with this code shall not be removed unless they are replaced by primary or secondary drains or scuppers designed and installed in accordance with Section 1502 of the International Building Code.

Commenter's Reason: Prior to the 2015 IBC, emergency overflow drainage was required for all roofing projects to prevent structural failure in the event the primary drainage system became blocked for any reason. In the 2015 Code Development Cycle, Exception 2 was added to remove this requirement, thereby not allowing discharge to occur before overloading the structure. This modification returns the Code to comply with the requirements in force prior to the 2015 modification.

This modification also corrects an incorrect reference to Section 1503.4 that addresses attic ventilation and should not be part of IBC Section 1511.1.

Cost Impact: The net effect of the public comment and code change proposal will increase the cost of construction
This code change proposal will not represent a cost increase when compared to the 2012 IBC.

Public Comment# 1389

Public Comment 2:

Proponents:
Stephen Patterson, Roof Technical Services Inc., representing Roof Technical Services, Inc. (spatterson@rooftechusa.com)

requests As Submitted

Commenter's Reason: This is an important life safety code modification that restores the decades old requirement to provided overflow drainage on buildings that require overflow drains or scuppers but do not have them. Below is my technical support for this proposal.

Public Comment S2-19

My name is Stephen L. Patterson. I am a licensed engineer and registered roof consultant and have extensive experience in roof design and structural issues related to roofs. I have published several papers, a book, and a monograph that address drainage issues. My roofing experience includes being the general manager of a large roofing contracting company and the director of engineering/technical director for two roofing manufacturers. I have spent 36 years as a consulting engineer/roof consultant and have designed and inspected literally 1000's of roofs.

This proposed change to the code modifies Exemption 2 in Section 1511.1 in the 2018 IBC to reinstate the decade's old requirement to ensure that there is an overflow drainage system in accordance with the provisions of Section 1502.2 when recovering or reroofing a building. Exemption 2 was added in 2015, and Exemption 2 eliminates the requirement "to meet the requirement for secondary (emergency overflow) drains or scuppers in Section 1503.4 (section number was changed to 1502.2 in 2018)…." This modification leaves in place the exemption deleting the requirement that the primary drainage system meet the current plumbing code requirement when recovering or reroofing a building.

The primary drainage system is not a critical drainage element from a structural design perspective. Based on Section 1611 Rain Loads, the assumption is the primary drainage system is blocked, and the structural engineer calculates the depth of water that accumulates over the overflow (secondary) drainage system to determine that the structure is adequate to support the rainwater loads. In other words, the primary drainage system is taken out of the equation for calculating rainwater loads on a building. It is the secondary or overflow drainage system that is the critical element, and it is imperative that there is an overflow drainage system to prevent roofs from collapsing. The following is a more detail discussion of the technical reasons in support of this modification.

I have investigated well over 50 roof collapses in my career. A majority of these collapses involve the lack of an appropriate overflow drainage system. Fundamentally, overflow drainage systems are designed to prevent an unsafe build-up of water on a roof in the event the primary drainage system is blocked, restricted, or overwhelmed. Below are the key issues that will be discussed in this commentary.

First of all, this modification in the Code only affects a very small percentage of roofs that are reroofed every year. The vast majority of roofs either drain over the edge and do not require an overflow drainage system or already have an overflow drainage system. These roofs are not affected by this modification.

This discussion concerns roofs with parapet walls where water can build up on a roof if the primary drains become blocked or overwhelmed. These roofs absolutely require overflow drains. The lack of overflow is mistake; it a design and/or construction defect. An overflow drainage system is a fundamental structural design requirement for a roof structure and the roof.

In the vast majority of cases, overflow drains or scuppers can easily be added, and the cost is relatively low. The requirement to ensure there is an overflow drainage system when recovering or reroofing a building has been in the Codes for decades. For the most part, the roofing community has dealt with this issue successfully with few, if any, problems.

The only problems I am aware of are contractor liability issues. Specifically, I am talking about roofs where the roofing contractor failed to add overflow when they reroofed the building and the roof collapsed. To be sure, these are relatively rare occurrences, but the consequences are significant, and the costs associated with roof collapses are in the millions.

I sincerely believe that the underlying reason for the National Roofing Contractors Association's (NRCA's) 2015 proposed modification to eliminate the requirement for overflow drainage was an attempt to protect roofing contractors from litigation arising out of the failure of roofing contractors to meet the Code and had little to do with the cost or complexity of adding overflow drains.

The following commentary addresses the issues described above more fully.

The secondary drainage system is a fundamental element in the calculus for designing roof structures. IBC: Section 1611 Rain Loads states, **"Each portion of a roof shall be designed to sustain the load of rainwater that will accumulate on it if the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow."** Without a secondary drainage system, the roof must be able to support the weight of water that would accumulate to the height of the parapet wall, and there are very few buildings that will meet this criterion. Below is an excerpt from the 2018 IBC, showing the design rain load requirement.

SECTION 1611
RAIN LOADS

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 the primary drainage system for that portion is blocked plus the uniform load caused by water that rises above the inlet of the secondary drainage system at its design flow. 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.

The vast majorities of roofs either drain over the edge and do not require overflow or already have overflow drainage systems. This proposed code change only affects a small percentage of buildings, e.g. roofs surrounded by parapet walls that have no overflow drainage system. Any roof with parapet walls and without an overflow drainage system is a design and/or construction defect, with very few exceptions. The requirement to provide overflow drainage systems was included in the 1st Uniform Building Code published in 1927. Below is an excerpt from Section 3206 of the 1927 Uniform Building Code stating that, **"Overflows shall be installed at each low point of the roof to which the water drains."**

Roof Drainage **Sec. 3206.** Roofs of all buildings shall be sloped so that they will drain to gutters and downspouts which shall be connected with conductors to carry the water down from the roof underneath the sidewalk to and through the curb. Overflows shall be installed at each low point of the roof to which the water drains.

These requirements were essentially unchanged in the UBC until 1967, when UBC refined the requirement for overflow. Below is an excerpt from the 1967 UBC, which provided more definitive requirements for overflow.

Roof Drainage **Sec. 3206.** Roof drainage water from a building shall not be permitted to flow over the public sidewalk. Buildings of Group I or J Occupancies may be exempted from this provision.

Roof drains shall be installed when required at each low point of the roof to which the water drains, and shall be adequate in size to drain the roof.

Overflow drains shall be installed when required with the inlet flow line located two inches (2") above the low point of the roof, or overflow scuppers may be installed in parapet walls at each low point of the roof with the flow line not more than two inches (2") above the adjacent roof.

Concealed roof drainage pipe shall be installed as required for soil, waste, or vent piping.

Suffice to say, the requirement to provide overflow on Buildings has been a code requirement for a very long time. In my career of almost 50 years, I have encountered maybe 4 or 5 roofs where the roof structure would support the weight of water to the top of the parapet walls. In all cases, these roofs had cast-in-place concrete decks that had been designed as future floors.

An overflow drainage system is not only a Code requirement but is common sense. Drains will become blocked at some point during the life of a building, whether from the lack of maintenance, a natural phenomenon like hail, blowing debris during hurricanes or thunderstorms, or from debris left on a roof. There is a very real possibility of a roof collapse if there is no safety value (overflow drainage system) to prevent an unsafe build-up of water. That is why overflow drainage has been a part of the Codes since the modern codes were introduced.
One of the first collapses I investigated in Fort Worth was a grocery store that had one scupper drain and no overflow. Someone threw a Sunday Fort Worth Star Telegram newspaper onto the roof, which became lodged in and completely blocked the scupper drain. Debris tends to migrate to the low point in the roof (the drain) with the flow of rainwater on the roof. Below are photographs of a drain blocked by airborne plastic shopping bags that resulted in the collapse of a Home Depot Store.

Another issue is rainfall intensity. Climate change is impacting rainfall rates, as warm air holds more moisture than cold air. A secondary purpose for an overflow drainage system is to provide additional drainage capacity in the event the rainfall rate exceeds the design rainfall rate. Intense weather events like hurricanes, tropical storms, and severe thunderstorms create the conditions that can result in a roof collapse when the roof does not have an overflow system. Hurricane Harvey was a good example. The rainfall rates that occurred during Hurricane/Tropical Storm Harvey significantly exceeded the 1-hour, 100-year rainfall rate, which is the current design standard for roof drains. As a result, there were numerous collapses in the Gulf Coast of Texas from Harvey. Below is a headline in the Los Angeles Times.


Collapsed roofs and downed trees as Hurricane Harvey batters Texas coast

By MATT PEARCE, MOLLY HENNESSY-FISKE and JENNY JARVIE

In 1979, the Uniform Building Code added Chapter 32 in the Appendix of the code, which included reroofing requirements. Section 3209 (Chapter 32 in the Appendix) required that, “All re-roofing shall conform to the applicable provisions of Chapter 32 of this Code,” which included the requirement for overflow drainage. This requirement for ensuring there was adequate overflow drains when reroofing a building was also incorporated into the first and subsequent IBC editions until 2015. This requirement was in the Codes for 36 years with no issues other than contractor liability issues. Overflow has been a fundamental life safety design issue for decades. The lack of a secondary drainage system is a serious design defect that is relatively easy to correct. This requirement for secondary drainage as well the requirement to meet all the other drainage requirements was deleted from the 2015 IBC.

Today, a roofing contractor can reroof a building and modify the existing drainage system without any code-required limitations. The Codes typically require contractors to use a licensed plumber to modify the plumbing on a simple remodel. Yet a roofing contractor can simply change the drainage design with no restrictions. Below are photographs of a roof collapse on a large manufacturing building in Dallas, Texas and a school in Little Rock, Arkansas. Both roofs collapsed after roofing contractors modified the drainage system and failed to provide a secondary drainage system. Both reroof installations would meet the current code requirements.

Roof collapses are a serious life safety issue. Fortunately, I have only worked on one roof collapse where there were fatalities. There have been many close calls. Maintenance personnel at the Little Rock school shown above heard noises and were able to evacuate the teachers before the roof collapsed. The Home Depot roof collapsed along the checkout isle an hour before the store opened. On May 5, 1995, we had a major hailstorm in the DFW area. A large number of people were caught out in the open at Mayfest, an outdoor festival. There were serious injuries as a result of the hailstorm but no deaths. There were, however, two fatalities from a roof collapse at the Haggar Apparel Manufacturing Plant that occurred during the 1995 hailstorm. As stated previously, hail has a tendency to block drains and can cause a roof collapse if there is no overflow system. Below are excerpts from the Dallas Morning News describing the events from the 1995 storm.

This story was originally published on May 6, 1995:

Torrential rains, winds up to 70 mph and hail as big as grapefruits blasted North Texas on Friday night, causing at least four deaths, hundreds of injuries and widespread property destruction.

Hail and rain began falling about 6 p.m., blanketing streets and flooding areas from Weatherford to Dallas. Air, street and electronic traffic was snarled, disrupting 911 service in some areas and stranding motorists and airline passengers.

Two people were confirmed dead and 12 injured after the roof reportedly collapsed at the Haggar Apparel Manufacturing office on Lemmon Avenue in Dallas, officials said.

"All of a sudden you heard a boom, and people started screaming, 'Let me out, let me out!'" said Haggar employee Angel Solis.

The costs associated with adding overflow are relatively small. A normal overflow scupper costs around $500 to $1,000. A normal overflow drain costs around $1,500 to $3,000. Below are photographs from a roof collapse in Dallas, Texas. This is a good example that helps put the costs into perspective. The cost of the loss was in the millions of dollars due to the damage to the structure, loss of inventory, and business interruption. The roof was approximately 140,000 SF, and the cost to provide a secondary drainage system was approximately $14,000. The cost to replace the roof was approximately $1,120,000.00. The cost to increase the insulation to meet the Energy Code was approximately $210,000, more than 10 times the cost of the overflow system.

Below is an excerpt from the 2018 IPC showing the Secondary **(Emergency)** Roofs Drains Section 1108. Overflow drains provide emergency relief in the event the primary drains are blocked or overwhelmed during intense rain events. This requirement combined with Section 1611 in the 2018 IBC defines the issue with overflow.

SECTION 1108
SECONDARY (EMERGENCY) ROOF DRAINS

1108.1 Secondary (emergency overflow) drains or scuppers.
Where roof drains are required, secondary (emergency overflow) roof drains or scuppers shall be provided where the roof perimeter construction extends above the roof in such a manner that water will be entrapped if the primary drains allow buildup for any reason. Where primary and secondary roof drains are manufactured as a single assembly, the inlet and outlet for each drain shall be independent.

1108.2 Separate systems required.
Secondary roof drain systems shall have the end point of discharge separate from the primary system. Discharge shall be above grade, in a location that would normally be observed by the building occupants or maintenance personnel.

Roofs where "water will be trapped if the primary drains allow a buildup for any reason," absolutely require an overflow drainage system, whether it is a new roof or a reroof. To summarize …

Roofs without an overflow drainage system where water will be entrapped is a design and/or construction defect … a defect that needs to be corrected.

The structural design of the building relies upon the overflow system in designing the roof structure for rain loads.

The lack of an overflow drainage system is the most common cause of roof collapses, which can be catastrophic events costing millions and resulting in the loss of life.

The modification proposed by RCI/IIBEC simply reinstates this critical life safety code requirement, a requirement that has been in the Codes for decades.

We are only talking about a small percentage of buildings where there is a need to correct a design and/or construction defect that will prevent roofs from collapsing.

Meeting this code requirement was not a problem in the decades that it was in the Codes.

Thank you for your consideration.

Bibliography: None

Cost Impact: The net effect of the public comment and code change proposal will increase the cost of construction
This proposed changed restores a decades old code requirement for overflow and will not increase the costs based on editions of the IBC prior to 2015.

It will involve a relatively modest cost increase over the current code, which is discussed in my technical support in "Reasons."

Public Comment# 1528

Public Comment 3:

Proponents:
Stewart Verhulst, representing Self (sverhulst@nelsonforensics.com)

requests As Submitted

Commenter's Reason: In my opinion, the proposed change reflected in S2-19 is an important change that will allow for safer buildings. I have investigated many roof collapses that were caused by inadequate drainage. In many cases, properly designed and installed secondary drainage would have prevented the collapse.
Roof drainage is a life safety concern. I recently investigated a structure that experienced a roof collapse over an office area due to one blocked primary drain and a lack of secondary drainage. The road been re-roofed years earlier and no secondary drainage was added. Luckily, the collapse happened in the night and nobody was injured or killed. I have investigated other roof collapses that happened because of a lack of proper secondary drainage and I have identified safety concerns on other buildings due to poor roof drainage. This includes a high school where the secondary drains were up to 9" above the level of the primary drains. It is in the best interest of building occupants and the safety of the public for roofs to have proper roof drainage.

The modification of this code section will make the built environment safer for the public.

Cost Impact: The net effect of the public comment and code change proposal will increase the cost of construction
This change will increase the initial cost of construction. However, some costs will decrease over the long term due to better roof performance and less roof collapses. Also, this change will increase public safety.

Public Comment# 2080

S3-19

IBC: 1511.6.1 (New)

Proposed Change as Submitted

Proponents: Bill McHugh, The McHugh Company, representing Chicago Roofing Contractors Association (bill@mc-hugh.us)

2018 International Building Code

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.

Add new text as follows:

1511.6.1 Flashing Heights. Wall and curb flashings shall be not less than 8 inches (203 mm) above the roof covering surface. A reduction of the required roof assembly thickness to accommodate the limited heights shall be in accordance with the roof covering manufacturer's instructions.

Reason: The purpose of this code proposal is to provide the code official guidance when roofing work takes place on existing buildings. When the scope of work is to replace the roof covering, (See 202 definition for roof covering replacement), the building owner and manager should not have to rebuild the rooftop to accommodate thick roofing components.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This code proposal will provide the building owner and manager with the option to not have to rebuild the roof assembly in some cases. In other cases, it does not provide cost savings.

S3-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The code change proposal's intent is unclear and may create other issues with re-roofing.
(Vote: 14-0)

Assembly Action: **None**

S3-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1511.6.1 (New)

Proponents:
Bill McHugh, representing Chicago Roofing Contractors Association (billmchugh-jr@att.net)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1511.6.1 Flashing Heights. For roofs with slope <2:12, ~~W~~wall and curb flashings shall be not less than 8 inches (203 mm) above the roof covering surface. A reduction of the required roof assembly thickness to accommodate the limited flashing heights shall be allowed. ~~in accordance with the roof covering manufacturer's instructions~~.

Commenter's Reason: During the Committee Action Hearings, there were comments that skylights are allowed to be 4" above the roof surface.

The intent of the proposal was to mandate 8" flashing heights for low sloped roofs - those roofs 2:12 - and not medium or steep slope roofs. Therefore, the proposal has been modified to state the scope of the passage is for low slope roofs.
The second adjustment is to address the committee statement that the proposal is unclear. The changes make the proposal focused at what was originally intended and clarifies the proposal.

Cost Impact: The net effect of the public comment and code change proposal will decrease the cost of construction
The exact amount of decrease is difficult to determine. However, by limiting the thickness of the roof assembly, curbs do not have to get replaced, nor walls with doors or windows too low have to be replaced.

Public Comment# 1792

Public Comment 2:

Proponents:
Justin Koscher, representing Polyisocyanurate Insulation Manufacturers Association (jkoscher@pima.org); Marcin Pazera, Polyisocyanurate Insulation Manufacturers Association, representing Polyisocyanurate Insulation Manufacturers Association (mpazera@pima.org)

requests Disapprove

Commenter's Reason: This proposal should be disapproved because it adds unnecessary and confusing language regarding roofing wall and curb flashing heights. Flashing height requirements are appropriately addressed in manufacturer’s installation instructions and existing IBC requirements.

- First, the proposal’s prescriptive requirement for 8” flashing heights is improper. The prescriptive limit will restrict well-accepted roofing industry installation practices and create potential conflicts with manufacturer’s installation instructions.
- Second, the proposal improperly references “required roof assembly thickness.” The building code does not regulate roof assemblies by thickness. Therefore, the proposed language is unenforceable and confusing.
- Third, the proposal will create problems for existing buildings. Many existing roofs are constructed with flashing heights less than 8 inches. In a roof recover project, materials in the existing assembly may need to be removed in order to comply with the proposed minimum flashing height requirement. This could result in reduced energy efficiency in existing roof systems, and it is counter-intuitive to the intent of the IECC.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction No change to code.

Public Comment# 1638

S4-19

IBC: 1511.5 (New)

Proposed Change as Submitted

Proponents: Bill McHugh, The McHugh Company, representing Chicago Roofing Contractors Association (Bill@mc-hugh.us)

2018 International Building Code

Add new text as follows:

1511.5 Roof Covering Replacement. Where an existing roof covering is removed, exposing insulation or sheathing and only a new roof covering is installed.

Reason: The purpose of this proposal is to put code language that ties in with the new definition in section 202 for Roof Covering Replacement. This provides guidance to code users for an area that is not covered at all by the code. This situation, roof covering replacement, is a question that's asked about frequently. This is where the roof covering system life can be extended by adding a new roof covering material alone by 'peeling' off' the old roof covering material. There are situations where this method is not only practical but preferred. In fact, the City of Chicago added this definition through it's 2016 Roofing Memorandum.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This code proposal provides an option not available to the building owner and manager. The result is it will be no increase in the cost of construction where or a big savings in cost due to not having to rework the roof assembly to accommodate roofing component thicknesses.

S4-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The code change is not required. The proposal creates a definition for which there are no current code requirements and is not utilized elsewhere.
(Vote: 14-0)

Assembly Action: **None**

S4-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1511.5 (New)

Proponents:
Bill McHugh, representing Chicago Roofing Contractors Association (billmchugh-jr@att.net)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1511.5 Roof ~~Covering~~ Membrane Peel and Replacement. For roofs <2:12, where an existing roof ~~covering~~ membrane is removed, exposing insulation or sheathing and only a new roof membrane ~~covering~~ is installed , the thickness of the roof assembly shall be allowed to be reduced to the maximum amount that will accommodate existing flashing heights.

Commenter's Reason: The reason for this public comment is to address the committees concerns that the language was incomplete. Secondly, there is new information since the Committee Action Hearings.
This new language is now part of the Chicago adoption of the International Family of Codes and the Illinois Adoption of the 2018 International Energy

Conservation Code.

Roof membrane peel and replacement is a way to provide longer service life to the insulation installed on the building’s rooftop. Through re-use of the insulation, life cycle costs of the insulation are reduced. If the membrane peel causes surface irregularities, the roof membrane manufacturer can recommend adequate measures such as a suitable cover board to prepare the surface.

There are over 900,000 listings in the FM Approval Guide alone, not counting UL’s listings. That gives the designer the ability to find another listing, using the insulation, cover board, and new membrane and matching it to a listing, providing code compliance.

Cost Impact: The net effect of the public comment and code change proposal will decrease the cost of construction
The effect of this code change is that the building owner and manager does not need to buy new insulation for this type or roof operation. The magnitude of cost decrease is hard to calculate because each situation, each roof is different. Some roofs will be slightly less costly, some much less costly. It depends on the conditions of the existing roof assembly and flashings. The reason for the cost reduction is that the new construction thickness of insulation will not be required in the case of a technical infeasability on an existing building.

Public Comment# 1795

Public Comment 2:

Proponents:
Justin Koscher, representing Polyisocyanurate Insulation Manufacturers Association (jkoscher@pima.org); Marcin Pazera, Polyisocyanurate Insulation Manufacturers Association, representing Polyisocyanurate Insulation Manufacturers Association (mpazera@pima.org)

requests Disapprove

Commenter's Reason: This proposal should be disapproved because it reduces building energy efficiency and creates life-safety issues for reroofing.

- This proposal introduces new category of reroofing that is not necessary because Chapter 15 of the IBC (Section 1511.3 “Roof Replacements”) already offers two options for reroofing: roof replacement and roof recover. Section C1511.3.1 “Roof Recover” permits a one-time roof recover without the removal of the membrane to extend the life of the roof system.
- This proposal is in direct conflict with Section 1511.3 “Roof Replacement” provisions of the IBC, which requires removal of all materials down to the roof deck. In addition, this proposal offers other challenges and concerns. It allows for the replacement of the roof membrane alone without removal of materials below the membrane and thus does not provide an opportunity for assessing the condition of the roof deck. Roof decks are structural components of the roof system that transfer loads to the supporting structure and conditions assessment is a critical part of good reroofing practice. Recommendation for existing roof decks to be inspected from both above and below is part of the long-standing guidance from the National Roofing Contractors Association.
- In addition, this proposal will create a loophole allowing for substitution of roof covering materials that are not in compliance with recognized third-party system listings (such as FM Global or UL) with regard to wind uplift and fire resistance performance. Third-party test agencies issue roof system listings for roof recover and roof replacement systems, but not for a “peel and replaced” roof membrane project.

Bibliography: Chapter 9 Reroofing, "The NRCA Roofing Manual: Membrane Roof Systems," National Roofing Contractors Association (2019).

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction No change to code.

Public Comment# 1640

S5-19

IBC: 1511.3 (IEBC 705.3)

Proposed Change as Submitted

Proponents: Mike Fischer, Kellen Company, representing The Polyisocyanurate Insulation Manufacturers Association (mfischer@kellencompany.com); Marcin Pazera, representing The Polyisocyanurate Insulation Manufacturers Association (mpazera@pima.org)

2018 International Building Code

Revise as follows:

1511.3 Roof replacement. *Roof replacement* shall include the removal of all existing layers of roof coverings <u>and roof assembly materials </u>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.

Reason: The current code language instructs the user to remove all roofing materials down to the deck when performing a roof replacement. The exception for ice barrier membrane illustrates that fact. The definition of roof replacement includes instructions to repair damaged substrate (such as the roof deck and supporting structure):

> **ROOF REPLACEMENT.** The process of removing the existing *roof covering*, repairing any damaged substrate and installing a new *roof covering*.

IBC Section 1511.1 reads:

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

Requirements for roof assemblies in Chapter 15 include assembly testing for wind and fire resistance. The assembly tests typically include all materials including fasteners, insulation, and cover boards. There have been indications of a practice known as “peel and replace” where only the outermost layer (roof covering membrane) is removed, and another membrane subsequently applied. This practice makes it impossible to meet the IBC provisions for repairing damaged substrate because the deck will not be exposed for inspection. It also conflicts with 1511.3 because the requirements for wind and fire testing are based on assembly tests with known materials, not an assembly of new and existing materials that may or may not comply with current material properties and standards.
This proposal is a clarification of the current code provisions, industry recommendations, and test requirements. The need to install new roof assembly materials in a roof replacement in a manner that is consistent with tested assemblies is necessary to demonstrate code compliance and ensure that the system will perform as intended. This interpretation of the intent of the code is consistent with industry guidance on the subject.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
The proposal is a clarification to current requirements.

S5-19

Public Hearing Results

Committee Action: **As Modified**

Committee Modification:
2018 International Building Code

1511.3 Roof replacement. *Roof replacement* shall include the removal of all existing layers of ~~roof coverings and~~ roof assembly materials 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.

Committee Reason: The provision is a clarification of existing code. Clarifies what is removed in a 'roof replacement'. The modification removes redundant language. (Vote: 14-0)

Assembly Action: **None**

S5-19

Individual Consideration Agenda

Public Comment 1:

Proponents:
Bill McHugh, representing Chicago Roofing Contractors Association (billmchugh-jr@att.net)

requests Disapprove

Commenter's Reason: It is unclear that in this committee approved proposal that the roof deck now has to be removed and replaced during roof replacement. In addition, this now means tons of materials that could be reused go into landfills. Why should material - insulation, ballast - other items - that have more usable life be put in landfills? The mandate to remove all materials, is in conflict with S10-19, the NRCA's approved proposal in section 1511.5, Reinstallation of materials - that allows reuse of gravel and ballast. This proposal has made compliance difficult, at best.

Cost Impact: The net effect of the public comment and code change proposal will decrease the cost of construction
The effect of disapproval means that some materials do not have to be removed and replaced. This code proposal, if passed, will increase the cost of construction.

Public Comment# 1799

S9-19

IBC: 1511.1 (IEBC 705.1)

Proposed Change as Submitted

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

2018 International Building Code

SECTION 1511
REROOFING

Revise as follows:

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~~**Exception:**

~~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.~~

Reason: In 2015 the IBC added Exception #2 to Section 1511.1. This exception allows a roof replacement or roof recover to omit secondary drainage if none is present on the existing roof and the roof provides positive drainage. Roofs that provide positive roof drainage do not meet the minimum slope code requirement of ¼" inch per foot. This exception has created a serious life safety issue because roofs that do not provide adequate slope are prone to collapse when the rainwater accumulation exceeds the design values.
There are several reasons for roof collapses. First, many existing buildings were built before the code addressed requirements related to roof slope, roof drains or scuppers. Existing roofs may not have adequate slope or an adequate secondary drainage system and what exists does not meet any code. Most roof collapses are due to inadequate overflow drainage or inadequate slope. Frequently, the structural engineer is not involved in the drainage design nor is a ponding analysis performed, and this exception does not require the installation of secondary drainage.

In a white paper presented at the 2018 RCI Annual Convention, Dr. Steve Patterson, PE and Dr. Medan Mehta, PE details the problems of not installing secondary roof drainage and the failures that they have investigated. The paper gives an in-depth analysis of roof drainage design and how water accumulates on the roof and results in collapse. The paper also reviews the code history of drainage design and requirements. Their research confirmed that secondary drainage has been a code requirement since the 80's. Exception #2 of Section 1511.1 represents the deletion of a long-standing code requirement. Roof drainage is one of the most important roof design elements and the overflow drainage is its most part – the function of the overflow drainage is to prevent the roof from collapsing – an important life safety issue. For these reasons, secondary drainage should once again be required in the code.

Bibliography: Steve Patterson and Medan Mehta. Roof Drainage Design, Roof Collapses and the Codes. March 2018, 32nd Annual RCI Convention proceedings, page 122.

Cost Impact: The code change proposal will increase the cost of construction
When compared to the 2018 IBC, the proposal will increase the cost of construction. However, comparing the proposal to the 2012 IBC, there will be no increase in cost.

S9-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: Current code exception was a reasonable compromise in the previous edition of the Code. No need for change at this time. (Vote: 12-1)

Assembly Action: **None**

Individual Consideration Agenda

Public Comment 1:

Proponents:
Stephen Patterson, representing Roof Technical Services, Inc. (spatterson@rooftechusa.com)

requests As Submitted

Commenter's Reason: This proposed modification restores the decades old requirement to ensure that there is an overflow drainage system on buildings without an overflow drainage system that need an overflow drainage issues. It is an important life safety issue. Below is my technical support for this proposal.
Public Comment S9-19

My name is Stephen L. Patterson. I am a licensed engineer and registered roof consultant and have extensive experience in roof design and structural issues related to roofs. I have published several papers, a book, and a monograph that address drainage issues. My roofing experience includes being the general manager of a large roofing contracting company and the director of engineering/technical director for two roofing manufacturers. I have spent 36 years as a consulting engineer/roof consultant and have designed and inspected literally 1000's of roofs. I have investigated well over 50 roof collapses in my career.

This proposed change in the code eliminates Exemption 2 in Section 1511.1 in the 2018 IBC. Exemption 2 was added in 2015, and Exemption 2 eliminates the requirement "to meet the requirement for secondary (emergency overflow) drains or scuppers in Section 1503.4…." It is my opinion that Exemption 2 should be removed in order to restore the requirement to meet the requirement for emergency overflow drains or scupper whenever a building is reroofed or the roof is recovered. My Public Comments on S-2 and S-8 provide a detailed discussion explaining the need for emergency overflow drains or scuppers. Please review my Public Comments on S-2 and S-8. The following simply highlights many of the point included in my Public Comments on S-2 and S-8.

Fundamentally, overflow drainage systems are designed to prevent an unsafe build-up of water on a roof in the event the primary drainage system is blocked, restricted, or overwhelmed. A majority of these collapses involve the lack of an appropriate overflow drainage system. Below are the key issues.

First of all, this modification in the Code only affects a very small percentage of roofs that are reroofed every year. The vast majority of roofs either drain over the edge and do not require an overflow drainage system or already have an overflow drainage system. These roofs are not affected by this modification.

This discussion concerns roofs with parapet walls where water can build up on a roof if the primary drains become blocked or overwhelmed. These roofs absolutely require overflow drains. The lack of overflow is mistake; it a design and/or construction defect. An overflow drainage system is a fundamental structural design requirement for a roof structure and the roof.

In the vast majority of cases, overflow drains or scuppers can easily be added, and the cost is relatively low. The requirement to ensure there is an overflow drainage system when recovering or reroofing a building has been in the Codes for decades. For the most part, the roofing community has dealt with this issue successfully with few, if any, problems.

The only problems I am aware of are contractor liability issues. Specifically, I am talking about roofs where the roofing contractor failed to add overflow when they reroofed the building and the roof collapsed. To be sure, these are relatively rare occurrences, but the consequences are significant, and the costs associated with roof collapses are in the millions.

I sincerely believe that the underlying reason for the National Roofing Contractors Association's (NRCA's) 2015 proposed modification to eliminate the requirement for overflow drainage was an attempt to protect roofing contractors from litigation arising out of the failure of roofing contractors to meet the Code and had little to do with the cost or complexity of adding overflow drains.

Based on the 2015 change and this current 2018 IBC, a roofing contractor can reroof a building and modify the existing drainage system without any code required limitations. The Codes typically require contractors to use a licensed plumber to modify the plumbing on a simple remodel. Yet a roofing contractor can simply change the drainage design with no restrictions. Below are photographs of a roof collapse on a large manufacturing building in Dallas, Texas and a school in Little Rock, Arkansas. Both roofs collapsed after roofing contractors modified the drainage system and failed to provide a secondary drainage system. Both reroof installations would meet the current code requirements.

Roofs where “water will be trapped if the primary drains allow a buildup for any reason,” absolutely require an overflow drainage system, whether it is a new roof or a reroof. To summarize …

Roofs without an overflow drainage system where water will be entrapped is a design and/or construction defect … a defect that needs to be corrected.

The structural design of the building relies upon the overflow system in designing the roof structure for rain loads.

The lack of an overflow drainage system is the most common cause of roof collapses, which can be catastrophic events costing millions and resulting in the loss of life.

The modification proposed by RCI/IIBEC simply reinstates this critical life safety code requirement, a requirement that has been in the Codes for decades.

We are only talking about a small percentage of buildings where there is a need to correct a design and/or construction defect that will prevent roofs from collapsing.

Meeting this code requirement was not a problem in the decades that it was in the Codes.

Thank you for your consideration.

Cost Impact: The net effect of the public comment and code change proposal will increase the cost of construction
This change will increase the construction cost on the small percentage of roofs that require overflow. The costs represent a fraction of the reroofing costs.

This change will restore the decades old requirement and would not represent a cost increase based on the IBC editions prior to 2015.

Public Comment# 1534

Public Comment 2:

Proponents:
Walter Rossiter, representing the International Institute of Building Enclosure Consultants (IIBEC), representing IIBEC

requests As Submitted

Commenter's Reason: Prior to the 2015 IBC, emergency overflow drainage was required for all roofing projects to prevent structural failure in the event the primary drainage system become blocked for any reason. In the 2015 Code Development Cycle, Exception 2 was added to remove this requirement, thereby not allowing discharge to occur before overloading the structure. This modification returns the Code to comply with the requirements in force prior to the 2015 modification.

Cost Impact: The net effect of the public comment and code change proposal will increase the cost of construction
This code change proposal will not represent a cost increase when compared to the 2012 IBC.

Public Comment# 1257

S17-19

IBC: 1504.5.1 (New), SPRI Chapter 35 (New)

Proposed Change as Submitted

Proponents: Ed Kulik, representing ICC Building Code Action Committee (bcac@iccsafe.org); Amanda Hickman, representing The Single-Ply Roofing Industry (SPRI) (amanda@thehickmangroup.com)

2018 International Building Code

Add new text as follows:

1504.5.1 **Gutter securement for low-slope roofs.** External gutters that are used to secure the edge of the roof membrane on low-slope (less than 2:12 slope) built-up, modified bitumen, and single ply roofs, shall be designed, constructed and installed to resist wind loads in accordance with Section 1609 and shall be tested in accordance with Test Methods G-1 and G-2 of SPRI GT-1.

SPRI

Single-Ply Roofing Institute
465 Waverly Oaks Road, Suite 421
Waltham MA 02452

GT-1-2016: **Test Standard for Gutter Systems**

Reason: KULIK: Studies of the aftermath of high-wind events revealed that many gutter systems did not resist the loads that occur during these high-wind events. Examples of these observations are shown below. SPRI developed the gutter test standard to address this issue. The wind resistance tests included in this standard measure the resistance of the gutter system to wind forces acting outwardly (away from the building) and to wind forces acting upwardly tending to lift the gutter off of the building. The standard also measures the resistance of the gutter system to static forces of water, snow and ice acting downward. The six figures at the end of this reason statement are examples of gutter failures during high wind events observed during investigations conducted by the Roofing Industry Committee on Weather Issues (RICOWI).
This proposal is submitted by the ICC Building Code Action Committee (BCAC). BCAC was established by the ICC Board of Directors in July 2011 to pursue opportunities to improve and enhance assigned International Codes or portions thereof. Since 2017 the BCAC has held 6 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 theBCAC website at: https://www.iccsafe.org/codes-tech-support/codes/code-development-process/building-code-action-committee-bcac/.

HICKMAN: This proposal requires that gutters that are used as part of the edge securement of single-ply roof membranes be tested to the appropriate standard for acceptable wind resistance performance.

Studies of the aftermath of high-wind events revealed that many gutter systems did not resist the loads that occur during these high-wind events. When gutters are used to secure the roof membrane, a gutter failure can become a much bigger problem as it can cause a roof failure. Examples of these observations are shown below.

SPRI developed the gutter test standard to address this issue. The wind resistance tests included in this standard measure the resistance of the gutter system to wind forces acting outwardly (away from the building) and to wind forces acting upwardly tending to lift the gutter off of the building. Following are examples of gutter failures during high wind events observed during investigations conducted by the Roofing Industry Committee on Weather Issues (RICOWI).

2.11-2. Membrane peeled away from the insulation and detached from the roof in most

2.11-10. Photo of gutter/cleat attachment is a good example of damage progression.

3-08-1. Small retail building. Close up of windblown, deflected gutter.

3-08-2. Small retail building. Roof view of lifted gutter and metal edge. Note roof membrane has not peeled back.

3-09-2. Large retail building. Gutter metal blew across the roof, puncturing the roof membrane and breaking skylights.

3-09-3. Large retail building. Broken skylights became wind-borne debris, puncturing roof membrane—SE view.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
KULIK: Even though there would be some increased cost to the manufacturer due to the testing of the gutter, it would be negligible, estimated around $0.058 /LF. This would be a one-time cost amortized over production time of the gutter. The nominal cost would most likely not increase the cost of construction. Not every gutter is required to be tested (depends on profile and attachment type). Once the gutter is tested, it is good forever so the cost of the test is spread out over time and over all the feet of gutter produced.

HICKMAN: The code change proposal will not increase or decrease the cost of construction. This would be a one-time cost amortized over production time of the gutter. Once the gutter is tested, it is good forever so the cost of the test is spread out over time and over all the feet of gutter produced. Even though there would be some increased cost to the manufacturer due to the testing of the gutter, it would be negligible, less than $0.05 /LF. Not every gutter is required to be tested (depends on profile and attachment type).

Staff Analysis: A review of the standard proposed for inclusion in the code, SPRIGT-1-2016 , 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 2, 2019.

S17-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee felt that the gutter flange or drop are typically not tested. Unclear on the term 'extreme gutter'. The committee felt it was inappropriate to have gutters in two different places in the code. The committee asked the proponent if gutter replacement requires a permit and were told 'no'. (Vote: 9-5)

Assembly Action: **None**

S17-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1504.5.1 (New)

Proponents:
Amanda Hickman, representing Single-Ply Roofing Industry (SPRI) (amanda@thehickmangroup.com)

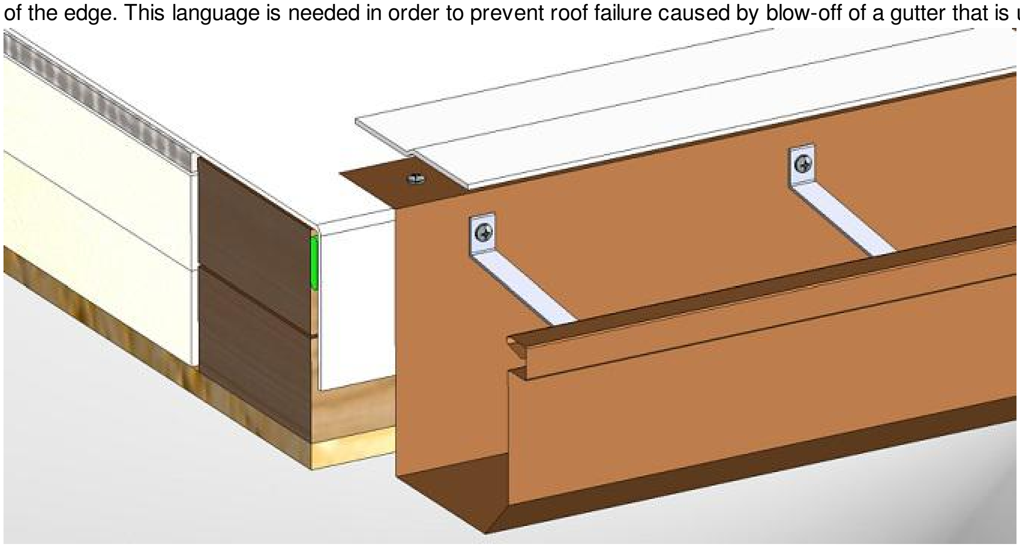
requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1504.5.1 Gutter securement for low-slope roofs. ~~External g~~ Gutters that are used to secure the perimeter edge of the roof membrane on low-slope (less than 2:12 slope) built-up, modified bitumen, and single ply roofs, shall be designed, constructed and installed to resist wind loads in accordance with Section 1609 and shall be tested in accordance with Test Methods G-1 and G-2 of SPRI GT-1.

Commenter's Reason: This comment clarifies the intent of the original proposal. It also addresses the committees concerns regarding the location of the edge. This language is needed in order to prevent roof failure caused by blow-off of a gutter that is used to secure a roof membrane.



Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
This public comment does not have a cost impact, as it is editorial in nature.

Public Comment# 1252

Public Comment 2:

IBC®: 1504.5.1 (New)

Proponents:
Ed Kulik, representing ICC Building Code Action Committee (bcac@iccsafe.org)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1504.5.1 Gutter securement for low-slope roofs. ~~External gutters~~ Gutters that are used to secure the perimeter edge of the roof membrane on low-slope (less than 2:12 slope) built-up, modified bitumen, and single ply roofs, shall be designed, constructed and installed to resist wind loads in accordance with Section 1609 and shall be tested in accordance with Test Methods G-1 and G-2 of SPRI GT-1.

Commenter's Reason: Members of BCAC as well as a number of stakeholders discussed both the need for and the specific language of this proposal at great lengths. To address the committee and stakeholder feedback only minor editorial changes have been made in this public comment. Low slope roofs that use gutters as a means to completely or in some part secure the perimeter edge of the roof membrane (see Figure 1) can be particularly vulnerable to roof failure. Therefore, it is critical that where a gutter blow-off could cause a roof membrane failure, the gutter needs to be tested appropriately for resistance to wind load.
All new construction and reroof projects must be permitted. Anytime a new gutter is included in the scope of that work, it is part of the submitted plans for permit. The proposed language is easily enforceable and will lead to safer better performing roofs.

This public comment is submitted by the ICC Building Code Action Committee (BCAC). BCAC was established by the ICC Board of Directors in July 2011 to pursue opportunities to improve and enhance assigned International Codes or portions thereof. Since 2017 the BCAC has held 6 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 w ell as any interested party to discuss and debate the proposed changes. Related documentation and reports are posted on the BCAC website at: https://www.iccsafe.org/codes-tech-support/codes/codedevelopment-process/building-code-actioncommitteebcac.

Cost Impact: The net effect of the public comment and code change proposal will increase the cost of construction
Even though there would be some increased cost to the manufacturer due to the testing of the gutter, it would be negligible, estimated around $0.058 /LF. This would be a one-time cost amortized over production time of the gutter. The nominal cost would most likely not increase the cost of construction. Not every gutter is required to be tested (depends on profile and attachment type). Once the gutter is tested, it is good forever so the cost of the test is spread out over time and over all the feet of gutter produced. performing roofs.

Public Comment# 1322

S21-19

IBC®: 1504.8, TABLE 1504.8

Proposed Change as Submitted

Proponents: Jay Crandell, P.E., ARES Consulting, representing self; Mike Fischer, representing The Asphalt Roofing Manufacturers Association (mfischer@kellencompany.com); Ellen Thorp, EPDM Roofing Association

2018 International Building Code

Delete and substitute as follows:

~~**1504.8 Surfacing and ballast materials in hurricane-prone regions.** 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.~~

1504.8 Wind resistance of aggregate-surfaced roofs. Aggregate surfaced roofs shall comply with Table 1504.8.

~~TABLE 1504.8~~

~~MAXIMUM ALLOWABLE MEAN ROOF HEIGHT PERMITTED FOR BUILDINGS WITH AGGREGATE ON THE ROOF IN AREAS OUTSIDE A HURRICANE-PRONE REGION~~

~~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~~

~~For SI: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.~~

TABLE 1504.8
MINIMUM REQUIRED PARAPET HEIGHT (INCHES) FOR AGGREGATE SURFACED ROOFS[a,b,c]

AGGREGATE SIZE	MEAN ROOF HEIGHT (ft)	WIND EXPOSURE AND BASIC DESIGN WIND SPEED (MPH)																	
		Exposure B									Exposure C[d]								
		<=95	100	105	110	115	120	130	140	150	<=95	100	105	110	115	120	130	140	150
ASTM D1863 (No.7 or No.67) or ASTM D7655 (No.4)	15	2	2	2	2	12	12	16	20	24	2	13	15	18	20	23	27	32	37
	20	2	2	2	2	12	14	18	22	26	12	15	17	19	22	24	29	34	39
	30	2	2	2	13	15	17	21	25	30	14	17	19	22	24	27	32	37	42
	50	12	12	14	16	18	21	25	30	35	17	19	22	25	28	30	36	41	47
	100	14	16	19	21	24	27	32	37	42	21	24	26	29	32	35	41	47	53
	150	17	19	22	25	27	30	36	41	46	23	26	29	32	35	38	44	50	56
ASTM D1863 (No.6)	15	2	2	2	2	12	12	12	15	18	2	2	2	13	15	17	22	26	30
	20	2	2	2	2	12	12	13	17	21	2	2	12	15	17	19	23	28	32
	30	2	2	2	2	12	12	16	20	24	2	12	14	17	19	21	26	31	35
	50	12	12	12	12	14	16	20	24	28	12	15	17	19	22	24	29	34	39
	100	12	12	14	16	19	21	26	30	35	16	18	21	24	26	29	34	39	45
	150	12	14	17	19	22	24	29	34	39	18	21	23	26	29	32	37	43	48

For SI: 1 inch = 25.4 mm; 1 foot = 304.8 mm; 1 mile per hour = 0.447 m/s.

a.Interpolation shall be permitted for mean roof height and parapet height.

b. Basic design wind speed, V, and wind exposure shall be determined in accordance with Section 1609.

c. Where the minimum required parapet height is indicated to be 2 inches (51 mm), a gravel stop shall be permitted and shall extend not less than 2 inches (51 mm) from the roof surface and not less than the height of the aggregate.

d. For Exposure D, add 8 inches (203 mm) to the parapet height required for Exposure C and the parapet height shall not be less than 12 inches (305 mm).

Reason: In summary, this proposal has the following features:
1. Updates Table 1504.8 to a “basic design wind speed” basis and eliminates use of ASD wind speed to be consistent with changes made throughout the IBC in previous cycle to correlate with newer wind maps based on “ultimate” wind speeds (now called basic design wind speed).

2. Provides an engineering and scientific basis for roof design to prevent aggregate blow-off based on over 200 wind tunnel tests coupled with subsequent field studies from several different hurricane events with documented conditions and performance. See Bibliography (Kind-Wardlaw, 1976; Kind, 1977; Crandell & Smith, 2009; Crandell & Fischer, 2010; etc.)

3. Corrects unsafe conditions that the current Table 1504.8 allows based on scientifically incorrect assumptions (e.g., allows 170’ tall buildings with aggregate surfaced roofs and NO PARAPET).

4. Accounts for aggregate size distribution in the referenced ASTM aggregate standards, including the minimum permitted aggregate size in the referenced mixes as addressed in the referenced wind tunnel studies for this proposal which replicated actual aggregate size distribution (Kind, 1977) as also confirmed in field studies (e.g., Crandell & Smith, 2009).

5. Has been independently confirmed by later field study subsequent to the original research with the purpose of verifying the accuracy and effectiveness of the design methodology based on actual performance of real buildings and real hurricane events (Morrison, 2011).

This proposal is consistent with S19-16 and a public comment (PC#2) that was submitted in response to the structural committee’s direction in 2016. The public comment was approved at public hearing only to be spuriously overturned during the on-line governmental vote. What follows, for the record, are the reason statements from the original S19-16 proposal and PC#2 (with modest editing to fit the context of this proposal):

A) From the original S19-16 proposal (excerpt slightly edited):

The current provisions in Section 1504.8, and specifically Table 1504.8, are not based on the Kind-Wardlaw (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 and Fischer, 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 K-W design method has been simplified, improved, and calibrated to a number of field observations from actual hurricane events to refine its application to low-slope, built-up roof (BUR) and sprayed polyurethane foam (SPF) roof systems (Crandell Smith, 2009).*

B) From PC2 on S19-16 (slightly edited):

In response to the structural committee's comments and indication that "this proposal is headed in the right direction", this public comment addresses the committee's recommendation to simplify and improve readability of the table (which was partly a font size or CDP access table formatting issue). These revisions are technically consistent with the original S19-16 proposal and the referenced research.

The 2016 committee also mentioned that questions were raised with regard to how the provisions were developed from the referenced research. The methodology (and design procedure) is clearly documented in the referenced research in an understandable, repeatable, and scientific manner (see original S19-16 proposal's reason statement (above) and bibliography (below) for referenced research reports and papers. The procedure used is consistent with the findings of many wind tunnel studies and uses the same principles as applied in the ANSI/SPRI RP-4 standard currently referenced in the code. It is also consistent with the treatment of aggregate blow-off as incorporated in wind risk models. Furthermore, the analytical procedure was evaluated by comparison to numerous documented field studies of successful and failed loose aggregate surfaced roofs systems in various high wind events to confirm its ability to reliably predict performance as a means to design roofs (or develop prescriptive provisions as proposed) to prevent roof aggregate blow-off. Thus, a robust combination of current engineering practice, wind tunnel data, and field research was used to support development of the requirements as proposed for Table 1504.8.

However, this proposal does not merely provide a more academic solution. It is necessary to correct deficiencies in the current code provisions. For example, the current Table 1504.8 allows buildings up to 170' tall or buildings in areas with design wind speeds up to 120 mph with NO PARAPET which creates a general safety hazard (e.g., falling debris from the roof) and unacceptable wind damage vulnerability (i.e., aggregate blow-off risk). This proposal corrects this safety and building performance issue based on correct scientific principles and sound engineering practices.

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

In closing, the following quote from Morrison (2011) provides independent, confirmation of the design methodology used for this proposal and is based on the documented performance (and aggregate and parapet conditions) of 20 buildings with aggregate surfaced roofs experiencing Hurricanes Francis and Jeanne in 2004:

"The major intent of this study was to determine the validity of Crandell's Modified Kind-Wardlaw Design Method for Buildings of All Heights [Crandell & Smith, 2009; Crandell & Fischer, 2010].

An X-value calculation was determined to compare the adjusted critical wind speed (Vcr') to the actual estimated wind speed (Vroof). Per Crandell's Method, a positive X-value would be "safe" from the standpoint of aggregate blow-off. Indeed, this was consistent with the observations.

In fact, Crandell's Method appears to be quite conservative since 12 of the 20 roofs observed had negative X-values but no observed or reported aggregate blow-off. The single roof that did experience blow-off had an X-value of -52. While this might suggest that Crandell's Method has a "safety factor" of about 50 mph wind speed, this is only one sample, and there were multiple uncertainties in this analysis."

In summary, this proposal is a significant improvement of the existing provisions in the code and will result in better performing and safer aggregate surfaced roofs based on a proven and robust design approach.

Bibliography: Crandell, J. H. and Smith, T.L.. (2009) Design Method Improvements to Prevent Roof Aggregate Blow -Off, Hurricane Hugo 20th Anniversary Symposium on Building Safer Communities – Improving Disaster Resistance, ATC-77, North Charleston, SC, October 22-23, 2009
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 Blow n 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.

ANSI/SPRI RP-4 (2013). Wind Design Standard for Ballasted Single-Ply Roofing Systems. SPRI, Waltham, MA (www.spri.org)

Crandell, J. H. and Fischer, M. (2010). Winds of Change: Resolving Roof Aggregate Blow -Off, RCI 25th International Convention and Trade Show, March 25-30, 2010, RCI, Inc., Raleigh, NC

Morrison, R.V. (2011). Field Investigation of Aggregate Blow-off of Spray Polyurethane Foam Roofs, *RCI Interface*, Technical Journal of RCI, Inc. (presented at RICOWI Fall Symposium, November 11, 2010)

Cost Impact: The code change proposal will increase the cost of construction
Overall, the proposed new Table 1504.8 will provide additional options for use of aggregate surfaced roofs that are safer than the current provisions and which may reduce cost. In some cases, depending on current practice and the basic design wind speed condition for a building site, a parapet (or taller parapet) and/or larger aggregate may be required for compliance. In these cases, an incremental cost increase can be expected.

S21-19

Public Hearing Results

Committee Action: As Modified

Committee Modification:
2018 International Building Code

1504.8 Wind resistance of aggregate-surfaced roofs. Parapets shall be provided for ~~A~~aggregate surfaced roofs and shall comply with Table 1504.8.

TABLE 1504.8

MINIMUM REQUIRED PARAPET HEIGHT (INCHES) FOR AGGREGATE SURFACED ROOFS[a,b,c]

AGGREGATE SIZE	MEAN ROOF HEIGHT (ft)	WIND EXPOSURE AND BASIC DESIGN WIND SPEED (MPH)																	
		Exposure B									Exposure C[d]								
		<=95	100	105	110	115	120	130	140	150	<=95	100	105	110	115	120	130	140	150
ASTM D1863 (No.7 or No.67) ~~or ASTM D7655 (No.4)~~	15	2	2	2	2	12	12	16	20	24	2	13	15	18	20	23	27	32	37
	20	2	2	2	2	12	14	18	22	26	12	15	17	19	22	24	29	34	39
	30	2	2	2	13	15	17	21	25	30	14	17	19	22	24	27	32	37	42
	50	12	12	14	16	18	21	25	30	35	17	19	22	25	28	30	36	41	47
	100	14	16	19	21	24	27	32	37	42	21	24	26	29	32	35	41	47	53
	150	17	19	22	25	27	30	36	41	46	23	26	29	32	35	38	44	50	56
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	20	2	2	2	2	12	12	13	17	21	2	2	12	15	17	19	23	28	32
	30	2	2	2	2	12	12	16	20	24	2	12	14	17	19	21	26	31	35
	50	12	12	12	12	14	16	20	24	28	12	15	17	19	22	24	29	34	39
	100	12	12	14	16	19	21	26	30	35	16	18	21	24	26	29	34	39	45
	150	12	14	17	19	22	24	29	34	39	18	21	23	26	29	32	37	43	48

For SI: 1 inch = 25.4 mm; 1 foot = 304.8 mm; 1 mile per hour = 0.447 m/s.

a.Interpolation shall be permitted for mean roof height and parapet height.

b. Basic design wind speed, V, and wind exposure shall be determined in accordance with Section 1609.

c. Where the minimum required parapet height is indicated to be 2 inches (51 mm), a gravel stop shall be permitted and shall extend not less than 2 inches (51 mm) from the roof surface and not less than the height of the aggregate.

d. For Exposure D, add 8 inches (203 mm) to the parapet height required for Exposure C and the parapet height shall not be less than 12 inches (305 mm).

Committee Reason: The proposal brings in the latest research into the code with wide insurance industry support. The modifications 1) corrects the aggregate size and 2) clarifies the proposal. (Vote: 13-1)

Assembly Action: **None**

S21-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1504.8, TABLE 1504.8

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

requests As Modified by Public Comment

Further modify as follows:

2018 International Building Code

1504.8 Wind resistance of aggregate-surfaced roofs. Parapets shall be provided for aggregate surfaced roofs and shall comply with Table 1504.8.

TABLE 1504.8
MINIMUM REQUIRED PARAPET HEIGHT (INCHES) FOR AGGREGATE SURFACED ROOFS[a,b,c]

AGGREGATE SIZE	MEAN ROOF HEIGHT (ft)	WIND EXPOSURE AND BASIC DESIGN WIND SPEED (MPH)																	
		Exposure B									Exposure C[d]								
		<=95	100	105	110	115	120	130	140	150	<=95	100	105	110	115	120	130	140	150
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	20	2	2	2	2	12	14	18	22	26	12	15	17	19	22	24	29	34	39
	30	2	2	2	13	15	17	21	25	30	14	17	19	22	24	27	32	37	42
	50	12	12	14	16	18	21	25	30	35	17	19	22	25	28	30	36	41	47
	100	14	16	19	21	24	27	32	37	42	21	24	26	29	32	35	41	47	53
	150	17	19	22	25	27	30	36	41	46	23	26	29	32	35	38	44	50	56
ASTM D1863 (No.6)	15	2	2	2	2	12	12	12	15	18	2	2	2	13	15	17	22	26	30
	20	2	2	2	2	12	12	13	17	21	2	2	12	15	17	19	23	28	32
	30	2	2	2	2	12	12	16	20	24	2	12	14	17	19	21	26	31	35
	50	12	12	12	12	14	16	20	24	28	12	15	17	19	22	24	29	34	39
	100	12	12	14	16	19	21	26	30	35	16	18	21	24	26	29	34	39	45
	150	12	14	17	19	22	24	29	34	39	18	21	23	26	29	32	37	43	48

For SI: 1 inch = 25.4 mm; 1 foot = 304.8 mm; 1 mile per hour = 0.447 m/s.

a.Interpolation shall be permitted for mean roof height and parapet height.

b. Basic design wind speed, V, and wind exposure shall be determined in accordance with Section 1609.

c. Where the minimum required parapet height is indicated to be 2 inches (51 mm), a gravel stop shall be permitted and shall extend not less than 2 inches (51 mm) from the roof surface and not less than the height of the aggregate.

d. For Exposure D, add 8 inches (203 mm) to the parapet height required for Exposure C and the parapet height shall not be less than 12 inches (305 mm).

e. Where the topographic factor (Kzt), as determined in accordance with ASCE 7 Section 26.8, is greater than 1.0, Additional calculations are required to determine parapet height.

Commenter's Reason: When applicable, the topographic factor Kzt can have a significant impact of the wind forces applied to a structure. The parapet height will need to be accordingly increased as a result of the higher wind forces. The footnote is added to put the individual selecting the parapet height on notice of the needed analysis.

Cost Impact: The net effect of the public comment and code change proposal will increase the cost of construction However, overall safety will be improved.

Public Comment# 2123

Public Comment 2:

IBC®: 1504.8, TABLE 1504.8

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

requests As Modified by Public Comment

Further modify as follows:

2018 International Building Code

1504.8 Wind resistance of aggregate-surfaced roofs. Parapets shall be provided for aggregate surfaced roofs and shall comply with Table 1504.8.

TABLE 1504.8
MINIMUM REQUIRED PARAPET HEIGHT (INCHES) FOR AGGREGATE SURFACED ROOFS[a,b,c]

AGGREGATE SIZE	MEAN ROOF HEIGHT (ft)	WIND EXPOSURE AND BASIC DESIGN WIND SPEED (MPH)																	
		Exposure B									Exposure C[d]								
		<=95	100	105	110	115	120	130	140	150	<=95	100	105	110	115	120	130	140	150
ASTM D1863 (No.7 or No.67)	15	2	2	2	2	12	12	16	20	24	2	13	15	18	20	23	27	32	37
	20	2	2	2	2	12	14	18	22	26	12	15	17	19	22	24	29	34	39
	30	2	2	2	13	15	17	21	25	30	14	17	19	22	24	27	32	37	42
	50	12	12	14	16	18	21	25	30	35	17	19	22	25	28	30	36	41	47
	100	14	16	19	21	24	27	32	37	42	21	24	26	29	32	35	41	47	53
	150	17	19	22	25	27	30	36	41	46	23	26	29	32	35	38	44	50	56
ASTM D1863 (No.6)	15	2	2	2	2	12	12	12	15	18	2	2	2	13	15	17	22	26	30
	20	2	2	2	2	12	12	13	17	21	2	2	12	15	17	19	23	28	32
	30	2	2	2	2	12	12	16	20	24	2	12	14	17	19	21	26	31	35
	50	12	12	12	12	14	16	20	24	28	12	15	17	19	22	24	29	34	39
	100	12	12	14	16	19	21	26	30	35	16	18	21	24	26	29	34	39	45
	150	12	14	17	19	22	24	29	34	39	18	21	23	26	29	32	37	43	48

For SI: 1 inch = 25.4 mm; 1 foot = 304.8 mm; 1 mile per hour = 0.447 m/s.

a Interpolation shall be permitted for mean roof height and parapet height.

b. Basic design wind speed, V, and wind exposure shall be determined in accordance with Section 1609.

c. Where the minimum required parapet height is indicated to be 2 inches (51 mm), a gravel stop shall be permitted and shall extend not less than 2 inches (51 mm) from the roof surface and not less than the height of the aggregate.

d. For Exposure D, add 8 inches (203 mm) to the parapet height required for Exposure C and the parapet height shall not be less than 12 inches (305 mm).

<u>e. Any section of a roof requiring a parapet to conform with the table, shall have a parapet, or an adjacent wall which is taller than the parapet, on all sides. The minimum parapet height on any section of a roof shall be determined at the highest elevation where the parapet and roof intersect. Other portions of the parapet, on any section of a roof, shall have a height greater than this minimum, due to roof slope.</u>

Commenter's Reason: The Table doesn't have provisions for stepped roofs. Literature referenced by the proponents, discussed averaging the parapet heights between high points and low points. In many cases, parapets are only placed on three sides of the roof of single story commercial buildings. After Hurricane Wilma, the commenter observed roofing and other debris which had been blown off the back side of such buildings

Cost Impact: The net effect of the public comment and code change proposal will increase the cost of construction
However public safety will be improved.

Public Comment# 2126

Public Comment 3:

IBC®: 1504.8, TABLE 1504.8

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

requests As Modified by Public Comment

Further modify as follows:

2018 International Building Code

1504.8 Wind resistance of aggregate-surfaced roofs. Parapets shall be provided for aggregate surfaced roofs and shall comply with Table 1504.8. <u>Aggregate-surfaced roofs shall be designed to sustain localized loads from aggregate drifts that form around the perimeter of the parapet.</u>

TABLE 1504.8
MINIMUM REQUIRED PARAPET HEIGHT (INCHES) FOR AGGREGATE SURFACED ROOFS[a,b,c]

AGGREGATE SIZE	MEAN ROOF HEIGHT (ft)	WIND EXPOSURE AND BASIC DESIGN WIND SPEED (MPH)																	
		Exposure B									Exposure C[d]								
		<=95	100	105	110	115	120	130	140	150	<=95	100	105	110	115	120	130	140	150
ASTM D1863 (No.7 or No.67)	15	2	2	2	2	12	12	16	20	24	2	13	15	18	20	23	27	32	37
	20	2	2	2	2	12	14	18	22	26	12	15	17	19	22	24	29	34	39
	30	2	2	2	13	15	17	21	25	30	14	17	19	22	24	27	32	37	42
	50	12	12	14	16	18	21	25	30	35	17	19	22	25	28	30	36	41	47
	100	14	16	19	21	24	27	32	37	42	21	24	26	29	32	35	41	47	53
	150	17	19	22	25	27	30	36	41	46	23	26	29	32	35	38	44	50	56
ASTM D1863 (No.6)	15	2	2	2	2	12	12	12	15	18	2	2	2	13	15	17	22	26	30
	20	2	2	2	2	12	12	13	17	21	2	2	12	15	17	19	23	28	32
	30	2	2	2	2	12	12	16	20	24	2	12	14	17	19	21	26	31	35
	50	12	12	12	12	14	16	20	24	28	12	15	17	19	22	24	29	34	39
	100	12	12	14	16	19	21	26	30	35	16	18	21	24	26	29	34	39	45
	150	12	14	17	19	22	24	29	34	39	18	21	23	26	29	32	37	43	48

For SI: 1 inch = 25.4 mm; 1 foot = 304.8 mm; 1 mile per hour = 0.447 m/s.

a.Interpolation shall be permitted for mean roof height and parapet height.

b. Basic design wind speed, V, and wind exposure shall be determined in accordance with Section 1609.

c. Where the minimum required parapet height is indicated to be 2 inches (51 mm), a gravel stop shall be permitted and shall extend not less than 2 inches (51 mm) from the roof surface and not less than the height of the aggregate.

d. For Exposure D, add 8 inches (203 mm) to the parapet height required for Exposure C and the parapet height shall not be less than 12 inches (305 mm).

Commenter's Reason: Similar to the provisions for snow drift contained in ASCE 7-16 Section 7.7, where the aggregate is restrained from blow-off by the parapet it is possible that significant aggregate weight can build up in localized areas. With allowable parapet heights of up to 56 inches, the "aggregate drift" that can build up along the parapet can be significantly larger than the roof design dead and live loading. The effect of this weight should be accounted for in the design of the roof and parapet framing.

Cost Impact: The net effect of the public comment and code change proposal will increase the cost of construction
However there will be an increase in safety.

Public Comment# 2129

S23-19

IBC®: 1507.1.1, TABLE 1507.1.1(1), ASTM Chapter 35 (New)

Proposed Change as Submitted

Proponents: Gregory Keeler, representing Owens Corning (greg.keeler@owenscorning.com)

2018 International Building Code

Revise 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, metal roof panels and *photovoltaic shingles* shall conform to the applicable standards listed in this chapter. Underlayment materials required to comply with ASTM D226, D1970, D4869, ~~and~~ D6757 <u>and ASTM WK51913</u> shall bear a label indicating compliance with 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 D1970 and 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 (102 mm) strip of self-adhering polymer modified bitumen membrane complying with ASTM D1970 and 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 (54 m/s) shall be applied over the 4-inch-wide (102 mm) membrane strips.
3. As an alternative, two layers of underlayment complying with ASTM D226 Type II ~~or~~ <u>,</u> ASTM D4869 Type IV<u>, ASTM WK51913</u> shall be permitted to be installed as follows: Apply a 19-inch (483 mm) strip of underlayment parallel with the eave. Starting at the eave, apply 36-inch-wide (914 mm) strips of underlayment ~~felt~~, overlapping successive sheets 19 inches (483 mm). 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. End laps shall be 4 inches (102 mm) 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 (25.4 mm). 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 (mm). Thickness of the outside edge of plastic caps shall be not less than 0.035 inch (mm). The cap nail shank shall be not less than 0.083 inch for ring shank cap nails and 0.091 inch (mm) for smooth shank cap nails. The cap nail shank shall have a length sufficient to penetrate through the roof sheathing or not less than $^3/_4$ inch (19.1 mm) into the roof sheathing.
4. Structural metal panels that do not require a substrate or underlayment.

TABLE 1507.1.1(1)
UNDERLAYMENT TYPES

ROOF COVERING	SECTION	MAXIMUM BASIC DESIGN WIND SPEED, V < 140 MPH	MAXIMUM BASIC DESIGN WIND SPEED, V ≥ 140 MPH
Asphalt shingles	1507.2	ASTM D226 Type I or II ASTM D4869 Type I, II, III or IV ASTM D6757 ASTM WK51913	ASTM D226 Type II ASTM D4869 Type IV ASTM D6757 ASTM WK51913
Clay and concrete tiles	1507.3	ASTM D226 Type II ASTM WK51913 ASTM D2626 Type I ASTM D6380 Class M mineral surfaced roll roofing	ASTM D226 Type II ASTM WK51913 ASTM D2626 Type I ASTM D6380 Class M mineral surfaced roll roofing
Metal panels	1507.4	Manufacturer's instructions	ASTM D226 Type II ASTM D4869 Type IV ASTM WK51913
Metal roof shingles	1507.5	ASTM D226 Type I or II ASTM D4869 Type I, II, III or IV ASTM WK51913	ASTM D226 Type II ASTM D4869 Type IV ASTM WK51913
Mineral-surfaced roll roofing	1507.6	ASTM D226 Type I or II ASTM D4869 Type I, II, III or IV ASTM WK51913	ASTM D226 Type II ASTM D4869 Type IV ASTM WK51913
Slate shingles	1507.7	ASTM D226 Type II ASTM D4869 Type III or IV ASTM WK51913	ASTM D226 Type II ASTM D4869 Type IV ASTM WK51913
Wood shingles	1507.8	ASTM D226 Type I or II ASTM D4869 Type I, II, III or IV ASTM WK51913	ASTM D226 Type II ASTM D4869 Type IV ASTM WK51913
Wood shakes	1507.9	ASTM D226 Type I or II ASTM D4869 Type I, II, III or IV ASTM WK51913	ASTM D226 Type II ASTM D4869 Type IV_ ASTM WK51913
Photovoltaic shingles	1507.17	ASTM D226 Type I or II ASTM D4869 Type I, II, III or IV ASTM D6757 ASTM WK51913	ASTM D226 Type II ASTM D4869 Type IV ASTM D6757_ ASTM WK51913

Add new text as follows:

ASTM

ASTM International
100 Barr Harbor Drive, P.O. Box C700
West Conshohocken PA 19428-2959

WK51913: New Specification for Mechanically Attached Polymeric Roof Underlayment Used in Steep Slope Roofing

Reason: This proposal references an ASTM Work Item for a new ASTM Standard that will apply exclusively to synthetic underlayments. The proposal simply stipulates new performance requirements for products that are already in widespread use.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This proposal references a proposed ASTM Standard that will, for the first time, apply specific performance requirements to synthetic underlayment products that are already in widespread use and will therefore not affect the cost of construction.

Staff Analysis: A review of the standard proposed for inclusion in the code, ASTM WK51913, 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 2, 2019.

S23-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: Proponent requested disapproval. The provided reference standard was incomplete (WK version). (Vote: 14-0)

Assembly Action: **None**

S23-19

Individual Consideration Agenda

Public Comment 1:

Proponents:
Gregory Keeler, representing Owens Corning (greg.keeler@owenscorning.com)

requests As Submitted

Commenter's Reason: The ASTM Work Item is still in process but there is a good chance that we will have a published standard prior to the FAH in October. This will establish a standard that relates directly to synthetic underlayments.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
This proposal only adds a referenced standard that applies directly and specifically to synthetic underlayments. Thus, there is no cost impact.

Staff Analysis: In accordance with Section 3.6.3.1 of ICC Council Policy 28, the new referenced standard ASTM WK51913-2019, *Specification for Mechanically Attached Polymeric Roof Underlayment Used in Steep Slope Roofing,* must be completed and readily available prior to the Public Comment Hearing in order for this public comment to be considered.

Public Comment# 2011

S26-19

IBC®: 1507.8.1

Proposed Change as Submitted

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

2018 International Building Code

Revise as follows:

1507.8.1 Deck requirements. Wood shingles shall be installed on solid or spaced sheathing. Where spaced sheathing is used, sheathing boards shall be not less than 1-inch by 4-inch (25 mm by 102 mm) nominal dimensions and shall be spaced on centers equal to the weather exposure to coincide with the placement of fasteners. <u>The spaced sheathing shall be open to the building interior and shall not be backed with spray foam or other material .</u>

Reason: Shingles installed over spaced sheathiong have underlayment that interweaves with the shingles and is subject to wetting. Although most drying of the underlayment is to the outside, there is some drying that must occur into the building. Spray foam prevents this drying, allowing moisture to accumulate below the shingle. Direct backing of the shingle with insulating foam also raises the temperature of the shingle and accelerates deterioration.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This change is primarily to stop a practice that often occurs as a retrofit. It is not a normal part of any construction process or system, but can sometimes is added to a building interior during modifications.

S26-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The code change proposal was unclear and would most likely increase the cost of construction (contrary to the provided 'cost impact' statement).
(Vote: 14-0)

Assembly Action: **None**

S26-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1507.8.1

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

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1507.8.1 Deck requirements. Wood shingles shall be installed on solid or spaced sheathing. Where spaced sheathing is used, sheathing boards shall be not less than 1-inch by 4-inch (25 mm by 102 mm) nominal dimensions and shall be spaced on centers equal to the weather exposure to coincide with the placement of fasteners. ~~The spaced sheathing shall be open to the building interior and shall not be backed with spray foam or other material.~~ <u>When wood shingles are installed over spaced sheathing the attic shall be ventilated in accordance with Section 1202.2.2 and shall not be backed with materials that prevents the free movement of air on the interior side of the spaced sheathing.</u>

Commenter's Reason: In this case the spaced sheathing serves as the roof deck, so I believe this wording belongs in 1507.8.1. **The alternative placement of this requirement is Chapter 12, but as the issue is having the inside surface of the shingle open to air movement to remove moisture that permeates the wood, the installation and requirement is most likely understood by the roofer. Placing anything that traps moisture in the shingle will shorten the shingles useful life.** Although most drying of the shingles is to the outside, there is some drying that must occur into the building. Any material that prevents the free movement of air on the interior side of the spaced sheathing prevents this drying, allowing moisture to accumulate in the bottom layer of shingles and accelerates wood deterioration. Direct backing of the shingle with insulating material of <u>any type</u> also raises the temperature of the shingle, changes the differential between interior and exterior temperature and accelerates deterioration.

Bibliography: Jerrold E. Winand, H. Michael Barnes, Robert H. Falk; Summer temperatures of roof assemblies using western redcedar, wood-thermoplastic composite, or fiberglass shingles: FOREST PRODUCTS JOURNAL Vol. 54, No. 11

Cost Impact: The net effect of the public comment and code change proposal will decrease the cost of construction
This change is primarily to stop a practice that occurs in new construction and as a retrofit. Insulation and or other barrier products are sometimes added to a building attic interior directly to the interior side of wood shingles. The cost of installation and future problems associated with deterioration of the wood will be eliminated if the material that prevents moisture movement is not installed and the system is free to breathe and dry. So in this case the there is a savings in material and installation cost.

Public Comment# 1522

S27-19

IBC®: 1507.8.6, 1507.9.7

Proposed Change as Submitted

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

2018 International Building Code

Revise as follows:

1507.8.6 Attachment. Fasteners for wood shingles shall be ~~corrosion resistant with~~ hot dipped galvanized box nails, or Type 304 stainless steel box nails. Where used within 15 miles of salt water coasts stainless steel box nails shall be Type 316. Fasteners for Fire retardant treated shingles or pressure impregnated preservative shingles shall be stainless steel type 316. Fasteners shall have a minimum penetration of $^{3}/_{4}$ inch (19.1 mm) into the sheathing. For sheathing less than $^{1}/_{2}$ inch (12.7 mm) in thickness, the fasteners shall extend through the sheathing. Each shingle shall be attached with not fewer than two fasteners.

1507.9.7 Attachment. Fasteners for wood shakes shall be ~~corrosion resistant with~~ hot dipped galvanized, or Type 304 stainless steal box nails. Where used within 15 miles of salt water coasts stainless steel box nails shall be Type 316. Fasteners for fire retardant treated shakes or pressure impregnated preservative treated shakes shall be stainless steel Type 316, Fasteners shall have a minimum penetration of $^{3}/_{4}$ inch (19.1 mm) into the sheathing. For sheathing less than $^{1}/_{2}$ inch (12.7 mm) in thickness, the fasteners shall extend through the sheathing. Each shake shall be attached with not fewer than two fasteners.

Reason: This change is to harmonize the text in 1507.8.6 and 1507.9.7 of the code, with the requirementsin Table 1507.8 and have the same requirements in the IBC as in the IRC.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
The code change proposal will not increase or decrease the cost of construction.This change is primarily to stop a practice that often occurs as a retrofit. It is not a normal part of any construction process or system, but can sometimes is added to a building interior during modifications.

S27-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee was unclear of the term 'saltwater coast' - needs definition.
(Vote: 14-0)

Assembly Action: **None**

S27-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1507.8.6, 1507.9.7

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

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1507.8.6 Attachment. Fasteners for wood shingles shall be hot dipped galvanized box nails, or Type 304 stainless steel box nails. Where used within 15 miles of salt water coasts stainless steel box nails shall be Type 316. Fasteners for Fire retardant treated shingles or pressure impregnated preservative shingles shall be stainless steel type 316. Fasteners shall have a minimum penetration of $^3/_4$ inch (19.1 mm) into the sheathing. For sheathing less than $^1/_2$ inch (12.7 mm) in thickness, furthermore the fasteners shall extend through the sheathing. Each shingle shall be attached with not fewer than two fasteners.

1507.9.7 Attachment. Fasteners for wood shakes shall be hot dipped galvanized, or Type 304 stainless steal box nails. Where used within 15 miles of salt water coasts stainless steel box nails shall be Type 316. Fasteners for fire retardant treated shakes or pressure impregnated preservative treated shakes shall be stainless steel Type 316, Fasteners shall have a minimum penetration of $^3/_4$ inch (19.1 mm) into the sheathing. For sheathing less than $^1/_2$ inch (12.7 mm) in thickness, furthermore the fasteners shall extend through the sheathing. Each shake shall be attached with not fewer than two fasteners.

Commenter's Reason: This change is to harmonize the text in 1507.8.6 and 1507.9.7 of the IBC code, with the requirements in Table 1507.8 and have the same requirements in the IBC as in the 2018 IRC Sections 905.8.6 and 905.8.7. Currently Table 1507.8 requires Type 316 Stainless Steel for coastal areas, and the manufacturers requirements specify that Type 316 Stainless Steel is required within 15 miles of water and when fire retardant or pressure treated shingles or shakes are installed. Although Table 1507.8 is a great pointer to the requirements for installation near salt water it does not cover the requirements for pressure impregnated fire retardant treated and preservative treated wood products. The wording in the proposed change makes the requirements clear and enforceable. The Stainless Steel Institute makes it clear for optimum life of stainless products Type 316 is to be used in areas within 15 miles of water. Jurisdictions where this is not seen as an issue can exclude this requirement for their local code. The salt water coast is easily defined by the local code official. The 15 mile inland distance is easily measured with current online mapping technology.
The issue is based on the fact that wind blown salt spray can travel long distances and deteriorate stainless steel fasteners. Fasteners failure will result in loss of wood shingles or shakes resulting in potential interior water damage.

The IRC and the IBC should be harmonized for the EXACT SAME products. Despite prior code cycle attempts requesting harmonization, the CSSB has yet to see the needed response from ICC.

Consumer safety is impacted when fasteners fail and the roof slides down the deck. Consumer wallets are affected when fasteners fail and the roof starts to leak. Roofing products are unfairly blamed when fasteners fail and the fastener manufacturer will not take responsibility.

The CSSB respectfully asks that ICC harmonize the IBC with the IRC code, using the more stringent fastener requirement ALREADY LISTED in IRC, to ensure consumers and manufacturers are both protected from needless fastener failures. There is an urgent need to ensure this harmonization is implemented and the CSSB finds it difficult to understand why product durability requirements, that are based on field experience, and the recommendations of the Stainless Steel institute have not been supported.

Bibliography: Cedar Shake and Shingle Bureau New Roof Construction Manual
Stainless Steel designer Handbook for Coastal and Salt Corrosion

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
The requirements as noted in this code change were first included in Table 1507.8 in the IBC in 2015, but the text did not match the table. The International Residential Code text in sections 905.8.6 and 905.7.5 beginning in the 2015 version requires that hot dipped galvanized, or stainless fasteners be used. This use of hot dipped galvanized or stainless steel was defined and has been required in the manufacture's installation literature since 2010. (Cedar Shake and Shingle Bureau; New Roof Construction Manual) Therefore code compliance has required the use of this grade of fastener, and therefore the change will not add to the cost of construction. This change is only to note that the table is correct and that the text should match.

The change also helpfully defines how far in from the coast this type of fastener is required and harmonizes the two major construction codes promoted by ICC.

Public Comment# 1518

S29-19

IBC®: 1507.9.1

Proposed Change as Submitted

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

2018 International Building Code

Revise as follows:

1507.9.1 Deck requirements. Wood shakes shall only be used on solid or spaced sheathing. Where spaced sheathing is used, sheathing boards shall be not less than 1-inch by 4-inch (25 mm by 102 mm) nominal dimensions and shall be spaced on centers equal to the weather exposure to coincide with the placement of fasteners. Where 1-inch by 4-inch (25 mm by 102 mm) spaced sheathing is installed at 10 inches (254 mm) on center, additional 1-inch by 4-inch (25 mm by 102 mm) boards shall be installed between the sheathing boards. The spaced sheathing shall be open to the building interior and shall not be backed with spray foam or other material.

Reason: Shakes installed over spaced sheathing have underlayment that interweaves with the shakes and is subject to wetting. Altough most drying of the underlayment is to the outside; there is some drying that must occur into the building. Spray foam prevents the drying, alllowing moisture to accumulate below the shake. Direct backing ot the shake with insulating foam also raises the temperture of the shake and accelerates deterioration.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
The requirements as noted in this code change were first included in Table 1507.8 in the IBC in 2015, but the text did not match the table. The International Residential Code text in sections 905.8.6 and 905.7.5 beginning in the 2015 version requires that hot dipped galvanized, or stainless fasteners be used. This use of hot dipped galvanized or stainless steel was defined and has been required in the manufacture's installation literature since 2010. (Cedar Shake and Shingle Bureau; New Roof Construction Manual) Therefore code compliance has required the use of this grade of fastener, and therefore the change will not add to the cost of construction. This change is only to note that the table is correct and that the text should match.

S29-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: Proponent requested disapproval based on action on S26. Committee action is consistent with action on S26.
(Vote: 14-0)

Assembly Action: **None**

S29-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1507.9.1

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

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1507.9.1 Deck requirements. Wood shakes shall only be used on solid or spaced sheathing. Where spaced sheathing is used, sheathing boards shall be not less than 1-inch by 4-inch (25 mm by 102 mm) nominal dimensions and shall be spaced on centers equal to the weather exposure to coincide with the placement of fasteners. Where 1-inch by 4-inch (25 mm by 102 mm) spaced sheathing is installed at 10 inches (254 mm) on center, additional 1-inch by 4-inch (25 mm by 102 mm) boards shall be installed between the sheathing boards. ~~The spaced sheathing shall be open to the building interior and shall not be backed with spray foam or other material.~~ When wood shakes are installed over spaced sheathing the attic shall be ventilated in accordance with Section 1202.2.2, the shakes shall not be backed with materials that prevents the free movement of air on the interior side of the spaced sheathing.

Commenter's Reason: In this case the spaced sheathing serves as the roof deck, so I believe this wording belongs in Section 1507.8.1. **The alternative placement of this requirement is Chapter 12, but as the issue is having the building interior surface of the shake open to air movement to remove moisture that permeates the wood, the installation and requirement is most likely understood by the roofer. Placing anything that traps moisture in the shake will shorten the shakes useful life.** Although most drying of the shake is to the outside, there is some drying that must occur into the building. Any material that prevents the free movement of air on the interior side of the spaced sheathing prevents this drying, allowing moisture to accumulate in the bottom layer of shakes and accelerates wood deterioration. Direct backing of the shakes with insulating material of any type also raises the temperature of the shake, changes the differential between interior and exterior temperature of the shake and accelerates deterioration.

Bibliography: Jerrold E. Winand, H. Michael Barnes, Robert H. Falk; Summer temperatures of roof assemblies using western redcedar, wood-thermoplastic composite, or fiberglass shingles: FOREST PRODUCTS JOURNAL Vol. 54, No. 11

Cost Impact: The net effect of the public comment and code change proposal will decrease the cost of construction
This change is primarily to stop a practice that occurs in new construction and as a retrofit. Insulation and or other barrier products are sometimes added to a building attic interior directly to the interior side of wood shakes. The cost of installation and future problems associated with deterioration of the wood will be eliminated if the material that prevents moisture movement is not installed and the system is free to breathe and dry. So in this case the there is a savings in material and installation cost.

Public Comment# 1525

S37-19

IBC: 1602.1, 1605.2, 1605.2.1, 1605.3.1, 1605.3.1.1

Proposed Change as Submitted

Proponents: Kelly Cobeen, Wiss Janney Elstner Associates, representing Federal Emergency Management Agency and Applied Technology Council Seismic Code Support Committee (FEMA/ATC SCSC) (KCobeen@wje.com); Jennifer Goupil, representing American Society of Civil Engineers (ASCE) (jgoupil@asce.org); Michael Mahoney, representing Federal Emergency Management Agency (mike.mahoney@fema.dhs.gov)

2018 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 Chapter 12 ~~Section 2.3.6~~ of ASCE 7.

E_h = Effect of horizontal seismic forces as determined in Chapter 12 of ASCE 7.

E_{mh} = Effect of horizontal seismic forces including overstrength as determined in Chapter 12 of ASCE 7.

E_v = Vertical seismic effect applied in the vertical downward direction as in determined in Chapter 12 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/m2) and floor live load.

L_r = Roof live load of 20 psf (0.96 kN/m2) or less.

R = Rain load.

S = Snow load.

T = Cumulative effects of self-straining load forces and effects.

V_{asd}= Allowable stress design wind speed, miles per hour (mph) (km/hr) where applicable.

V = Basic design wind speeds, miles per hour (mph) (km/hr) determined from Figures 1609.3(1) through 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.

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(Lr or S or R) **(Equation 16-2)**

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

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

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

0.9D+ 1.0W+ 1.6H **(Equation 16-5 ~~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.

Add new text as follows:

1605.2.1 Load combinations with seismic load effects. Where a structure is subject to seismic load effects, the following load combinations shall be considered in addition to the basic combinations In Section 1605.2. The most unfavorable effects from seismic loads shall be investigated, where appropriate. The seismic loads need not be considered to act simultaneously with wind loads.

Where the prescribed seismic load effect is combined with the effects of other loads, the following seismic load combinations shall be used:

$1.2(D + F) + 1.0E_v + 1.0\ E_h + f_1L + 1.6H + f_2S$ **(Equation 16-6)**

$0.9(D + F) - 1.0E_v + 1.0E_h + 1.6H$ **(Equation 16-7)**

Where the seismic load effect with overstrength is combined with the effects of other loads, the following seismic load combinations shall be used:

$1.2(D + F) + 1.0E_v + 1.0\ E_{mh} + f_1L + 1.6H + f_2S$ **(Equation 16-8)**

$0.9(D + F) - 1.0E_v + 1.0E_{mh} +1.6H$ **(Equation 16-9)**

where:

f_1 = 1 for places of public assembly live loads in excess of 100 pounds per square foot (4.79 kN/m2), and parking garages; and 0.5 for other live loads.

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

Exceptions:

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

Revise as follows:

~~1605.2.1~~ 1605.2.2 Other loads. Where flood loads, F_a, are to be considered in the design, the load combinations of Section 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 2.3.4 of ASCE 7. Where an ice-sensitive structure is subjected to loads due to atmospheric icing, the load combinations of Section 2.3.3 of ASCE 7 shall be considered.

1605.3 Load combinations using allowable stress design. Load combinations for allowable stress design shall be in accordance with Section 1605.3.1 or 1605.3.2.

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~~ 10)

$D + H + F + L$

(Equation 16-~~9~~ 11)

$D + H + F + (L_r \text{ or } S \text{ or } R)$

(Equation 16-~~10~~ 12)

$D + H + F + 0.75(L) + 0.75(L_r \text{ or } S \text{ or } R)$

(Equation 16-~~11~~ 13)

D + H + F + (0.6W ~~or 0.7E~~) **(Equation 16-~~12~~ 14)**

$D + H + F + 0.75(0.6W) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$

(Equation 16-~~13~~ 15)

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

$0.6D + 0.6W + H$

(Equation 16-~~15~~ 16)

~~*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.~~

Add new text as follows:

1605.3.1.1 Load combinations with seismic load effects. When a structure is subject to seismic load effects, the following load combinations shall be considered in addition to the basic combinations In Section 1605.3.1. The most unfavorable effects from seismic loads shall be investigated, where appropriate, but they need not be considered to act simultaneously with wind loads.

Where the prescribed seismic load effect is combined with the effects of other loads, the following seismic load combinations shall be used:

$D + H + F + 0.7E_v + 0.7E_h$ **(Equation 16-17)**

$D + H + F + 0.525E_v + 0.525E_h + 0.75(L) + 0.75(L_r \text{ or } S \text{ or } R)$ **(Equation 16-18)**

$0.6(D + F) - 0.7E_v + 0.7E_h + H$ **(Equation 16-19)**

Where the seismic load effect with overstrength is combined with the effects of other loads, the following seismic load combinations shall be used:

$D + H + F + 0.7E_v + 0.7E_{mh}$ **(Equation 16-20)**

$D + H + F + 0.525E_v + 0.525E_{mh} + 0.75(L) + 0.75(L_r \text{ or } S \text{ or } R)$ **(Equation 16-21)**

$0.6(D + F) - 0.7E_v + 0.7E_{mh} + H$ **(Equation 16-22)**

Exceptions:

1. In Equations 16-18 and 16-21, 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.
2. 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.
3. In Equation 16-19 and 16-22, 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.

Revise as follows:

~~1605.3.1.1~~ 1605.3.1.2 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~~ <u>1605.3.1.3</u> **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.

Reason: This proposal modifies the load combinations of Sections 1605.2 and 1605.3.1 to more closely align with ASCE 7-16. This editorial change is intended to aid designers by incorporating the ASCE 7 change to more specifically present vertical and horizontal components of seismic loading. See Sections 1605.2.1 and 1605.3.1.1.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This proposal is a clarification of the use of currently existing design provisions. It will not change the cost of construction. It may modestly decrease the cost of design by providing greater clarity.

S37-19

Public Hearing Results

Errata: This proposal includes unpublished errata
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~~

~~3~~2. 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~~3. 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.

Committee Action: **Disapproved**

Committee Reason: Disapproved based on action on S47.
(Vote: 14-0)

Assembly Action: **None**

S37-19

Individual Consideration Agenda

Public Comment 1:

Proponents:
Kelly Cobeen, Wiss Janney Elstner Associates, representing Federal Emergency Management Agency and Applied Technology Council Seismic Code Support Committee (FEMA/ATC SCSC) (kcobeen@wje.com); Michael Mahoney, Federal Emergency Management Agency, representing Federal Emergency Management Agency (mike.mahoney@fema.dhs.gov)

requests As Submitted

Commenter's Reason: Changes to load combinations in ASCE 7-16 make it necessary to modify Chapter 16 load combinations to coordinate. Proposals S37-19 and S47-19 provided alternative ways to implement coordination. S47-19 was determined to be the preferred method, and was supported for approval at the CAH. This public comment for approval as submitted will allow for updating of load combinations, should proposal S47-19 not be approved at the final action hearings. If S47-19 is approved, the proponent intends to withdraw this public comment. See originally submitted statement of reason for further information.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
See original code change proposal for discussion.

S40-19

IBC: 1604.3

Proposed Change as Submitted

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

2018 International Building Code

Revise as follows:

SECTION 1604
GENERAL DESIGN REQUIREMENTS

1604.3 Serviceability. Structural systems and members thereof shall be designed to have adequate stiffness to limit deflections as indicated in Table 1604.3. ~~Drift limits applicable to earthquake loading shall be in accordance with ASCE 7 Chapter 12, 13, 15 or 16, as applicable.~~

Reason: This sentence regarding drift limits does not belong in the section for serviceability. Serviceability and the referenced Table define requirements due to non-lateral loading. The requirements for drift from lateral loads are defined in Section 1613, along with all of the other requirements for lateral loading.
This change is not a technical change in the requirements, rather a clarification of the content of the requirements for Serviceability.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
There is no technical change from this proposal, but a clarification of the appropriate content in this section on serviceability. The drift limit requirements are already included in Section 1613 Earthquake Loading.

S40-19

Public Hearing Results

Committee Action: **As Submitted**

Committee Reason: This change is not a technical change in the requirements, rather a clarification of the content of the requirements for Serviceability.
(Vote: 14-0)

Assembly Action: **None**

S40-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1604.3, 1613.1

Proponents:
Randy Shackelford, representing Simpson Strong-Tie Co. (rshackelford@strongtie.com)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1604.3 Serviceability. Structural systems and members thereof shall be designed to have adequate stiffness to limit deflections as indicated in Table 1604.3.

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 Chapters 11, 12, 13, 15, 17 and 18 of ASCE 7, as applicable. Drift limits applicable to earthquake loading shall be in accordance with ASCE 7 Chapter 12, 13, 15 or 16, as applicable. The *seismic design category* for a structure is permitted to be determined in accordance with Section 1613 or ASCE 7.

Exceptions:

1. Detached one- and two-family dwellings, assigned to *Seismic Design Category* A, B or C, or located where the mapped short-period spectral response acceleration, S_S, is less than 0.4 g.
2. The seismic force-resisting system of wood-frame buildings that conform to the provisions of Section 2308 are not required to be analyzed as specified in this section.
3. Agricultural storage structures intended only for incidental human occupancy.
4. Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances and nuclear reactors.
5. References within ASCE 7 to Chapter 14 shall not apply, except as specifically required herein.

Commenter's Reason: The original code change deleted requirements to check drift limits using ASCE 7 because the requirements were in the wrong section. Drift limits for earthquake are not necesarily servicability requirements, I agree. However this is an important statement that should be included somewhere. So it is proposed that instead of completely deleting the sentence, it be moved back to Section 1613.1.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
There should be no effect on construction, this proposal is just proposing to move the language about checking seismic drift from the Serviceability section to the Earthquake Loads section.

Public Comment# 2157

S44-19

IBC®: TABLE 1604.5

Proposed Change as Submitted

Proponents: Lee Kranz, City of Bellevue, WA, representing Washington Association of Building Officials Technical Code Development Committee (lkranz@bellevuewa.gov)

2018 International Building Code

Revise as follows:

TABLE 1604.5
RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

Portions of table not shown remain unchanged.

RISK CATEGORY	NATURE OF OCCUPANCY
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. <u>Buildings and other structures containing one or more public assembly spaces with an occupant load greater than 300 and a cumulative occupant load of the public assembly spaces of greater than 2,500.</u> •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 50 or more care recipients. •Group I-2, Condition 2 occupancies 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]

a. For purposes of occupant load calculation, occupancies required by Table 1004.5 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 that 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: There are examples of R-1 hotel buildings having multiple large ball rooms or other public assembly spaces but “public assembly” is not the “primary occupancy” as is currently specified in Table 1604.5 so these buildings are classified as Risk Category II. Conversely, there are smaller stand-alone buildings where the primary occupancy is "public assembly" with an occupant load just over 300 that must be designed to the higher Risk Category III even though the total occupant load is much smaller when compared with the example above.
This proposal adds a new criteria for buildings containing at least one assembly space of 300 or more and also having a cumulative occupant load of all assembly spaces of 2,500 or more. This proposal would not include buildings that have multiple assembly spaces, each with an occupant load of less than 300 (like a movie theatre), in Risk Category III unless the total occupant load of the building was greater than 5,000 people. It would also not include a building having multiple assembly spaces, each with an occupant load greater than 300 but the cumulative occupant load of the assembly spaces were less than 2,500, unless the primary occupancy was public assembly or the total occupant load of the building was greater than 5,000 people.

If approved, buildings having one or more assembly rooms with an occupant load of 300 or more and a cumulative occupant load of public assembly spaces of 2,500 or more would be classified as Risk Category III.

Cost Impact: The code change proposal will increase the cost of construction
If approved, more buildings will fall under the Risk Category III which will add cost to construct the building due to a higher importance factor.

S44-19

Public Hearing Results

Committee Action: **As Modified**

Committee Modification:
2018 International Building Code

TABLE 1604.5

RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

RISK CATEGORY	NATURE OF OCCUPANCY
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 one or more public assembly spaces ~~with~~ <u>each having</u> an occupant load greater than 300 and a cumulative occupant load of the public assembly spaces of greater than 2,500. •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 50 or more care recipients. •Group I-2, Condition 2 occupancies 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]

a. For purposes of occupant load calculation, occupancies required by Table 1004.5 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 that 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.

Committee Reason: The proposal provides a reasonable threshold for when to trigger risk category 3. The modification clarifies the intent. (Vote: 13-1)

Assembly Action: **None**

S44-19

Individual Consideration Agenda

Public Comment 1:

IBC®: TABLE 1604.5

Proponents:
Lee Kranz, City of Bellevue, WA, representing Washington Association of Building Officials Technical Code Development Committee (lkranz@bellevuewa.gov)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

TABLE 1604.5
RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

RISK CATEGORY	NATURE OF OCCUPANCY
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 one or more public assembly spaces, each having an occupant load greater than 300 and a cumulative occupant load of ~~the~~ these public assembly spaces of greater than 2,500. •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 50 or more care recipients. •Group I-2, Condition 2 occupancies 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]

a. For purposes of occupant load calculation, occupancies required by Table 1004.5 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 that it can be demonstrated by a hazard assessment in accordance with Section 1.5.3 of ASCE 7 that a release of the toxic, highly toxic or explosive materials is not sufficient to pose a threat to the public.

Commenter's Reason: This public comment is to follow up on a suggestion by one of the ICC Structural Committee members to change "the" to "these" in the proposed criteria for Risk Category III. This change will make it clear that only public assembly spaces with an occupant load of 300 or more are to be included in determining if the cumulative occupant load exceeds 2,500.

Cost Impact: The net effect of the public comment and code change proposal will increase the cost of construction
Very few projects will be impacted by this code change but for those that do the cost will be higher due to the structural design having to meet higher lateral force demands of Risk Category III.

Public Comment# 1158

S52-19

IBC: SECTION 106, 106.1, 106.2, 106.3. 111.5 (New), SECTION 1607 (New), 1607.1 (New), 1607.7.5

Proposed Change as Submitted

Proponents: Ed Kulik, representing ICC Building Code Action Committee (bcac@iccsafe.org)

2018 International Building Code

Delete without substitution:

~~SECTION 106~~ ~~FLOOR AND ROOF DESIGN LOADS~~

~~**[A] 106.1 Live loads posted.** In commercial or industrial buildings, for each floor or portion thereof designed for *live loads* exceeding 50 psf (2.40 kN/m^2), such design *live loads* shall be conspicuously posted by the owner or the owner's authorized agent in that part of each *story* in which they apply, using durable signs. It shall be unlawful to remove or deface such notices.~~

~~**[A] 106.2 Issuance of certificate of occupancy.** A certificate of occupancy required by Section 111 shall not be issued until the floor load signs, required by Section 106.1, have been installed.~~

~~**[A] 106.3 Restrictions on loading.** It shall be unlawful to place, or cause or permit to be placed, on any floor or roof of a building, structure or portion thereof, a load greater than is permitted by this code.~~

Revise as follows:

SECTION 111 CERTIFICATE OF OCCUPANCY

Add new text as follows:

111.5 Live load posted. A certificate of occupancy required shall not be issued until floor load signs, where required by Section 1607.1.1, and maximum weight of vehicles, where required by Section 1607.7.5, have been posted.

SECTION 1607 LIVE LOADS

1607.1 General. Live loads are those loads defined in Chapter 2 of this code.

1607.1.1 Live loads posted. In commercial or industrial buildings, for each floor or portion thereof designed for live loads exceeding 50 psf (2.40 kN/m2), such design live loads shall be posted in a readily visible location by the owner or the owner's authorized agent in the portion of each story in which they apply. It shall be unlawful to remove or deface such notices.

Revise as follows:

1607.7 Heavy vehicle loads. Floors and other surfaces that are intended to support vehicle loads greater than a 10,000-pound (4536 kg) gross vehicle weight rating shall comply with Sections 1607.7.1 through 1607.7.5.

1607.7.5 Posting. The maximum weight of vehicles allowed into or on a garage or other structure shall be posted by the owner or the owner's authorized agent in ~~accordance with Section 106.1.~~ a readily visible location at the vehicle entrance of the building or other approved location. It shall be unlawful to remove or deface such notices.

Reason: The purpose of this code change is to restore the live load posting requirements to Chapter 16. These provisions had been moved to Section 106 by proposal S48-07/08 on the basis that they were administrative requirements rather than technical requirements. The BCAC reviewed the provisions and determined they are in fact technical construction requirements, not administrative enforcement requirements. It is noted they are tied to specific loading requirements in Chapter 16 and are the responsibility of the owner to provide, not the building department. Thus these requirements should be relocated to Chapter 16, with a note left in Section 110 for the building department to verify the loads have been posted. The terminology "commercial or industrial buildings" is existing text that has been in place for several code cycles and B-CAC decided to leave it unchanged. Further, separate provisions have been created for floor live loads and maximum vehicle weights. The reference to a "readily visible" location parallel those for stairway identification signs (Section 1023.9) and signage for public toilet facilities (Section 2902.4 and 2902.4.1). It is noted this signage is not tied to egress or accessibility requirements for the space. Therefore, it is not necessary to require the sign comply with ICC A117.1 or otherwise meet legibility requirements.

This proposal is submitted by the ICC Building Code Action Committee (BCAC). BCAC was established by the ICC Board of Directors in July 2011 to pursue opportunities to improve and enhance assigned International Codes or portions thereof. Since 2017 the BCAC has held 6 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 w ell as any interested party to discuss and debate the proposed changes. Related documentation and reports are posted on the BCAC website at: https://www.iccsafe.org/codes-tech-support/codes/codedevelopment-process/building-code-actioncommittee-bcac.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This relocation of requirements may reduce the cost of construction because all necessary requirements are located in the appropriate Chapter.

S52-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The proposal appears to be intended to be administrative provisions to be located in a design chapter. Section 1607.1.1 language needs revision for clarity on intent.
(Vote: 14-0)

Assembly Action: **None**

S52-19

Individual Consideration Agenda

Public Comment 1:

IBC®: SECTION, [A], 1607.7, 1607.7.5

Proponents:
Ed Kulik, representing ICC Building Code Action Committee (bcac@iccsafe.org)

requests As Modified by Public Comment

Replace as follows:

2018 International Building Code

SECTION ~~106~~
~~FLOOR AND ROOF DESIGN LOADS~~

[A] ~~106.1 Live loads posted.~~ ~~In commercial or industrial buildings, for each floor or portion thereof designed for *live loads* exceeding 50 psf (2.40 kN/m^2), such design *live loads* shall be conspicuously posted by the owner or the owner's authorized agent in that part of each *story* in which they apply, using durable signs. It shall be unlawful to remove or deface such notices.~~

[A] ~~106.2 Issuance of certificate of occupancy.~~ ~~A certificate of occupancy required by Section 111 shall not be issued until the floor load signs, required by Section 106.1, have been installed.~~

[A] ~~106.3 Restrictions on loading.~~ ~~It shall be unlawful to place, or cause or permit to be placed, on any floor or roof of a building, structure or portion thereof, a load greater than is permitted by this code.~~

1607.7 Heavy vehicle loads. Floors and other surfaces that are intended to support vehicle loads greater than a 10,000-pound (4536 kg) gross vehicle weight rating shall comply with Sections 1607.7.1 through 1607.7.5.

1607.7.5 Posting. The maximum weight of vehicles allowed into or on a garage or other structure shall be posted in a readily visible location at the vehicle entrance of the building or other approved location by the owner or the owner's authorized agent ~~in accordance with Section 106.1.~~ It shall be unlawful to remove or deface such notices.

Commenter's Reason: There were concerns expressed by the committee with the current text:
- The requirement in Section 106.1 is too low of a weight load.
- This signage requirement is lost/hidden in Chapter 1.
- Inspectors never see it provided;
- If required there is no mechanism to make sure it is maintained.
- There is no specifics on what to do with the information provided.

The BCAC looked at increasing the weight requirements, but instead decided with the concerns raised that this requirement is better not in the code. Therefore, this public comment seeks to delete this signage requirement from Chapter 1.

Regarding the current language for garage posting currently in 1607.7.5. The current Section 106.1 referenced did not provide sufficient guidance. The committee had suggestions to improve the order of the new proposed language for additional clarity. Where this sign is required is very limited since it is only needed in parking areas inside a building for heavy vehicles.

This public comment is submitted by the ICC Building Code Action Committee (BCAC). BCAC was established by the ICC Board of Directors in July 2011 to pursue opportunities to improve and enhance assigned International Codes or portions thereof. Since 2017 the BCAC has held 6 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 w ell as any interested party to discuss and debate the proposed changes. Related documentation and reports are posted on the BCAC website at: https://www.iccsafe.org/codes-tech-support/codes/codedevelopment-process/building-code-actioncommittee-bcac.

Cost Impact: The net effect of the public comment and code change proposal will decrease the cost of construction
This will eliminate the requirements for some signs.

Public Comment# 1183

S60-19

IBC®: 1603.1.1, TABLE 1607.1

Proposed Change as Submitted

Proponents: Paul Armstrong, MHI, representing MHI

2018 International Building Code

Revise as follows:

1603.1.1 Floor live load. The uniformly distributed, concentrated and impact floor live load used in the design shall be indicated for floor areas. Use of live load reduction in accordance with Section 1607.11 shall be indicated for each type of live load used in the design.

For Group S storage warehouses the floor shall be designed for the maximum uniformly distributed or concentrated live load. In areas with storage rack, the concentrated live load shall be designed for a minimum concentrated load of 5,000 lbs (2268 kg) where the clear ceiling height is 15 feet (4572 mm) minimum. The concentrated load shall be increased an additional 2,500 lbs (1123 kg) for each additional 5 feet (1524 mm) clear ceiling height or portion thereof, over 15 feet (4572 mm). The concentrated loads shall be located on a 4 foot by 8 foot (1219 mm by 2438 mm) grid over the floor area with storage racks.

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, *L0*, AND MINIMUM CONCENTRATED LIVE LOADS[g]

Portions of table not shown remain unchanged.

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (pounds)
31. Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		See Section 1603.1.1
Heavy	250[n]	
Light	125[n]	

Reason: Many warehouse structures in Use Group S have storage rack located in them resulting in localized loading on the concrete floor slab. We wish to bring this to the attention of the registered design professional of the building when they are designing the new concrete floor slab if the actual floor loads are not known. New warehouse buildings are becoming taller and the 125 psf or 250 psf floor loads are no longer adequate when designing the concrete floor slab.

Cost Impact: The code change proposal will increase the cost of construction
While this might increase the cost of construction in warehouses slightly, it will serve to decrease the cost when evaluating existing warehouse slabs.

S60-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: Proposal and wording needs to be vetted through ASCE 7. Proposal needs clarification of threshold.
(Vote: 14-0)

Assembly Action: **None**

S60-19

Individual Consideration Agenda

Public Comment 1:

Proponents:
Paul Armstrong, MHI, representing MHI (paul.armstrong@pacodeservices.com)

requests As Submitted

Commenter's Reason: The Engineering Committee of the Rack Manufacturer's Institute has submitted this Public Comment to recognize new loading that is imposed by rack structures. It is understood that ultimately this loading criteria should reside in ASCE 7 but the next edition is in 2022 and that process is only just underway. The committee commits to remove this provision in the IBC once it is published in ASCE 7. Please support this Public Comment so that such storage warehouse floor systems can be properly designed.

Cost Impact: The net effect of the public comment and code change proposal will increase the cost of construction
However, this will decrease the cost of construction when applying this criteria in existing warehouses due to the floors being adequately designed for the new rack loads.

Public Comment# 2000

S72-19

IBC®: 1607.13.5.3, 1607.13.5.4, 1607.13.5.5

Proposed Change as Submitted

Proponents: Joseph H. Cain, Solar Energy Industries Association (SEIA), representing Solar Energy Industries Association (SEIA) (JoeCainPE@gmail.com)

2018 International Building Code

Revise as follows:

~~**1607.13.5.2.1**~~ **1607.13.5.3 Photovoltaic panels installed on open grid roof structures.** Structures with open grid framing and without a roof deck or sheathing supporting photovoltaic panel systems shall be designed to support the uniform and concentrated roof live loads specified in Section 1607.13.5.1, except that the uniform roof live load shall be permitted to be reduced to 12 psf (0.57 kN/m^2).

~~**1607.13.5.3**~~ **1607.13.5.4** ~~**Photovoltaic panels or modules installed as an independent structure.**~~ **Ground-mounted photovoltaic (PV) panel systems.** ~~Solar photovoltaic panels or modules~~ Ground-mounted *photovoltaic (PV) panel systems* that are independent structures and do not have accessible/occupied space underneath are not required to accommodate a roof photovoltaic live load~~, provided that the area under the structure is restricted to keep the public away~~. 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 1607.13.5.1 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 1607.13.5.1.~~

~~**1607.13.5.4**~~ **1607.13.5.5 Ballasted photovoltaic panel systems.** Roof structures that provide support for ballasted *photovoltaic panel systems* shall be designed, or analyzed, in accordance with Section 1604.4; checked in accordance with Section 1604.3.6 for deflections; and checked in accordance with Section 1611 for ponding.

Reason: In development of the 2018 IBC, new Section 1607.13.5.2.1 was created to use language similar to ASCE 7-16. As the second paragraph of Section 1607.13.5.3 was intended to state the requirements for the same type of structure, Section 1607.13.5.3 is now redundant and outdated in the 2018 IBC. This proposal strikes out the redundance second paragraph.
The first paragraph of Section 1607.13.5.3 is intended to state the requirements for ground-mounted PV systems, so is now updated to use that term.

Sections are re-numbered for better flow, such that:

1607.13.5.2 is for rooftop-mounted PV systems

1607.13.5.3 is for overhead structures with open-grid framing(renumbered from 1607.13.5.2.1)

1607.13.5.4 is for ground-mounted PV systems (renumbered from 1607.13.5.3)

1607.13.5.5 is for ballasted rooftop PV systems (renumbered from 1607.13.5.4)

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This proposal clarifies the language, and will not increase or decrease cost of construction.

S72-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee expressed concerns that this change had not yet been vetted through ASCE 7.
(Vote: 12-2)

Assembly Action: **None**

Individual Consideration Agenda

Public Comment 1:

Proponents:
Joseph H. Cain, P.E., Solar Energy Industries Association (SEIA), representing Solar Energy Industries Association (SEIA) (JoeCainPE@gmail.com)

requests As Submitted

Commenter's Reason: In development of the 2018 IBC, new Section 1607.13.5.2.1 was created to use language similar to ASCE 7-16. As the second paragraph
of Section 1607.13.5.3 was intended to state the requirements for the same type of structure, Section 1607.13.5.3 is now redundant and outdated in the 2018 IBC.
This Proposal S72-19 and this Public Comment seeks to correct a mistake that exists in the 2018 IBC, by striking out the redundant second paragraph.

NOTE: Subsection 1607 is titled "Live Loads." This proposal and public comment seek to clarify live loads on PV systems, consistent with the original intent of previous Subsection 1607.12.5 of the 2015 IBC, as developed by NCSEA in collaboration with SEAOC.

The first paragraph of Section 1607.13.5.3 is intended to state the requirements for ground-mounted PV systems, so is now updated to use that term.

Sections are re-numbered for better flow, such that:
1607.13.5.2 is for rooftop-mounted PV systems
1607.13.5.3 is for overhead structures with open-grid framing(renumbered from 1607.13.5.2.1)
1607.13.5.4 is for ground-mounted PV systems (renumbered from 1607.13.5.3)

Section 1607.13.5.4 simply replaces:

"Solar photovoltaic panels or modules that are independent structures and do not have accessible/occupied space underneath ..."

with the common industry term "ground mount" accompanied by the defined term "photovoltaic panel systems":

"Ground-mounted photovoltaic (PV) panel systems that are independent structures and do not have accessible/occupied space underneath ..."

1607.13.5.5 is for ballasted rooftop PV systems (renumbered from 1607.13.5.4, with no other changes)

When viewed as clean text, the reader should see that the proposed language will clarify the provisions applicable to overhead structures and ground-mounted systems with no occupancy beneath. These changes are editorial only, and do not change the fundamental provisions for live load for these structures.

1607.13.5.3 Photovoltaic panels installed on open grid roof structures. Structures with open grid framing and without a roof deck or sheathing supporting photovoltaic panel systems shall be designed to support the uniform and concentrated roof live loads specified in Section 1607.13.5.1, except that the uniform roof live load shall be permitted to be reduced to 12 psf (0.57 kN/m2).

1607.13.5.4 Ground-mounted photovoltaic (PV) panel systems. Ground-mounted photovoltaic (PV) panel systems that are independent structures and do not have accessible/occupied space underneath are not required to accommodate a roof photovoltaic live load. Other loads and combinations in accordance with Section 1605 shall be accommodated.

1607.13.5.5 Ballasted photovoltaic panel systems. Roof structures that provide support for ballasted photovoltaic panel systems shall be designed, or analyzed, in accordance with Section 1604.4; checked in accordance with Section 1604.3.6 for deflections; and checked in accordance with Section 1611 for ponding.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
This proposal clarifies the language, and will not increase or decrease cost of construction.

Public Comment# 2147

S75-19

IBC: 1609.1.1, Chapter 35 AASHTO (New)

Proposed Change as Submitted

Proponents: Ray Minor, representing Self (ray.minor@hapco.com); Jay Baumgartner, Valmont Industries, representing Valmont Industries (jay.baumgartner@valmont.com)

2018 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. The type of opening protection required, the basic design wind speed, *V*, and the exposure category for a site is permitted to be determined in accordance with Section 1609 or ASCE ~~7.~~ 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 that 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. Luminaire support structures designed in accordance with AASHTO LTS-6. Athletic field lighting structures taller than 55' shall be designed to meet the 50 year design life wind load and the Fatigue Importance Category I Natural Wind Gust requirements of AASHTO LTS-6.

The wind speeds in Figures 1609.3(1) through 1609.3(8) are basic design wind speeds, *V*, and shall be converted in accordance with Section 1609.3.1 to *allowable stress design* wind speeds, V_{asd}, when the provisions of the standards referenced in Exceptions 4 , 5 and ~~5~~ 7 are used.

Add new standard(s) as follows:

AASHTO

Association of State Highway and
Transportation Officials
4444 North Capitol Street, NW Suite 249
Washington DC 20001
US

LTS-6-2013: Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals

Reason: The AASHTO LTS-6 specification is based on much research and many years of experience in using primarily pole type structures to support signs, luminaires and traffic signals along roadways. These types of structures are also used for many non-roadway applications such as sports lighting and parking lot lighting which may come under the jurisdiction of the IBC. The AASHTO LTS-6 wind pressure calculations are based on ASCE 7. ASCE 7-16 C29.4 states "For the design of structural supports for highway signs, luminaires and traffic signals, see AASHTO LTS-6 (AASHTO 2013).". The AASHTO LTS-6 contains provisions for the fatigue design of structural supports for signs, luminaires and traffic signals that are exclusive to AASHTO. Several athletic field lighting structures that would not meet these fatigue requirements have failed (See Consumer Product Safety Commision link in Bibliography and Stadium Pole Failures file in Attachments). These failures most likely would not have occurred if the poles had been designed to meet the natural wind gust fatigue requirements of the AASHTO LTS-6 specification.

Bibliography: United States Consumer Product Safety Commission report "Recall to Repair: Whitco Company LP Stadium Light Poles Can Fall Over, Posing Risk of Serious Injury and Death" link:
https://www.cpsc.gov/Recalls/2010/recall-to-repair-whitco-company-lp-stadium-light-poles-can-fall-over-posing-risk-of

Cost Impact: The code change proposal will increase the cost of construction
The sports lighting poles that failed would not meet the transverse plate minimum thickness requirement of AASHTO LTS-6 Paragraph 5.14.3 which likely contributed to the failures. These poles would be identified as high level luminaire supports in LTS-6 Paragraph 1.4.2 which would require them to be designed for fatigue according to LTS-6 Paragraph 11.3. Fatigue design specifications of LTS-6 Section 11 generally requires heavier poles than designing for maximum wind speed alone.

Staff Analysis: A review of the standard proposed for inclusion in the code, AASHTO LTS-6-2013, 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 2, 2019.

Public Hearing Results

Committee Action: Disapproved

Committee Reason: The committee expressed concerns as to the following:
1. why reference the older ASCE 7-05?

2. questioned the background for using a 50 year design life

3. format - many requirements are provided in the exceptions

(Vote: 12-2)

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

IBC®: 1609.1.1 (New)

Proponents:
Jay Baumgartner, representing Valmont Industries (jay.baumgartner@valmont.com)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1609.1.1 Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapters 26 to 30 of ASCE 7. The type of opening protection required, the basic design wind speed, *V*, 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 that 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. Luminaire support structures designed in accordance with AASHTO LTS-6. Athletic field lighting structures taller than 55' shall be designed to meet ~~the 50 year design life wind load and the Fatigue Importance Category I Natural Wind Gust~~ Fatigue Importance Category I of the high-mast lighting tower fatigue requirements of AASHTO LTS-6.

The wind speeds in Figures 1609.3(1) through 1609.3(8) are basic design wind speeds, *V*, and shall be converted in accordance with Section 1609.3.1 to *allowable stress design* wind speeds, V_{asd}, when the provisions of the standards referenced in Exceptions 4, 5 and 7 are used.

Commenter's Reason: The inclusion of the AASHTO LTS-6 Specifications will provide safer lighting structures as well as a more refined design analysis. In accordance with Section 1609.1.1, the wind speed maps from Figures 1609.3(1) through 1609.3(8) shall be used and converted in accordance with Section 1609.3.1 for use with AASHTO. In addition, the Risk Category II wind speeds (MRI = 700 Years) are generally equivalent to the basic wind speeds found in AASHTO LTS-6 (50-year return period) based upon research referenced by ASCE 7 after moving the wind load factor for the strength design approach.
Adding the AASHTO LTS-6 Specifications will also bring consistency to Section 1609.1.1, where similar non-building structures are already addressed. Section 1609.1.1 Exceptions 4 and 5 are used for the design of flagpoles (NAAMM FP 1001) and antenna-supporting structures (TIA-222), respectively. ASCE 7-16 C29.4 also states "It is not the intent of this standard to exclude the use of other recognized literature for the design of special structures ... For the design of flagpoles, see NAAMM (2007). For the design of structural supports for highway signs, luminaires, and traffic signals, see AASHTO LTS-6 (AASHTO 2013)." I therefore urge approval of this proposal.

Cost Impact: The net effect of the public comment and code change proposal will increase the cost of construction
This proposal would likely only increase the cost of construction for those structures requiring a fatigue design.

Public Comment# 1434

S83-19

IBC: 1703.1.3.1 (New)

Proposed Change as Submitted

Proponents: Stephen Szoke, American Concrete Institute, representing American Concrete Institute (steve.szoke@concrete.org); Scott Campbell, National Ready Mixed Concrete Association, representing National Ready Mixed Concrete Association (scampbell@nrmca.org)

2018 International Building Code

1703.1.3 Personnel. An *approved agency* shall employ experienced personnel educated in conducting, supervising and evaluating tests and *special inspections*.

Add new text as follows:

1703.1.3.1 **Structural concrete special inspector.** Individuals with current credentials demonstrating that the requirements of ACI Concrete Construction Special Inspector or ICC Reinforced Concrete Special Inspector have been satisfied shall be permitted to act as special inspectors for structural concrete construction.

Reason: This code change proposal provides the criteria for personnel to be considered qualified to conduct special inspections of structural concrete. The American Concrete Institute Committee C630 - Construction Inspector Certification has developed a rigorous program to certify individuals as qualified to perform special inspection of concrete construction. This code change proposal does not alter any existing criteria of other individuals qualified as special inspectors, but adds provisions for individuals who are ACI or ICC certified concrete construction special inspectors to be permitted to satisfy the code criteria as special inspectors for concrete construction. This proposal provides the criteria, but does not require individuals to be certified as an ACI Concrete Construction Special Inspector. The ACI requirements are provided in the attached file, cpp-6301-15.pdf, or may be found at: https://www.concrete.org/Portals/0/Files/PDF/cpp_6301-15.pdf.
Jurisdictions are adding these requirements to their codes. As a model code, this requirement should be inlcuded in the IBC to assist the jurisidcitons in having the language properly incorporated into their respective codes. For example, the Georgia Building Code now incudes certified inspectors. See pages 12 through 15 of the attached file, 2014-ibcamendments.pdf.

The American Concrete Institute. as a professional society whose mission includes working to facilitate the use and adoption of current concrete technology to assure the desired performance for the benefit of the public, encourages the committee to approve of this code change as submitted.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
The code change allows current practice for selection of individuals or entities to perform special inspection. The change adds qualifications for individuals to assist the building code official in approving such individuals and provides a degree of confidence that special inspections will be properly conducted.

S83-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: While the committee agreed that training of special inspectors is important, they expressed concerns that the proposal is in the wrong section of the code.
(Vote: 10-4)

Assembly Action: **None**

S83-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1704.2.1.1 (New), ACI Chapter 35 (New)

Proponents:

Stephen Szoke, representing American Concrete Institute (steve.szoke@concrete.org)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

~~1703.1.3.1~~ 1704.2.1.1 Structural concrete special inspector. Individuals satisfying the requirements of Section 1.6 of ACI 311.7 as Concrete Special Inspector with current credentials demonstrating that the requirements of ACI Concrete Construction Special Inspector ~~or ICC Reinforced Concrete Special Inspector have been satisfied~~ shall be permitted to act as special inspectors for structural concrete construction.

ACI

American Concrete Institute
38800 Country Club Drive
Farmington Hills MI 48331

311.7-18: Specification for Inspection of concrete Construction

Commenter's Reason: This public comment provides the criteria for personnel to be considered qualified to conduct special inspections of structural concrete. The American Concrete Institute Committee 311 has developed language that identifies individual qualified to conduct special inspection of structural concrete construction. These provisions are identified in Section 1.6 Qualifications of ACI 311.7 Inspection Services Specification for Cast-in-Place Concrete Construction. This proposal does not alter any existing criteria of other individuals qualified as special inspectors, but adds provisions for individuals certified as concrete construction special inspectors to be permitted to satisfy the code criteria as special inspectors for concrete construction. This proposal increases the pool of individuals that may be identified as qualified to conduct such inspections. These qualifications create an improved confidence that the individuals conducting inspections have had the appropriate training and demonstrated competence in conducting special inspections of structural concrete construction.
During the Committee Action Hearings *the committee agreed that training of special inspectors is important, they expressed concerns that the proposal is in the wrong section of the code*. This modification relocates the provision from section 1703.3.1.3 to Section 1704.2.1 Special inspector qualifications.

Bibliography: 311.7-18: Specification for Inspection of Concrete Construction
CPP 630.1-15 Certification Policies for Concrete Construction Special Inspector

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
The proposal allows current practice for selection of individuals or entities to perform special inspection while including qualifications for additional individuals. This increases the pool of qualified individuals and is expected to decrease costs.

Public Comment# 1272

S86-19

IBC®: 1704.2

Proposed Change as Submitted

Proponents: Terry Kozlowski, representing Southern Nevada Chapter; Nenad Mirkovic, representing City of Las Vegas; Amanda Moss, representing SN-ICC Member; Cassidy Wilson, representing SN-ICC Member

2018 International Building Code

Revise as follows:

1704.2 Special inspections and tests. Where application is made to the *building official* for construction as specified in Section 105, the owner or the owner's authorized agent, other than the contractor, shall employ one or more *approved agencies* to provide *special inspections* and tests during construction on the types of work specified in Section 1705 and identify the *approved agencies* to the *building official*. These *special inspections* and tests are in addition to the inspections by the *building official* that are identified in Section 110.

Exceptions:

1. *Special inspections* and tests are not required for construction of a minor nature or as warranted by conditions in the jurisdiction as *approved* by the *building official*.
2. Unless otherwise required by the *building official*, *special inspections* and tests are not required for detached 1 & 2 family dwellings and Group U ~~occupancies that are accessory to a residential occupancy including,~~ occupancy accessory structures, 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 2211.1.2 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.

Reason: Local inspectors have previously been required to inspect the shear walls and other details needed to resist lateral forces. This proposal will provide the building official with discretion in the application of special inspection requirements for residential construction and would allow the local building inspector to inspect detached 1 & 2 family dwellings and accessory structures.

Cost Impact: The code change proposal will decrease the cost of construction
This proposal will decrease the cost of construction by eliminating the requirement for special inspection.

S86-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee expressed concerns that the proposal should not eliminate all detached 1 and 2 family dwellings from special inspections.
(Vote: 12-1)

Assembly Action: **None**

S86-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1704.2

Proponents:
Gary Ehrlich, representing National Association of Home Builders (gehrlich@nahb.org)

requests As Modified by Public Comment

Replace as follows:

2018 International Building Code

1704.2 Special inspections and tests. Where application is made to the *building official* for construction as specified in Section 105, the owner or the owner's authorized agent, other than the contractor, shall employ one or more *approved agencies* to provide *special inspections* and tests during construction on the types of work specified in Section 1705 and identify the *approved agencies* to the *building official*. These *special inspections* and tests are in addition to the inspections by the *building official* that are identified in Section 110.

Exceptions:

1. *Special inspections* and tests are not required for construction of a minor nature or as warranted by conditions in the jurisdiction as *approved* by the *building official*.
2. Unless otherwise required by the *building official*, *special inspections* and tests are not required for Group U occupancies that are accessory to a residential occupancy including, but not limited to, those listed in Section 312.1.
3. *Special inspections* and tests are not required for portions of structures designed and constructed in accordance with the cold-formed steel light-frame construction provisions of Section 2211.1.2 or the conventional light-frame construction provisions of Section 2308.
4. *Special inspections* and tests are not required for portions of one- and two-family dwellings and townhouses and their accessory structures designed in accordance with Section R301.1.3 of the *International Residential Code* .

~~4.~~ 5. The contractor is permitted to employ the *approved agencies* where the contractor is also the owner.

Commenter's Reason: The purpose of this public comment is to replace the initial proposal with a more targeted exception for engineered portions of dwellings otherwise complying with the IRC.
Section R301.1.3 of the IRC allows for design in accordance with accepted engineering practice for portions of a detached dwelling, townhouse, or accessory structure that generally falls within the scope of the IRC but that contains structural elements which exceed an individual limit of the IRC.

Permits for such dwellings are generally issued and inspections performed under a jurisdiction's policies and procedures for residential structures. However, NAHB members have reported some building departments requiring special inspections for engineered components of a dwelling otherwise and constructed under the prescriptive structural provisions of the IRC.

Special inspections were originally conceived to address elements and systems of construction for commercial buildings that due to their unique nature or complexity needed a level of review beyond the standard building department plan review and inspections. Typical elements of a dwelling where engineering is frequently performed include tall foundation walls, structural composite lumber beams and posts, steel framing over a basement, and truss roof assemblies. Some engineers have argued these systems can be complex and require a special inspector. However, in most typical dwellings, these elements are still designed using common material strengths, standard configurations, typical connection types, and standard construction details.

Estimates obtained from the Home Innovation Research Labs suggest a minimum cost to a homeowner for a special inspection of one component (e.g. a long-span truss) is on the order of $530. The minimum cost to a homeowner for a more extensive set of special inspections that could include the components of the wind or seismic force-resisting system could be as much as $900. There is no need to burden homeowners with hundreds of dollars of additional costs simply because a typical size dwelling just happens to incorporate an engineered foundation wall, roof trusses, a few LVL's, or a few steel beams and posts.

Cost Impact: The net effect of the public comment and code change proposal will decrease the cost of construction
As noted in the reason statement, cost studies by Home Innovation Research Lab show a homeowner could save between $500 and $900 by avoiding the need for an unnecessary special inspection.

Public Comment# 1140

S87-19

BC: 1704.2.6 (New), ACI Chapter 35 (New)

Proposed Change as Submitted

Proponents: Stephen Szoke, American Concrete Institute, representing American Concrete Institute (steve.szoke@concrete.org); Scott Campbell, National Ready Mixed Concrete Association, representing National Ready Mixed Concrete Association (scampbell@nrmca.org)

2018 International Building Code

Add new text as follows:

1704.2.6 Concrete tests. Field and laboratory technicians qualifications shall comply with ACI 311.6.

Add new standard(s) as follows:

ACI

American Concrete Institute
38800 Country Club Drive
Farmington Hills MI 48331

ACI 311.6-18: Specification for Ready Mixed Concrete Testing Services

Reason: Proper sampling, specimen preparation and acceptance testing of concrete delivered to construction projects is crucial for assuring proper performance of structural concrete. Inaccurate test results and the negative implications on the performance of concrete occur far too frequently. When field testing, preparation of samples and laboratory testing are not conducted properly there may be significant expenses and delays added to the cost of construction, such as extracting cores of hardened concrete to verify concrete strength. Improper sampling, preparation and testing often cause project delays, further increasing costs.
On many projects the qualifications for technicians are included in the construction documents. There is a need to assure cast-in-place concrete is properly sampled, prepared and tested. Cast-in-place concrete is one of the few building materials provided to the construction site in a condition other than its final state. Verification of properties should only be performed by qualified individuals.

Local jurisdictions have already begun to address this concern. In 2014 the Georgia Building Code included an amendment to the IBC which added ACI Concrete Field Testing Technician with Grade 1 certification: https://dca.ga.gov/sites/default/files/2014_ibcamendments.pdf. In 2018 the Georgia Building Code included another amendment to the IBC which added American Concrete Institute (ACI) Strength Testing Technician: https://dca.ga.gov/sites/default/files/2018_ibcamendments.pdf. This demonstrates the need to more clearly communicate the necessary qualifications for technicians conduction sampling, specimen preparation and testing of concrete.

ACI, a technical professional society, recommends that the committee approve this code change proposal as submitted to 1) improve the quality assurance processes for structural concrete, 2) reduce project cost increases due to inappropriate sampling, preparation and testing, 3) reduce the frequency of related construction delays, and 4) help assure that the concrete being used in structural elements will provide the life safety and property protection necessary to satisfy the intent of the code.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
There is no cost increase for this code change proposals, as for most projects these requirements are included in the contract documents between the owners, designers, and contractors. This code change proposal helps to assure that these requirements are included for structural concrete.

Staff Analysis: A review of the standard proposed for inclusion in the code, ACI 311.7-18, 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 2, 2019.

S87-19

Public Hearing Results

Committee Action: Disapproved

Committee Reason: The committee felt that the proponent did not clearly justify why the proposal is needed in the IBC. The proposal would greatly benefit from adding 'or equivalent' during the public comment phase. (Vote: 12-2)

Assembly Action: None

S87-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1704.2.6 (New), ACI Chapter 35 (New)

Proponents:
Stephen Szoke, representing American Concrete Institute (steve.szoke@concrete.org)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1704.2.6 Concrete tests. <u>Unless otherwise approved, individuals conducting field tests of structural concrete shall satisfy the requirements of ACI 311.6 Section 1.2.1.1 Field technician. Unless otherwise approved, individuals conducting laboratory tests of structural concrete elements shall satisfy the requirements of ACI 311.6 section 1.2.1.2 Laboratory technician.</u> ~~Field and laboratory technicians qualifications shall comply with ACI 311.6.~~

ACI

American Concrete Institute
38800 Country Club Drive
Farmington Hills MI 48331

ACI 311.6-18: Specification for Ready Mixed Concrete Testing Services

Commenter's Reason: This proposal adds criteria to the building code to help assure proper sampling, specimen preparation and acceptance testing of concrete delivered to construction projects. This is crucial for assuring proper performance of structural concrete. Inaccurate test results and the negative implications on the performance of concrete occur far too frequently and add to the cost of construction by requiring sampling and testing of cores or other verification of concrete properties. Too, there are additional costs related to delays in construction.
The language in ACI 311.6 is:

1.2.1.1 Field technician—Technicians conducting field tests of concrete shall be certified as ACI Concrete Field Testing

Technician – Grade I, unless otherwise specified.

1.2.1.2 Laboratory technician—Technicians conducting laboratory testing shall be certified as ACI Concrete Laboratory

Testing Technician – Level 1 or ACI Concrete Strength Testing Technician, unless otherwise specified.

During the Committee Action Hearings, *the committee felt that the proponent did not clearly justify why the proposal is needed in the IBC. The proposal would greatly benefit from adding 'or equivalent' during the public comment phase.* The language in the modification specifically includes “unless otherwise specified” to permit testing by any individuals approved by the building official. This language is deemed to better capture the intent of the committee than requiring an equivalent to the specific ACI requirements.

Bibliography: <u>ACI 311.6-18: Specification for Ready Mixed Concrete Testing Services</u>

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
Testing is already required. This modifications sets criteria for technician qualifications.

Public Comment# 1273

S90-19

IBC®: 1704.6

Proposed Change as Submitted

Proponents: Gregory Robinson, representing National Council of Structural Engineers Associations (NCSEA) (grobinson@lbyd.com)

2018 International Building Code

Revise as follows:

1704.6 Structural observations. Where required by the provisions of Section 1704.6.1, 1704.6.2 or 1704.6.3, the owner or the owner's authorized agent shall employ a *registered design professional* to perform structural observations. The structural observer shall visually observe representative locations of structural systems, details, and load paths for general conformance to the design intent as defined in the approved construction documents. 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.

Reason: The definition of structural observations in Chpater 2 is vague and disconnected from the requirements in Chapter 17. As a result, the various roles that form a comprehensive program of tests and inspections often get confused, and application is inconsistent. Including the proposed description in Chapter 17 provides a clearer understanding of what an observer is expected to "visually" observe - systems, details, and load paths. It is also intended to help address a widespread perception of overlap between special inspections and structural observation.
Special inspections are very detailed inspections of smaller components. They require certification and specialized training to perform, but they don't necessarily require an understanding of how systems are designed to function as part of the overall building.

On the other hand, structural observations are broad, general, visual overviews of a bigger picture. Broad knowledge of structural design issues and specific knowledge of their application to the project is necessary, but observations do not strictly adhere to a standard written procedure like special inspections do.

The distinct levels of oversight are complimentary, but intended to address different aspects of quality assurance.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
Clarification only. No additional cost is anticipated.

S90-19

Public Hearing Results

Committee Action: **As Submitted**

Committee Reason: After much discussion, the committee acknowledged that the proposal was a reasonable addition to explain to all parties what constitutes a 'structural observation'.
(Vote: 9-5)

Assembly Action: **None**

S90-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1704.6

Proponents:
Jenifer Gilliland, representing Seattle Department of Construction and Inspections (SDCI) (jenifer.gilliland@seattle.gov); Jonathan Siu, representing City of Seattle Department of Construction and Inspections (jon.siu@seattle.gov)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1704.6 Structural observations. Where required by the provisions of Section 1704.6.1, 1704.6.2 or 1704.6.3, the owner or the owner's authorized agent shall employ a *registered design professional* to perform structural observations. The structural observer shall visually observe representative locations of structural systems, details, and load paths for general conformance to ~~the design intent as defined in~~ the approved construction documents. 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.

Commenter's Reason:
This public comment is intended to address an issue raised by one of the members of the Structural Committee during the hearings in May.

The current proposal requires the structural observer to evaluate construction based on the design intent as defined in the *approved construction documents*. This goes further than the *structural observation* definition (see below) where conformance to the *approved construction documents* is evaluated. This PC incorporates language from the *structural observation* definition to ensure conformance with the *approved construction documents* and keeps the section aligned with the definition.

In addition, the structural observer may or may not know the "intent" of the design. While it would be ideal for the registered design professional (RDP) performing the structural observation to also be the person who designed the structure, it is not a requirement in the code. If the owner chooses to employ a different RDP, it will be difficult (and in some cases, impossible) for the structural observer to know the design intent.

[BS] STRUCTURAL OBSERVATION. The visual observation of the structural system by a registered design professional for general conformance to the approved construction documents.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
Aligning the duties of the structural observer with the definition of *structural observation* brings clarification of their duties and limits the scope of the observation that they perform. Without the public comment, structural observers might see their duty as extending beyond the information found on the approved *construction documents* which could add cost.

Public Comment# 1426

S96-19

IBC®: TABLE 1705.3

Proposed Change as Submitted

Proponents: Jason Krohn, representing Precast/Prestressed Concrete Institute (jkrohn@pci.org)

2018 International Building Code

Revise as follows:

TABLE 1705.3
REQUIRED SPECIAL INSPECTIONS AND TESTS OF CONCRETE CONSTRUCTION

Portions of table not shown remain unchanged.

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 bars other than ASTM A706; b.Inspect ~~single-pass fillet welds, maximum $^5/_{16}$″~~ welding of reinforcement for special moment frames, boundary elements of special structural walls, and coupling beams; c. Inspect welded reinforcement splices; and d. Inspect all other welds.	— X X ~~X~~ —	X ~~X~~ — — X	AWS D1.4 ACI 318: 26.~~6.4~~ 13.3	—

For SI: 1 inch = 25.4 mm.

a. Where applicable, see 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 for special moment frames, boundary elements of special structural walls, and coupling beams) 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 represented an unnecessary expansion of special inspection requirements that did not result in any apparent benefit.

Modifications to the items requiring inspection have been made in ACI 318-19 Section 26.13.3. ACI 318 has determined that continuous special inspection of welding of reinforcement for intermediate moment frames is unnecessary. It has also determined that continuous special inspection of shear reinforcement is necessary only for special moment frames, boundary elements of special structural walls, and coupling beams. These determinations are reflected in this submitted code change.

Cost Impact: The code change proposal will decrease the cost of construction
The cost of precast concrete construction, where welding of reinforcing bars is not uncommon, should decrease modestly through elimination of unnecessary continuous special inspection in many cases.

S96-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee expressed concerns that the proponent did not sufficiently justify why the change was necessary. The reason statement implies that the change is 'organizational' only; however, it has technical changes included.
(Vote: 11-2)

Assembly Action: **None**

S96-19

Individual Consideration Agenda

Public Comment 1:

IBC®: TABLE 1705.3

Proponents:
Edith Smith, representing Precast/Prestressed Concrete Institute (esmith@pci.org)

requests As Submitted

Commenter's Reason: We are asking for approval of S96 as submitted.
The original proposal was written to revert the special inspection criteria back to what they were in the 2012 IBC.

The proposed change ensures continuous special inspection of reinforcing bar welding in critical locations based on the loading conditions and adds continuous special inspection for shear reinforcing. It allows periodic inspection in less critical regions and continuous special inspection can then be mandated for welds, the failure of which is liable to have serious, even catastrophic, consequences. That is the way IBC Chapter 17 requirements were from the 2000 through the 2012 IBC.

This change also provides consistency with the AWS inspection requirement at “suitable intervals” and recognizes ACI 318-19 Section 26.13.3, which requires only special moment frames, boundary elements of special structural walls, and coupling beams necessitate continuous special inspection of flexural and shear reinforcement. There is no supporting evidence that other continuous inspections are necessary.

There is no evidence that suggests the 2015 IBC change was necessary due to a life safety hazard. All the proposed changes are in line with earlier versions of the code and enhance safety through inspection of critical items.

Please note, this change was approved during the public comment of the last code cycle. Unfortunately, the approval was overturned by on-line balloting. We would request that once again; the membership approve the code change proposal as submitted.

Cost Impact: The net effect of the public comment and code change proposal will decrease the cost of construction
The cost of precast concrete construction, where welding of reinforcing bars is not uncommon, should decrease modestly through elimination of unnecessary continuous special inspection in many cases.

Public Comment# 1408

S98-19

IBC®: 1705.4

Proposed Change as Submitted

Proponents: Terry Kozlowski, representing Southern Nevada Chapter; Amanda Moss, representing SN-ICC Member; Cassidy Wilson, representing SN-ICC Member

2018 International Building Code

Revise as follows:

1705.4 Masonry construction. *Special inspections* and tests of masonry construction shall be performed in accordance with the quality assurance program requirements of TMS 402 and TMS 602.

Exception: Special inspections and tests shall not be required for:

1. Empirically designed masonry, glass unit masonry or masonry veneer designed in accordance with Section 2109, 2110 or Chapter 14, respectively, where they are part of a structure classified as *Risk Category* I, II or III.
2. Masonry foundation walls constructed in accordance with Table 1807.1.6.3(1), 1807.1.6.3(2), 1807.1.6.3(3) or 1807.1.6.3(4).
3. Masonry fireplaces, masonry heaters or masonry chimneys installed or constructed in accordance with Section 2111, 2112 or 2113, respectively.
4. Masonry fences less than or equal to 8'-0" in height, retaining walls less than or equal to 6'-0" in height and combined masonry fences and retaining walls less than or equal to 14'-0" in overall height with the fence portion less than or equal to 8'-0" in height provided that the walls are designed in accordance with Chapter 2 of TMS 402-16 with allowable stresses for masonry reduced by one-half and f'm does not exceed 1500 psi. Wall heights shall be measured from the top of footing to the top of wall.

Reason: This proposal, eliminating the need for an additional inspection, has been utilized and evaluated in Southern Nevada for several years without any adverse structural and/or safety-related issues.

Cost Impact: The code change proposal will decrease the cost of construction
Regionally, this has resulted in reduced design, permitting, construction and inspection time frames and reduced construction costs.

S98-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee expressed concerns that the proposal, as written, would allow larger structures than currently permitted to be constructed without special inspections. The proponent did not sufficiently justify the increase. As written, the proposal would allow a fence to be on top of a wall to create a 'tall element' to be built without special inspections.
(Vote: 14-0)

Assembly Action: **None**

S98-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1705.4

Proponents:
Gary Ehrlich, representing National Association of Home Builders (gehrlich@nahb.org)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1705.4 Masonry construction. *Special inspections* and tests of masonry construction shall be performed in accordance with the quality assurance program requirements of TMS 402 and TMS 602.

Exception: Special inspections and tests shall not be required for:

1. Empirically designed masonry, glass unit masonry or masonry veneer designed in accordance with Section 2109, 2110 or Chapter 14, respectively, where they are part of a structure classified as *Risk Category* I, II or III.
2. Masonry foundation walls constructed in accordance with Table 1807.1.6.3(1), 1807.1.6.3(2), 1807.1.6.3(3) or 1807.1.6.3(4).
3. Masonry fireplaces, masonry heaters or masonry chimneys installed or constructed in accordance with Section 2111, 2112 or 2113, respectively.
4. Masonry fences less than or equal to 7 feet (2134 mm) ~~8'-0"~~ in height from the base of the fence or~~,~~ retaining walls less than or equal to 4 feet (1219 mm) ~~6'-0"~~ in height measured from the top of footing to the top of the wall.~~ and combined masonry fences and retaining walls less than or equal to 14'-0" in overall height with the fence portion less than or equal to 8'-0" in height provided that the walls are designed in accordance with Chapter 2 of TMS 402-16 with allowable stresses for masonry reduced by one-half and f'm does not exceed 1500 psi. Wall heights shall be measured from the top of footing to the top of wall.~~

Commenter's Reason: The purpose of this public comment is to revise the proposed exception to correlate with the fence and retaining wall heights for which a permit is not required under Section 105.2. If a project consisted entirely of such a fence or such a retaining wall, a special inspection would never be required since no permit would be required, and application for a permit is necessary to trigger a special inspection under Section 1704.2. It stands to reason that a special inspection should therefore not be required for such a fence or retaining wall merely because it is part of a larger project for which a permit is sought and special inspections are triggered. By aligning with the heights required to trigger a permit, the additional language on reduced masonry stresses is no longer required, because the Section 105.2 exceptions are not linked to wall materials or material strengths.

Cost Impact: The net effect of the public comment and code change proposal will decrease the cost of construction
While the public comment would cover fewer masonry fences and retaining walls than the original proposal, the net effect of the comment and the proposal would still be a reduction in the cost of construction relative to current code, as fences up to 7 feet and retaining walls up to 4 feet included as part of a larger permit submittal would otherwise trigger special inspection.

Public Comment# 1374

S100-19

IBC: 1705.5.3 (New), TABLE 1705.5.3 (New)

Proposed Change as Submitted

Proponents: Stephen DiGiovanni, representing ICC Ad Hoc Committee on Tall Wood Buildings (TWB) (TWB@iccsafe.org)

2018 International Building Code

Add new text as follows:

1705.5.3 Mass timber construction. *Special inspections* of Mass Timber elements in Types IV-A, IV-B and IV-C construction shall be in accordance with Table 1705.5.3.

TABLE 1705.5.3
REQUIRED SPECIAL INSPECTIONS OF MASS TIMBER CONSTRUCTION

Type	Continuous Special Inspection	Periodic Special Inspection
1. Inspection of anchorage and connections of mass timber construction to timber deep foundation systems.		X
2. Inspect erection of mass timber construction		X
3. Inspection of connections where installation methods are required to meet design loads		
3.1. Threaded fasteners		
3.1.1. Verify use of proper installation equipment.		X
3.1.2. Verify use of pre-drilled holes where required.		X
3.1.3. Inspect screws, including diameter, length, head type, spacing, installation angle, and depth.		X
3.2. Adhesive anchors installed in horizontal or upwardly inclined orientation to resist sustained tension loads	X	
3.3. Adhesive anchors not defined in 3.2.		X
3.4. Bolted connections		X
3.5. Concealed connections		X

Reason: This proposal adds special inspection provisions to Section 1705 for mass timber. This new and unique type of construction requires a level of inspection consistent with other large buildings and unique applications where milestone inspections by the jurisdictional inspectors are not rigorous enough to ensure a level of quality control or quality assurance of the construction process. The proposed special inspections are similar to what is required for other prefabricated systems such as pre-cast concrete and structural steel.
Special Inspection is the monitoring of materials, installation, fabrication, erection and placement of components and connections that require special expertise that are critical to the integrity of the building structure. The special inspectors are required to ensure compliance with the approved construction documents and referenced standards. The program allows jurisdictions to have access to highly specialized and trained inspectors. Some special inspection activities require construction activities to be continuously inspected; which would be logistically difficult for a typical building inspection program. Special inspection is a vital part of the compliance path for successful and compliant building projects constructed under the International Building Code.

The specific elements requiring special inspection are:

1. Periodic inspection of the connection of mass timber elements to wood foundation elements. These connections are critical to transfer loads from the mass timber elements to the piles, particularly for lateral loading. The connections to concrete foundations are addressed in Table 1705.3, Item #3.
2. Periodic inspection of erection of mass timber elements. Similar to pre-cast concrete (Table 1705.3, Item #10), tall wood buildings utilizing pre-fabricated elements needs to have verification that the correct elements are placed in the right location in accordance with the design drawings.
3. Inspection of specialized connections.

Connections between mass timber products that utilized threaded, bolted, or concealed connections are considered periodic in a similar manner that concrete special inspections are required in Table 1705.3. The strength of many connection designs is predicated on specific screw lengths and installation angles. Bolted connections require specific diameters, and for lag bolts, specific lengths. Concealed connectors, many of which are proprietary, must be installed correctly for structural performance. Most of these cannot be verified by the jurisdictional inspector, so special inspections are required.

Adhesive anchorage installed in horizontal or upwardly inclined positions resisting tension loads shall be continuously inspected, again similar to Table 1705.3, Item 4a. This is required because of issues with creep of the adhesives under long-term tension loading discussed in previous code change cycles. However, once again similar to the requirements for precast concrete, all other adhesive anchors need only be inspected periodically (ref. Table 1705.3, Item 4b).

If there are other unusual items not covered in the proposed table, the existing text in Section 1705.1.1 gives the building official the authority to require special inspections for those unusual items. The same section also says the building official can require special inspections where manufacturers' installation instructions prescribe requirements not contained in the code. For example, field-glued mass timber beam or panel splices, while currently rare in North America, may become more prevalent in the future. This is not an item that is covered in the proposed Table 1705.5. While the AHC-TWB is not aware of any of those types of splices that are not currently proprietary, Section 1705.1.1 would allow the building official to require special inspections for either proprietary or non-proprietary field-glued splices. Note that many design engineers will also specify the need for special inspections for unusual conditions in their structural notes in the construction documents, or in the statement of special inspections (see Sections 1704.2.3 and 1704.3).

No changes are being proposed to address fabrication of mass timber structural elements. Mass timber structural assembled in a fabricator shop should be addressed by sections 1704.2.5 and 1704.2.5.1 of the current codes regarding fabrication

The Ad Hoc Committee for Tall Wood Buildings (AHC-TWB) was created by the ICC Board of Directors to explore the building science of tall wood buildings with the scope to investigate the feasibility of and take action on developing code changes for these buildings. Members of the AHC-TWB were appointed by the ICC Board of Directors. Since its creation in January, 2016, the AHC-TWB has held 8 open meetings and numerous Work Group conference calls. Four Work Groups were established to address over 80 issues and concerns and review over 60 code proposals for consideration by the AHC-TWB. Members of the Work Groups included AHC-TWB members and other interested parties. Related documentation and reports are posted on the AHC-TWB website at https://www.iccsafe.org/codes-tech-support/cs/icc-ad-hoc-committee-on-tall-wood-buildings/.

Cost Impact: The code change proposal will increase the cost of construction
Since all the code proposals related to Mass Timber products are to address new types of building construction, in theory this will not increase the cost of construction, but rather provides design options not currently provided for in the code. The committee took great care to not change the requirements of the pre-existing construction types, and our changes do not increase the cost of construction using those pre-existing construction types. However, based on a typically residential or office building of typical floor plates an estimate of Special Inspection costs would range from $1,000 to $2,000 per floor. Another approach to the cost of special inspection is a percentage of total construction costs; for typical pre-fabricated construction elements the cost of special inspection can range between 0.15% to 0.30%, depending on labor cost and complexities of the construction in the building. These estimates are based on responses to surveys of special inspection agencies in the Seattle and Las Vegas areas.

S100-19

Public Hearing Results

Committee Action: **As Submitted**

Committee Reason: This proposal adds special inspection provisions to Section 1705 for mass timber consistent with the findings of the Tall Wood Ad Hoc Committee and consistent with the Group A actions. This new and unique type of construction requires a level of inspection consistent with other large buildings and unique applications where milestone inspections by the jurisdictional inspectors are not rigorous enough to ensure a level of quality control or quality assurance of the construction process. The proposed special inspections are similar to what is required for other prefabricated systems such as pre-cast concrete and structural steel.
(Vote: 13-1)

Assembly Action: **None**

S100-19

Individual Consideration Agenda

Public Comment 1:

IBC®: TABLE 1705.5.3 (New)

Proponents:
Scott Campbell, representing National Ready Mixed Concrete Association (scampbell@nrmca.org)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

TABLE 1705.5.3
REQUIRED SPECIAL INSPECTIONS OF MASS TIMBER CONSTRUCTION

Type	Continuous Special Inspection	Periodic Special Inspection
1. Inspection of anchorage and connections of mass timber construction to timber deep foundation systems.		X
2. Inspect erection of mass timber construction		X
3. Inspection of connections where installation methods are required to meet design loads		
3.1. Threaded fasteners		
3.1.1. Verify use of proper installation equipment.		X
3.1.2. Verify use of pre-drilled holes where required.		X
3.1.3. Inspect screws, including diameter, length, head type, spacing, installation angle, and depth.		X
3.2. Adhesive anchors installed in horizontal or upwardly inclined orientation to resist sustained tension loads	X	
3.3. Adhesive anchors not defined in 3.2.		X
3.4. Bolted connections		X
3.5. Concealed connections	X	~~X~~

Commenter's Reason: This public comment is attempting to incorporate the comments from the Committee on S100 and S101, and to balance the need for additional inspections on new systems with the desire to treat all superficially similar systems equally. The proposal suggests changing the periodic special inspection requirement to continuous special inspection for concealed connections in mass timber. The specific reasons why continuous special inpsections are advised for concealed connections are:
(1) The tests performed and reviewed by the Tall Wood Ad-Hoc Committee indicated that the connections are crucial for not only the structural performance of the mass timber systems, but also for achieving the desired fire resistance. While many connection types can be visually inspected after installation, those connections that are concealed cannot be evaluated post-install. The criticality of the connections and the lack of ability to review after the fact are indicators that continuous special inspection should be required.

(2) The connections currently in use are almost entirely proprietary, requiring special training for installation, and thus should require continuous special inspection until such a time as sufficient experience has been gained that code officials can be confident that the contractors know how to correctly install the connections.

(3) The main argument for having periodic special inspections is that the mass timber systems are similar to precast concrete and steel framing systems. While this argument makes sense at first glance, the fact is that precast concrete systems have been widely used for over 60 years, and steel framing for over 100. Everyone involved, from the designers to the contractors to the code officials is aware of how the systems should be constructed and what to look for during inspections. This is not true for mass timber construction where most jurisdictions do not have any mass timber buildings. Until code officials can be confident that the contractors have sufficient experience to justify relaxed inspection requirements the more stringent continuous inspection requirements should be put into place.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
This comment is dealing with a new system that is not currently in the code and therefore has no cost impact on current construction practice.

Public Comment# 2004

Public Comment 2:

IBC®: 1705.5.3 (New), TABLE 1705.5.3 (New)

Proponents:
Edith Smith, representing Precast/Prestressed Concrete Institute (esmith@pci.org)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1705.5.3 Mass timber construction. *Special inspections* of Mass Timber elements in Types IV-A, IV-B and IV-C construction shall be in accordance with Table 1705.5.3.

TABLE 1705.5.3
REQUIRED SPECIAL INSPECTIONS OF MASS TIMBER CONSTRUCTION

Type	Continuous Special Inspection	Periodic Special Inspection
1. Inspection of anchorage and connections of mass timber construction to timber deep foundation systems.		X
2. Inspect erection of mass timber construction		X
3. Inspection of connections where installation methods are required to meet design loads		
3.1. Threaded fasteners		
3.1.1. Verify use of proper installation equipment.		X
3.1.2. Verify use of pre-drilled holes where required.		X
3.1.3. Inspect screws, including diameter, length, head type, spacing, installation angle, and depth.		X
3.2. Adhesive anchors installed in horizontal or upwardly inclined orientation to resist sustained tension loads	X	
3.3. Adhesive anchors not defined in 3.2.		X
3.4. Bolted connections		X
3.5. Concealed connections		X
4.0. Connections where installation methods are required to meet the fire resistance design in 2304.10.1.		X

Commenter's Reason: This public comment adds special inspection provisions to Table 1705.5.3 for mass timber. Buildings of mass timber over 6-stories involves new challenges in the construction of tall buildings, and contractors and inspectors have little or no experience working with these systems of wood material for tall buildings. Due to the importance of connections in the successful fire performance of mass timber systems, and the lack of long term experience for involved parties constructing these taller buildings, a level of inspection beyond that commonly required of other construction methods is warranted. This is consistent with the intent of Section 1705.1.1 where special inspections are intended for unusual design applications of materials included in the code, or where adherence to manufacturer’s instructions for materials and systems are not specified in the code is required.
Requiring special inspection of these connections for fire resistance is also similar to the requirements in Section 1705.14, where sprayed fire-resistant materials must undergo special inspections and tests to document acceptance. These requirements for mass timber are similar in nature to these special inspections.

Finally, this public comment adds Section 2304.10.1 to specify how the fire resistance rating of connections for the Types IV-A, IV-B and IV-C construction is to be determined. This language is identical to the language proposed by the ICC Ad-Hoc Committee on Tall Wood Buildings in S170-19, which was recommended for approval by the Structural Committee. It is included in this public comment to show how it would relate to the reference in Table 1705.5.3 and should not be considered as a separate code proposal.

We request that the membership approve the code change S100-19 AS MODIFIED.

Cost Impact: The net effect of the public comment and code change proposal will increase the cost of construction
Code proposals for mass timber address a new types of construction in the IBC. In theory this will not increase the cost of construction instead providing design alternatives in the code. However, because of the newness of mass timber as a method of construction there will be some additional costs to provide special inspections to ensure the code is met.

Public Comment# 1590

S107-19

IBC: 1705.12.7, TABLE 1705.12.7 (New), 2209.3 (New), MHI Chapter 35(New)

Proposed Change as Submitted

Proponents: Paul Douglas Armstrong, PACCS, representing MHI

2018 International Building Code

Delete and substitute as follows:

~~**1705.12.7 Storage racks.** *Periodic special inspection* is required for the anchorage of storage racks that are 8 feet (2438 mm) or greater in height in structures assigned to *Seismic Design Category* D, E or F.~~

1705.12.7 Storage racks. If required by the Engineer of Record storage racks that are 8 feet in height or greater and assigned to Seismic Design Category D, E, or F shall be inspected by an inspector designated by the Engineer of Record as detailed in Table 1705.12.7 for adherence with the approved construction documents.

Add new text as follows:

TABLE 1705.12.7
Required Inspections of Storage Rack Systems

Type	Continuous Inspection	Periodic Inspection	Referenced Standard	IBC Reference
Verify materials used comply with one or more of the material test reports in accordance with the *approved construction documents*	—	X	—	—
Fabricated storage rack elements	—	X	—	1704.2.5
Installation of storage rack anchorage	—	X	MH16.1 Section 7.3.2	—
If required by the Engineer of Record, a final inspection of the completed storage rack system for compliance with the Load Application and Rack Configuration documents	—	X	—	—

2209.3 Certification For Storage Structures 8 feet in height or greater to the top load level and assigned to Seismic Design Category D, E, or F, if required by the Engineer of Record, at completion of the storage rack installation, the Engineer of Record shall submit a *certificate of compliance* to the owner or the owner's authorized agent stating that the work was performed in accordance with *approved construction documents* and with specifications listed in this section.

MHI

Material Handling Institute
8720 Red Oak Blvd. Suite 201
Charlotte NC 28217

MH16.1: 2012: **Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks**

Reason: The design of the components that go into the storage rack are based upon minimum thickness, minimum yield strength, etc. and it is imperative that these minimum properties are complied with in the fabrication of the components and included in storage rack installations. Storage rack systems canbe complex and it is important that they how they are installed complies with the permitted drawings on file with the local building department, which is why they may need to be monitored.

Cost Impact: The code change proposal will increase the cost of construction
In high seismic areas budgets will need to include this required set of inspections for installations of storage rack structures.

Staff Analysis: A review of the standard proposed for inclusion in the code, MHI MH16.1: 2012, 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 2, 2019.

S107-19

Public Hearing Results

Committee Action: **As Modified**

Committee Modification:
2018 International Building Code

1705.12.7 Storage racks. ~~If required by the Engineer of Record~~ S~~s~~torage racks , that are 8 feet in height or greater and assigned to Seismic Design Category D, E, or F , shall be provided with *periodic special inspection* as required by ~~inspected by an inspector designated by the Engineer of Record as detailed in~~ Table 1705.12.7. ~~for adherence with the approved construction documents~~.

TABLE 1705.12.7

Required Inspections of Storage Rack Systems

Type	Continuous Inspection	Periodic Inspection	Referenced Standard	IBC Reference

Verify materials used comply with one or more of the material test reports in accordance with the *approved construction documents*	__	X	__	__
Fabricated storage rack elements	__	X	__	1704.2.5
Installation of storage rack anchorage	__	X	ANSI/MH16.1 Section 7.3.2	__
~~If required by the Engineer of Record, a~~ At final inspection of the completed storage rack system, to indicate ~~for~~ compliance with the ~~Load Application and Rack Configuration~~ *approved construction documents*	__	X	__	__

2209.3 Certification. For rack s~~S~~torage s~~S~~tructures that are 8 feet in height or greater to the top load level and assigned to Seismic Design Category D, E, or F~~, if required by the Engineer of Record,~~ at completion of the storage rack installation, ~~the Engineer of Record shall submit~~ a *certificate of compliance* shall be submitted to the owner or the owner's authorized agent stating that the work was performed in accordance with *approved construction documents* ~~and with specifications listed in this section~~.

~~MHI~~

~~Material Handling Institute~~

~~8720 Red Oak Blvd. Suite 201~~

~~Charlotte~~

~~NC~~

~~28217~~

~~MH16.1: 2012:~~

~~Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks~~

Committee Reason: The design of the components that go into the storage rack are based upon minimum thickness, minimum yield strength, etc. and it is imperative that these minimum properties are complied with in the fabrication of the components and included in storage rack installations. Storage rack systems can be complex and it is important that they how they are installed complies with the permitted drawings on file with the local building department, which is why they may need to be monitored. The committee expressed concerns on the contractual aspects of proposed section 2209.3 for review during the public comment phase. The approved floor modifications clarified the intent of the proposal and deletes the 'addition' of the reference as the reference is already in the IBC. (Vote: 14-0)

Assembly Action: **None**

S107-19

Individual Consideration Agenda

Public Comment 1:

IBC®: TABLE 1705.12.7 (New), 2209.3 (New)

Proponents:
Paul Armstrong, MHI, representing MHI (paul.armstrong@pacodeservices.com)

requests As Modified by Public Comment

Further modify as follows:

2018 International Building Code

TABLE 1705.12.7
Required Inspections of Storage Rack Systems

Type	Continuous Inspection	Periodic Inspection	Referenced Standard	IBC Reference
Verify materials used comply with one or more of the material test reports in accordance with the *approved construction documents*, *when required by the engineer of the rack structure.*	—	X	—	—
Fabricated storage rack elements	—	X	—	1704.2.5
Installation of storage rack anchorage	—	X	ANSI/MH16.1 Section 7.3.2	—
At final inspection of the completed storage rack system, to indicate compliance with the *approved construction documents*, *when required by the engineer of the rack structure.*	—	X	—	—

2209.3 Certification For rack storage structures that are 8 feet in height or greater to the top load level and assigned to Seismic Design Category D, E, or F at completion of the storage rack installation,a *certificate of compliance* shall be submitted to the owner or the owner's authorized agent stating that the work was performed in accordance with *approved construction documents*, *when required by the engineer of the rack structure*.

Commenter's Reason: In addition to the original reason statement, engineers of storage racking systems have determined that there is a need for special inspection in specific situations. With this amendment, they can have the opportunity then to have special inspectors verify that the designed rack systems are erected with the intended components approved plan and that the owner of such systems can be assured that the system will operate in its intended fashion. This is not always a requirement so Table 1705.12.7, Items 1 and 4 and Section 2209.3 are available to be used at the rack system engineer's discretion.

Cost Impact: The net effect of the public comment and code change proposal will increase the cost of construction
The cost of a special inspection in identified cases will increase the cost of construction.

Public Comment# 2048

Public Comment 2:

IBC®: 1705.12.7, TABLE 1705.12.7 (New)

Proponents:
Jenifer Gilliland, representing Seattle Department of Construction and Inspections (SDCI) (jenifer.gilliland@seattle.gov); Jonathan Siu, representing City of Seattle Department of Construction and Inspections (jon.siu@seattle.gov)

requests As Modified by Public Comment

Further modify as follows:

2018 International Building Code

1705.12.7 Storage racks. *Steel storage racks* and *steel cantilevered storage racks* ~~Storage racks,~~ that are 8 feet in height or greater and assigned to Seismic Design Category D, E, or F, shall be provided with *periodic special inspection* as required by Table 1705.12.7.

TABLE 1705.12.7
Required Inspections of Storage Rack Systems

Type	Continuous Inspection	Periodic Inspection	Referenced Standard	IBC Reference
Verify materials used comply with one or more of the material test reports in accordance with the *approved construction documents*	—	X	—	—
Fabricated storage rack elements	—	X	—	1704.2.5
Installation of storage rack anchorage a. *Steel storage rack* b. *Steel cantilevered storage rack*	— —	X X	ANSI/MH 16.1Section 7.3.2 ANSI/MH 16.3 Section 8.5.2	— —
At final inspection of the completed storage rack system, to indicate compliance with the *approved construction documents*	—	X	—	—

Commenter's Reason: The definition *steel cantilevered storage rack* was approved by the ICC Structural Committee in proposal S161-19. This public comment incorporates *steel cantilevered storage rack* special inspection requirements and references the appropriate ANSI standard. It also clarifies that there are now two defined storage rack systems and italicizes these terms.

Cost Impact: The net effect of the public comment and code change proposal will increase the cost of construction
As mentioned in the proponents original proposal, the cost of construction will increase because these storage rack systems will now be subject to special inspection. The public comment clarifies that the requirement for special inspection, and therefore the cost increase, applies to *steel cantilevered storage racks*, not just *steel storage racks*.

Public Comment# 1850

S108-19

IBC®: 1709.5, AAMA Chapter 35 (New)

Proposed Change as Submitted

Proponents: Jennifer Hatfield, representing American Architectural Manufacturers Association (jen@jhatfieldandassociates.com)

2018 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 exterior windows and doors tested in accordance with Sections 1709.5.1 or 1709.5.2, required design wind pressures determined from ASCE 7 shall be permitted to be converted to allowable stress design by multiplying by 0.6.

> **Exception:** Structural wind load design pressures for window ~~units smaller~~ or door assemblies other than the size tested in accordance with Section 1709.5.1 or 1709.5.2 shall be permitted to be ~~higher~~ different than the design value of the tested ~~unit~~ assembly provided such ~~higher~~ pressures are determined by accepted engineering ~~analysis.~~ analysis or validated by an additional test of the window or door assembly to the alternative allowable design pressure in accordance with Section 1709.5.2. Components of the ~~small unit~~ alternate size assembly shall be the same as the tested ~~unit. Where such calculated design pressures are~~ or labeled assembly. Where engineering analysis is used, ~~they~~ it shall be ~~validated by an additional test of the window unit having the highest allowable design pressure.~~ performed in accordance with the analysis procedures of AAMA 2502.

Add new standard(s) as follows:

AAMA

American Architectural Manufacturers Association
1827 Waldon Office Square, Suite 550
Schaumburg IL 60173

2502--2019: Comparative Analysis Procedure for Window and Door Products

Reason: The current exception limits the use of comparative analysis to window units smaller than the size originally tested. If comparative analysis is used to provide a higher design pressure rating of the smaller unit, it must be verified by testing of the unit as well. Additional testing should not be required if accepted engineering analysis is used.
It is also appropriate to use comparative analysis to rate window units larger than the size originally tested to lower design pressures. Testing should not be required to verify this level of performance since a higher pressure level has already been determined by testing of the same components in a smaller window unit and accepted engineering analysis is used.

This proposal revises this section as appropriate to permit the use of comparative analysis for larger as well as smaller window units than those tested. The last sentence of the section is also revised to define accepted engineering analysis as that which is specified and performed in accordance with the analysis procedures of AAMA 2502, a reference standard being added by this proposal that provides a standardized comparative analysis procedure for determining the structural integrity of window and door products.

The proposal also replaces the term "unit" with the word "assembly," as the term “assemblies” is used in the title of section 1709.5 and is the appropriate terminology that is reflected in AAMA 2502.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
The code change will not increase the cost of construction but rather it simply permits the use of comparative analysis for larger assemblies.

Staff Analysis: A review of the standard proposed for inclusion in the code, AAMA 2502-2019, 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 2, 2019.

S108-19

Public Hearing Results

Committee Action: **As Submitted**

Committee Reason: Proposal provides a convenient way to validate based on existing test results.
(Vote: 11-3)

Assembly Action: **None**

S108-19

Individual Consideration Agenda

Public Comment 1:

Proponents: CP28 Administration

Commenter's Reason: The administration of ICC Council Policy 28 (CP28) is not taking a position on this code change.
This public comment is being submitted to bring a procedural requirement to the attention of the ICC voting membership.
In accordance with Section 3.6.3.1.1 of ICC Council Policy 28 (partially reproduced below), the new referenced standard
AAMA 2502-2019: Comparative Analysis Procedure for Window and Door Products, must be completed and readily available prior to the Public Comment
Hearing in order for this public comment to be considered.

(CP28) 3.6.3.1.1 Proposed New Standards. In order for a new standard to be considered for reference by the Code, such standard shall be submitted in at least a consensus draft form in accordance with Section 3.4. If the proposed new standard is not submitted in at least consensus draft form, the code change proposal shall be considered incomplete and shall not be processed. The code change proposal shall be considered at the Committee Action Hearing by the applicable code development committee responsible for the corresponding proposed changes to the code text. If the committee action at the Committee Action Hearing is either As Submitted or As Modified and the standard is not completed, the code change proposal shall automatically be placed on the Public Comment Agenda with the recommendation stating that in order for the public comment to be considered, the new standard shall be completed and readily available prior to the Public Comment Hearing

Public Comment# 2193

S113-19

IBC: 1805.3.2

Proposed Change as Submitted

Proponents: Craig Conner, representing self (craig.conner@mac.com); Joseph Lstiburek, representing self (joe@buildingscience.com)

2018 International Building Code

1805.3.1 Floors. Floors required to be waterproofed shall be of concrete and designed and constructed to withstand the hydrostatic pressures to which the floors will be subjected.

Waterproofing shall be accomplished by placing a membrane of rubberized asphalt, butyl rubber, fully adhered/fully bonded HDPE or polyolefin composite membrane or not less than 6-mil [0.006 inch (0.152 mm)] polyvinyl chloride with joints lapped not less than 6 inches (152 mm) or other *approved* materials under the slab. Joints in the membrane shall be lapped and sealed in accordance with the manufacturer's installation instructions.

1805.3.2 Walls. Walls required to be waterproofed shall be of concrete or masonry and shall be designed and constructed to withstand the hydrostatic pressures and other lateral loads to which the walls will be subjected.
Waterproofing shall be applied from the bottom of the wall to not less than 12 inches (305 mm) above the maximum elevation of the ground-water table. The remainder of the wall shall be dampproofed in accordance with Section 1805.2.2 . Waterproofing shall consist of two-ply hot-mopped felts, not less than 6-mil (0.006 inch; 0.152 mm) polyvinyl chloride, 40-mil (0.040 inch; 1.02 mm) polymer-modified asphalt, 6-mil (0.006 inch; 0.152 mm) polyethylene ; a drainage layer of not less than 4 inches (100 mm) of free draining granular material; a drainage layer that can be shown to provide equivalent performance to not less than 4 inches (100 mm) of free draining granular material; or other approved methods or materials capable of bridging nonstructural cracks. Joints in the membrane or layers shall be lapped and sealed in accordance with the ~~manufacturer's~~ manufacturer's installation instructions.

1805.3.2.1 Surface preparation of walls. Prior to the application of waterproofing materials on concrete or masonry walls, the walls shall be prepared in accordance with Section 1805.2.2.1 .

1805.3.3 Joints and penetrations. Joints in walls and floors, joints between the wall and floor and penetrations of the wall and floor shall be made water tight utilizing *approved* methods and materials.

Reason: Objective:
Provide more options for foundation waterproofing and dampproofing.

This code change provides additional options for foundation waterproofing and dampproofing.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This change simply adds more options. In some cases it might decrease costs.

S113-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee did not believe that this is an 'equivalent' option as it does not bridge the nonstructural cracks and it is not suitable for heavy clay soils. In general drainage is not a substitute for waterproofing.
(Vote: 13-1)

Assembly Action: **None**

S113-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1805.3.2

Proponents:
Craig Conner, representing self (craig.conner@mac.com); Joseph Lstiburek, representing self (joe@buildingscience.com)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1805.3.2 Walls. Walls required to be waterproofed shall be of concrete or masonry and shall be designed and constructed to withstand the hydrostatic pressures and other lateral loads to which the walls will be subjected.
Waterproofing shall be applied from the bottom of the wall to not less than 12 inches (305 mm) above the maximum elevation of the ground-water table. The remainder of the wall shall be dampproofed in accordance with Section 1805.2.2 . Waterproofing shall consist of two-ply hot-mopped felts, not less than 6-mil (0.006 inch; 0.152 mm) polyvinyl chloride, 40-mil (0.040 inch; 1.02 mm) polymer-modified asphalt, 6-mil (0.006 inch; 0.152 mm) polyethylene; a drainage layer of not less than 4 inches (100 mm) of ~~free draining granular material~~ clean aggregate having a void ratio of not less than 35 percent or a Size Number of 4, 5, 56, or 6 as classified by ASTM C33; a drainage layer that can be shown to provide equivalent performance to not less than 4 inches (100 mm) of ~~free draining granular material~~ clean aggregate having a void ratio of not less than 35 percent or a Size Number of 4, 5, 56, or 6 as classified by ASTM C33; or other approved methods or materials capable of bridging nonstructural cracks. Joints in the membrane or layers shall be lapped and sealed in accordance with the manufacturer's installation instructions.

Commenter's Reason: Committee reason is incorrect. In general drainage is a substitute for waterproofing and recognized as such in international codes.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
This change does not change costs. It provides alternative means and methods of construction.

Public Comment# 1932

S114-19

IBC®: 1807.2.3

Proposed Change as Submitted

Proponents: Terry Kozlowski, representing Southern Nevada Chapter; Nenad Mirkovic, representing City of Las Vegas; Amanda Moss, representing SN-ICC Member; Cassidy Wilson, representing SN-ICC Member

2018 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 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 or wind are included, the minimum safety factor for retaining wall sliding and overturning shall be 1.1.

Reason: The intent is to address loads that a building is likely to experience and precludes consideration of a FACTORED LOAD which applies to limit state or strength design.
The term “nominal loads” is defined in Chapter 2 as “The magnitudes of the loads specified in Chapter 16 (dead, live, soil, wind, snow, rain, flood and earthquake)”. The term “service loads” as used in the definition of “dangerous” is synonymous with the definition of “nominal loads” loads as defined in the IBC Interpretation 23-10.

The International Building Code Section 1807.2.3 covers retaining walls but it does not clearly address safety factor when the freestanding wall, fence or other structures that are constructed on top of the retaining wall or are in the close proximity of the retaining wall and supported by a retaining wall that is subject to nominal loads that include wind and not earthquake load in the load combination. This provides clarification to indicate service wind load to be used in lieu of nominal load (ultimate wind load).

Bibliography: IBC Section 1602 Definitions and Notations
FACTORED LOAD. The product of a nominal load and a load factor.

NOMINAL LOADS. The magnitudes of the loads specificed in this chapter (dead, live, soil, wind, snow, rain, flood and earthquake).

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This proposal will not increase or decrease the cost of construction but rather provides clarification to indicate service wind load to be used in lieu of nominal load (ultimate wind load).

S114-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The proposal assumes that wind loads are typically ignored or missed; however, the committee did not concur - the load combinations include wind loads.
(Vote: 13-1)

Assembly Action: **None**

S114-19

Individual Consideration Agenda

Public Comment 1:

Proponents:

Gerald Gunny, P.E., S.E., City of Henderson, representing Southern Nevada Chapter (gerald.gunny@cityofhenderson.com)

requests As Submitted

Commenter's Reason: The committee disapproved the proposal with modification because the committee felt the winds loads are not ignored or missed and the load combinations include wind loads. In rebuttal however the section clearly states that the load combinations of Section 1605 shall not apply for the retaining wall stability check requirement. In addition the section is not clear how wind loads are considered in the retaining wall stability check requirement.
This IBC section requires that retaining wall designs include a stability check for sliding and overturning and that a factor of safety be applied. Additionally, the section includes an exception that addresses where earthquake loads are included the safety factor is reduced from 1.5 to 1.1 since they are considered short-term loads. For a structural engineer to properly design a retaining wall they will need to also consider wind loads. Yet this section currently is silent when it comes to wind loads or it assumes it is considered as '*other nominal loads'* in which case the wind load case will always govern the stability of the wall when considering the load cases without earthquake loads. This is a significant structural design consideration since wind loads are generated from ASCE 7 using strength design level forces, no 0.6 load reduction factor is allowed and then required to use the 1.5 factor of safety for the wall stability check. Almost all retaining walls have a wall extension above the retained earth and exposed to wind. There are many cases where the construction of a retaining wall extends above the grade of the retained earth or where fences are directly supported on the retaining wall or constructed integral with the retaining wall.

Both earthquake and wind loads are *nominal* lateral loads generated from ASCE 7 at strength design level forces. This modification clearly includes consideration of wind loads when checking the stability of a retaining wall by multiplying the wind load by 0.6 to align the ASCE 7 strength design level wind load to an allowable stress design load as the section does for the 0.7 factor applied to ASCE 7 strength design level earthquake loads. See 2018 IBC Section 1605.3.1 using allowable stress design for the factor justification, i.e., especially Equation 16-12 which has for a combination variable (0.6W or 0.7E). The 1.1 minimum factor of safety for wind loads is consistent with the long-standing geotechnical practice of considering earthquake and wind loads interchangeably as short-term lateral loads.

Cost Impact: The net effect of the public comment and code change proposal will decrease the cost of construction
The code change proposal with modification will decrease the cost of retaining wall construction where wind loads govern the design since the ultimate design level wind loads will be reduced to an allowable stress design loads and the safety factor for the wall stability check is reduced consistent with earthquake loads.

Public Comment# 1547

S118-19

IBC®: 1808.8.1

Proposed Change as Submitted

Proponents: Stephen Szoke, American Concrete Institute, representing American Concrete Institute (steve.szoke@concrete.org); Scott Campbell, National Ready Mixed Concrete Association, representing National Ready Mixed Concrete Association (scampbell@nrmca.org)

2018 International Building Code

Revise as follows:

1808.8.1 Concrete or grout strength and mix proportioning. Concrete or grout in foundations shall have a specified compressive strength (f'_c) not less than the largest applicable value indicated in Table 1808.8.1.

Where concrete is placed through a funnel hopper at the top of a deep foundation element, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 8 inches (204 mm).~~ Where concrete or grout is to be pumped, the mix design including slump shall be adjusted to produce a pumpable mixture.~~

Reason: This code change removes an inappropriate requirement. Grout to be pumped needs to satisfy more requirements than just those required to facilitate pumping. The consistency of the concrete must also satisfy other requirements including but not limited to workability, durability and structural performance requirements. ACI 301 *Specifications for Structural Concrete* provides that: "*4.2.2.2 Slump—Unless otherwise specified, select a target slump or slump flow at the point of delivery for all concrete mixtures. Selected target slump shall not exceed 9 in. Selected target slump flow shall not exceed 30 in. Concrete shall not show visible signs of segregation. The target slump or slump flow value shall be enforced for the duration of the project.*" Current concrete technology provides for both concrete slump and flow as applicable for concrete placement and performance.
ACI 318 *Building Code Requirements for Structural Concrete* which is a reference in the IBC references ACI 301 for concrete mix design criteria. Thus the appropriate criteria are applicable for concrete are requirements of the IBC by reference. This text should be deleted to assure the appropriate criteria for concrete slump and flow are satisfied regardless of delivery methods. ACI, a technical professional society, recommends the committee approve this code change proposal as submitted.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
In general, this code change will not increase nor decrease the cost of construction except there may be cost savings due to the use of admixtures that improve pumpability of concrete while retaining the other necessary properites of the concrete.

S118-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee respected the intent of the proposal; however, as written, the proposal needs work. The committee highly encourage updating / rewording during the public comment phase.
(Vote: 9-5)

Assembly Action: **None**

S118-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1808.8.1

Proponents:
Stephen Szoke, representing American Concrete Institute (steve.szoke@concrete.org)

requests As Modified by Public Comment

Replace as follows:

2018 International Building Code

1808.8.1 Concrete or grout strength and mix proportioning. Concrete or grout in foundations shall have a specified compressive strength (f'_c) not less than the largest applicable value indicated in Table 1808.8.1.

Where concrete is placed through a funnel hopper at the top of a deep foundation element, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 8 inches (204 mm). Where concrete or grout is to be pumped, the mix design ~~including slump~~ shall ~~be adjusted to~~ produce a pumpable mixture.

Commenter's Reason: This provision should be struck from the IBC. The current language requires that the "mix design shall be adjusted to produce a pumpable mixture." This does not specify who has the ability to adjust the mix design, the concrete producer, the pumping contractors or the design professional. Specification of the mix design for structural concrete should be by the design professional and deviations from the mix design required for structural performance should not be permitted. This language permits the mix design specified for the project to be adjusted to produce pumpable concrete. As with all concrete the mix design should be coordinated with the producer, contractors, and design professionals, regardless of method of placement, i.e. ready mixed concrete truck chute, pump, conveyor, funnel hopper, etc. This language should be removed form the IBC. The appropriate direction to the design professional is provided in ACI 318 Building Code Requirements for Structural Concrete, Section 26.5.2.1 and specifically subsection (f):
Concrete shall be placed in accordance with (1) through (5):

(1) At a rate to provide an adequate supply of concrete at the location of placement.
(2) At a rate so concrete at all times has sufficient workability such that it can be consolidated by the intended methods.
(3) Without segregation or loss of materials.
(4) Without interruptions sufficient to permit loss of workability between successive placements that would result in cold joints.
(5) Deposited as near to its final location as practicable to avoid segregation due to rehandling or flowing.

To avoid discrepancies between the design professional, contractors, and producers with the language in the IBC should be deleted. also the issue is addressed in ACI 318 which is referenced for concrete in the IBC.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
This code change maintains that the where concrete or grout are intended to be pumped, the concrete mix design shall be pumpable and thus there should be no increase or decrease in cost. This modification retains the concept that mix designs be as specified by the design professional and not adjusted by contractors, producers, or sub-contractors without approval of the design professional.

Public Comment# 1944

S119-19

IBC®: 1808.8.1, TABLE 1808.8.1

Proposed Change as Submitted

Proponents: Stephen Szoke, American Concrete Institute, representing American Concrete Institute (steve.szoke@concrete.org); Scott Campbell, representing National Ready Mixed Concrete Association (scampbell@nrmca.org)

2018 International Building Code

Revise as follows:

1808.8.1 Concrete or grout strength and mix proportioning. Concrete or grout in foundations shall have a specified compressive strength (f'_c) not less than the largest applicable value indicated in Table ~~1808.8.1.~~ 19.2.1.1 of ACI 318.

Where concrete is placed through a funnel hopper at the top of a deep foundation element, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 8 inches (204 mm). Where concrete or grout is to be pumped, the mix design including slump shall be adjusted to produce a pumpable mixture.

Delete without substitution:

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

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

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

Reason: Removes the table for compressive strength requirements for the 2018 IBC and directs the user to ACI 318 Table 19.2.1.1 Limits for f'_c. The user is already required to use information from ACI 318 for foundations. For example. Table 1808.8.2 Minimum Concrete Cover directs the user to the requirements of Section 20.6 of ACI 318. By not having information in two places will reduce confusion, avoid unintended differences and reduce the potential for errors. Rather than having criteria in two locations this change places criteria on one reference and helps assure that other applicable provisions of ACI 318 as required by 2018 IBC Chapter 19 are not overlooked. Table 1 below shows the comparison of criteria in 2018 IBC and ACI 318. It is noteworthy that, consistent with the overall methodology throughout ACI 318, the user is directed to one section for all relevant criteria. Note that Table 19.2.1.1 has all limits for specified compressive strength in one location. This improves the user-friendliness provided by ACI 318. Further with criteria in two documents that user is required to refer to both to identify potential differences which can be a cumbersome process.

TABLE 1

Comparison of IBC AND ACI 318 MIN. COMPRESSIVE STRENGTH OF CONCRETE OR GROUT

2018 IBC	**ACI 318**	**2018 IBC**	**ACI 318**
Foundation Element of Condition		**Specified Compressive Strength f'_c**	**Minimum f'_c, psi**
1. Foundations for structures assigned to Seismic Design Category A, B or C	1. General	2,500	2,500
2a. Foundations for two stories or less in height, assigned to Seismic Design Category D, E or F 2b. Foundations for other structures assigned to Seismic Design Category D, E or F2	2. Special Moment Frames 2a. Special structural walls with Grade 60 or 80 reinforcement 2b. Special Structural walls with Grade 100 reinforcement	2,500 3,000	3,000[1] 5,000
3. Precast nonprestressed driven piles		4,000 psi	4,000
4. Socketed drilled shafts		4,000 psi	4,000
5. Micropiles		4,000 psi	4,000
6. Precast prestressed driven piles		5,000 psi	5,000

[1] The f'_c for lightweight concrete in special moment frames and special structural walls shall not exceed 5000psi. The limit is permitted to be exceeded where demonstrated by experimental evidence that members made with lightweight concrete provide strength and toughness equal to or exceeding those of comparable members made with normalweight concrete of the same strength.

[2]Does not include foundations for stud bearing wall construction two stories or less.

ACI, a professional technical society, recommends the deletion of the specified compressive strength criteria form the IBC to better assure that all applicable requirements of ACI 318 are properly considered for design and construction of concrete foundations. ACI encourages the committee to approve this code change as submitted.

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

Technical criteria remain unchanged and thus no cost impact.

S119-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: Proponent requested Disapproval. For foundations such as micro piles and deep foundations, ACI 318-19 is not coordinated with the current IBC. (Vote: 14-0)

Assembly Action: **None**

S119-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1808.8.1, TABLE 1808.8.1

Proponents:
Stephen Szoke, representing American Concrete Institute (steve.szoke@concrete.org)

requests As Modified by Public Comment

Replace as follows:

2018 International Building Code

1808.8.1 Concrete or grout strength and mix proportioning. Concrete or grout in shallow foundations and micropiles shall have a specified compressive strength (f'_c) not less than the largest applicable value indicated in Table 1808.8.1. Concrete or grout for deep foundations shall have specified compressive strengths in accordance with Section 13.4.2.1 of ACI 318.

Where concrete is placed through a funnel hopper at the top of a deep foundation element, the concrete mix shall be designed and proportioned so as to produce a cohesive workable mix having a slump of not less than 4 inches (102 mm) and not more than 8 inches (204 mm). Where concrete or grout is to be pumped, the mix design including slump shall be adjusted to produce a pumpable mixture.

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

FOUNDATION ELEMENT OR CONDITION	**SPECIFIED COMPRESSIVE STRENGTH, f'_c**
1.Foundations for structures assigned to *Seismic Design Category* A, B or C	2,500 psi
2a.Shallow foundations for Group R or U occupancies of light-frame construction, two stories or less in height, assigned to *Seismic Design Category* D, E or F	2,500 psi
2b. Shallow foundations for other structures assigned to *Seismic Design Category* D, E or F	3,000 psi
3. ~~Precast nonprestressed driven piles~~ Deep foundations for structures assigned to *Seismic Design Category* D, E, or F	~~4,000 psi~~ ACI 318 18.13.5
4. Socketed drilled shafts	4,000 psi
5. Micropiles	4,000 psi
~~6.Precast prestressed driven piles~~	~~5,000 psi~~

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

Commenter's Reason: This public comment reflects the intent of industry efforts to provide design and construction for deep foundations in ACI 318 through a coordinated effort involving ASCE. New provisions are now included in ACI 318-19 to address this need for coordination. The original proposal failed to retain provisions for shallow foundations, micropiles, and deep foundations in seismic design categories A, B, and C. This public comment retains the provisions for micropiles, shallow foundations, and deep foundations in seismic design categories A, B, and C in the IBC and directs the user to ACI 318 for deep foundations where more comprehensive discussion of requirements is provided.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
Criteria are not altered. Criteria for deep foundations are provided in ACI 318 with more comprehensive discussion than in the 2018 edition of the IBC.

Public Comment# 1271

S120-19

IBC: 1808.8.2, 1808.8.2.1(New), TABLE 1808.8.2

Proposed Change as Submitted

Proponents: Stephen Szoke, American Concrete Institute, representing American Concrete Institute (steve.szoke@concrete.org); Scott Campbell, representing National Ready Mixed Concrete Association (scampbell@nrmca.org)

2018 International Building Code

Revise as follows:

1808.8.2 Concrete cover. The concrete cover ~~provided for prestressed and nonprestressed reinforcement in~~ for all concrete deep foundations shall be ~~not less than the largest applicable value specified in Table 1808.8.2. Longitudinal bars spaced less than 1$^1/_2$ inches (38 mm) clear distance apart shall be considered to be bundled bars for which the concrete cover provided shall be not less than that required by Section 20.6.1.3.4 of ACI 318. Concrete cover shall be measured from the concrete surface to the outermost surface of the steel to which the cover requirement applies. Where concrete is placed in a temporary or permanent casing or a mandrel, the inside face of the casing or mandrel shall be considered to be the concrete surface.~~ in accordance with ACI 318 Section 20.5.1.3.4 and this section.

Add new text as follows:

1808.8.2.1 Structural steel deep foundations. The concrete cover for structural steel cores within a steel pipe, tube or permanent casing shall not be less than 2 inches.

Delete without substitution:

~~**TABLE 1808.8.2**~~
~~**MINIMUM CONCRETE COVER**~~

~~**FOUNDATION ELEMENT OR CONDITION**~~	~~**MINIMUM COVER**~~
~~1.Shallow foundations~~	~~In accordance with Section 20.6 of ACI 318~~
~~2. Precast nonprestressed deep foundation elementsExposed to seawater Not manufactured under plant conditionsManufactured under plant control conditions~~	~~3 inches2 inchesIn accordance with Section 20.6.1.3.3 of ACI 318~~
~~3. Precast prestressed deep foundation elementsExposed to seawater Other~~	~~2.5 inchesIn accordance with Section 20.6.1.3.3 of ACI 318~~
~~4. Cast-in-place deep foundation elements not enclosed by a steel pipe, tube or permanent casing~~	~~2.5 inches~~
~~5. Cast-in-place deep foundation elements enclosed by a steel pipe, tube or permanent casing~~	~~1 inch~~
~~6. Structural steel core within a steel pipe, tube or permanent casing~~	~~2 inches~~
~~7. Cast-in-place drilled shafts enclosed by a stable rock socket~~	~~1.5 inches~~

~~For SI: 1 inch = 25.4 mm.~~

Reason: This code change removes the requirements in IBC Section 1808.2 and Table 1808.2 on concrete cover for foundations to avoid confusion and conflicts between the IBC and ACI 318. Plus, the references are no longer correct, as concrete cover requirements for deep foundations are addressed in Section 20.5.1.3.4 and Table 20.5.1.3.4 of ACI 318. The 2018 IBC incorrectly directs the user to Section 20.6.1.3.3 of ACI 318.
The 2018 IBC advises that ACI 318 is to be followed in addition to any requirements in the IBC by the reference to Chapter 19 of the IBC:

"1808.8 Concrete foundations. The design, materials and construction of concrete foundations shall comply with Sections 1808.8.1 through 1808.8.6 and the provisions of Chapter 19."

and Chapter 19 of the 2018 IBC reads:

" 1901.2 Plain and reinforced concrete. Structural concrete shall be designed and constructed in accordance with the requirements of this chapter and ACI 318…"

There is no reason to duplicate requirements of ACI 318 in the IBC.

With regard to removal of text, there are two provisions in the text of IBC Section 1808.2.

1. There are criteria for longitudinal reinforcement and bundled bars, but the requirements in the IBC refer the user to ACI 318 Section 20.6.1.3.4. This is unnecessary language due to the IBC language in Section 1808.8 and 1901.2 as shown above.

2. The IBC language provides a definition for concrete cover which is already addressed in ACI 318: "distance between the outermost surface of embedded reinforcement and the closest outer surface of the concrete." note that concrete cover is a specified dimension. Thus, where concrete is placed inside casings or mandrels the closest outer surface of the concrete is clearly the inside of the casing or mandrel.

With regard to the criteria in Table 1808.2, the requirements are shown as a side-by-side comparison in the Table below. The requirements remain identical for all concrete cover requirements for foundations except as follows:

1. Concrete cover for precast elements exposed to seawater is permitted to be 2 inches in ACI 318 where the 2018 IBC requires 3 inches and 2-1/2 inches for precast nonprestressed and prestressed, respectively. This modification recognizes the performance of centrifugally manufacturers precast concrete piles, which were probably not a consideration when the cover provisions were introduced into the 2018 IBC. Where additional information on cover requirements as related to manufacturing process and materials the commentary of ACI 318 directs the user to ACI 543R *Guide to Design, Manufacture, and Installation of Concrete Piles*. Now that centrifugally are becoming more commonplace, the code would be remiss in not providing for the minimum requirement that reflect current practice and materials. This lowers costs by recognizing the performance of piles manufactured using zero-slump concrete.

2. Where the 2018 IBC permits cover to be a little as 2.5 inches for deep foundations not enclosed by a steel pipe, tube or permanent casing, ACI 318 finds that the ability to assure proper cover in deep foundations is more challenging than that required for shallow foundations. ACI 318 requires the minimum cover to remain the same for deep foundations as that required for shallow foundations, 3 inches.

3. ACI 318 does not differentiate the minimum concrete cover requirements between deep foundations enclosed within a steel pipe, tube or permanent casing whether there is a structural steel core. Further ACI 318 does not consider the requirements for structural steel deep foundations to be with their purview. Section 1808.2 is retained to include the provisions for these deep foundation systems.

4. Research considered by ACI Committee 318 and Subcommittee 318-0F on Foundations showed comparable performance for cover of precast elements regardless of whether manufactured at a plant or site cast.

ACI, a 501.C.3 professional society recommends approval as submitted to reflect current concrete technology and to assure appropriate minimum requirements are provided for the protection of reinforcement.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
There is no significant increase in cost of construction. Cost is decreased for precast prestressed concrete piles by reducing cover and providing for acceptable performance of new technologies and materials. There may be a slight increase in costs where deep foundations are cast without casings or tubes because the cover is increased from 2-1/2 inches to 3 inches.

S120-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: Proponent requested disapproval. The committee disapproved the proposal due to a lack of coordination (example: micro piles) (Vote: 14-0)

Assembly Action: **None**

S120-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1808.8.2, TABLE 1808.8.2

Proponents:
Stephen Szoke, representing American Concrete Institute (steve.szoke@concrete.org)

requests As Modified by Public Comment

Replace as follows:

2018 International Building Code

1808.8.2 Concrete cover. The concrete cover provided for prestressed and nonprestressed reinforcement in foundations shall be not less than the largest applicable value specified in Table 1808.8.2. Longitudinal bars spaced less than $1^1/_2$ inches (38 mm) clear distance apart shall be considered to be bundled bars for which the concrete cover provided shall be not less than that required by Section ~~20.6.1.3.4~~ 20.5.1.3.5 of ACI 318. Concrete cover shall be measured from the concrete surface to the outermost surface of the steel to which the cover requirement applies. Where concrete is placed in a temporary or permanent casing or a mandrel, the inside face of the casing or mandrel shall be considered to be the concrete surface.

TABLE 1808.8.2
MINIMUM CONCRETE COVER

FOUNDATION ELEMENT OR CONDITION	MINIMUM COVER
1.Shallow foundations a. Cast-in-place non-prestressed concrete members b. Cast-in-place prestressed concrete members c. Precast non-prestressed or prestressed concrete members manufactured under plant conditions	 ~~In accordance with Section 20.6 of ACI 318~~ ACI 318: 20.5.1.3.1 ACI 318: 20.5.1.3.2 ACI 318: 20.5.1.3.3
2. Precast prestressed deep foundation elements~~Exposed to seawater Other~~	~~2.5 inchesIn accordance with Section 20.6.1.3.3 2 of~~ ACI 318: 20.5.1.3.4
3. Cast-in-place deep foundation elements not enclosed by a steel pipe, tube or permanent casing	~~2.5 inches~~ ACI 318: 20.5.1.3.4
~~4. Case-in-place deep foundation elements not enclosed by a steel pite, tube or permanent casing~~	~~2.5 inches~~
4~~5~~. Cast-in-place deep foundation elements enclosed by a steel pipe, tube or permanent casing	1 inch
5~~6~~. Structural steel core within a steel pipe, tube or permanent casing	2 inches
~~7. Cast-in-place drilled shafts enclosed by a stable rock socket~~	~~1.5 inches~~

For SI: 1 inch = 25.4 mm.

Commenter's Reason: The committee recommended disapproval of the orginal code change. The disapproval resulted in incorrect references to section of ACI 318-19. This pulblc comment inserts the correct references. This code change proposal aligns the IBC with the appropriate sections of ACI 318. ACI 318 Section 20.5.2.3.4 addresses deep foundations made of either cast-in-place and precast concrete and thus the separate delineation in Table of the IBC is no longer required. Further provisions in ACI 318 are more complete for foundation systems, allowing different cover for various exposures.
Table 20.5.1.3.4—Specified concrete cover for deep foundation members in ACI 318 provides that:

Concrete exposure	Deep foundation member type	Reinforcement	Specified cover, in.
Cast against and permanently in contact with ground, not enclosed by steel pipe, tube permanent casing, or stable rock socket	Cast-in-place	All	3
Enclosed by steel pipe, tube,permanent casing, or stable rock socket	Cast-in-place	All	1-1/2
Permanently in contact with ground	precast-nonprestressed	All	1-1/2
	precast-prestressed		
Exposed to sea water	Precast-non-prestressed	All	2-1/2
	Precast-prestressed	All	2

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
Generally, this code change proposal will not increase or decrease cost of construction as for the most part, this change only corrects references to ACI 318. However, by referencing ACI 318, more exposure conditions are addressed, several requiring less concrete cover than required in the 2018 IBC. The IBC does not address as many exposures as ACI 318.

S123-19

IBC: 1810.2.4.1, 1810.3.2.1(New), 1810.3.2.1.1, 1810.3.2.1.2, 1810.3.2.2, 1810.3.8, 1810.3.8.1, 1810.3.8.2, 1810.3.8.2.1, 1810.3.8.2.2, 1810.3.8.2.3, 18 1818.3.9.3, 1810.3.9.4, 1810.3.9.4.1, 1810.3.9.4.2, 1810.3.9.4.2.1, 1810.3.9.4.2.2

Proposed Change as Submitted

Proponents: Stephen Szoke, representing American Concrete Institute (steve.szoke@concrete.org)

2018 International Building Code

Revise as follows:

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 1613.2.2, shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains modified for soil-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 .~~ 18.13.5.10.5 in ACI 318.
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 .~~ Section 18.13.5.5 of ACI 318.

Add new text as follows:

1810.3.2.1 Concrete. Concrete materials shall conform to ACI 318.

Revise as follows:

~~**1810.3.2.1**~~ **1810.3.2.1.1 Concrete cast in steel pipe.** Where concrete is cast in a steel pipe or where an enlarged base is formed by compacting concrete, the maximum size for coarse aggregate shall be 3/4 inch (19.1 mm). Concrete to be compacted shall have a zero slump.

Delete without substitution:

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

~~**1810.3.2.1.2 ACI 318 Equation (25.7.3.3).** Where this chapter requires detailing of concrete deep foundation elements in accordance with Section 18.7.5.4 of ACI 318, compliance with Equation (25.7.3.3) of ACI 318 shall not be required.~~

~~**1810.3.2.2 Prestressing steel.** Prestressing steel shall conform to ASTM A416.~~

Revise as follows:

1810.3.8 Precast concrete piles. Precast concrete piles shall be designed and detailed in accordance with ~~Sections 1810.3.8.1 through 1810.3.8.3.~~ ACI 318.

Exception: For structures assigned to Seismic Design Category C, D, E or F, the minimum spiral reinforcement index required by Section 18.13.5.10.4 and 18.13.5.10.5 of ACI 318 shall not apply in cases where the design includes full consideration of load combinations specified in ASCE 7, Section 2.3.6 and the applicable overstrength factor, Ω0. In such cases, minimum spiral reinforcement index shall be as specified in Section 13.4.5.6 of ACI 318.

Delete without substitution:

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

~~1.At not more than 1 inch (25 mm) for the first five ties or spirals at each end; then~~
~~2.At not more than 4 inches (102 mm), for the remainder of the first 2 feet (610 mm) from each end; and then~~
~~3.At not more than 6 inches (152 mm) elsewhere.~~

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

~~1.For piles having a least horizontal dimension of 16 inches (406 mm) or less, wire shall not be smaller than 0.22 inch (5.6 mm) (No. 5 gage).~~

~~2.For piles having a least horizontal dimension of more than 16 inches (406 mm) and less than 20 inches (508 mm), wire shall not be smaller than 0.238 inch (6 mm) (No. 4 gage).~~

~~3.For piles having a least horizontal dimension of 20 inches (508 mm) and larger, wire shall not be smaller than $^{1}/_{4}$ inch (6.4 mm) round or 0.259 inch (6.6 mm) (No. 3 gage).~~

~~**1810.3.8.2 Precast nonprestressed piles.** Precast nonprestressed concrete piles shall comply with the requirements of Sections 1810.3.8.2.1 through 1810.3.8.2.3.~~

~~**1810.3.8.2.1 Minimum reinforcement.** Longitudinal reinforcement shall consist of not fewer than four bars with a minimum longitudinal reinforcement ratio of 0.008.~~

~~**1810.3.8.2.2 Seismic reinforcement in *Seismic Design Categories* C through F.** For structures assigned to *Seismic Design Category* C, D, E or F, precast nonprestressed piles shall be reinforced as specified in this section. The minimum longitudinal reinforcement ratio shall be 0.01 throughout the length. Transverse reinforcement shall consist of closed ties or spirals with a minimum 3/8 inch (9.5 mm) diameter. Spacing of transverse reinforcement shall not exceed the smaller of eight times the diameter of the smallest longitudinal bar or 6 inches (152 mm) within a distance of three times the least pile dimension from the bottom of the pile cap. Spacing of transverse reinforcement shall not exceed 6 inches (152 mm) throughout the remainder of the pile.~~

~~**1810.3.8.2.3 Additional seismic reinforcement in *Seismic Design Categories* D through F.** For structures assigned to *Seismic Design Category* D, E or F, transverse reinforcement shall be in accordance with Section 1810.3.9.4.2 .~~

~~**1810.3.8.3 Precast prestressed piles.** Precast prestressed concrete piles shall comply with the requirements of Sections 1810.3.8.3.1 through 1810.3.8.3.3.~~

~~**1810.3.8.3.1 Effective prestress.** The effective prestress in the pile shall be not less than 400 psi (2.76 MPa) for piles up to 30 feet (9144 mm) in length, 550 psi (3.79 MPa) for piles up to 50 feet (15 240 mm) in length and 700 psi (4.83 MPa) for piles greater than 50 feet (15 240 mm) in length.~~

~~Effective prestress shall be based on an assumed loss of 30,000 psi (207 MPa) in the prestressing steel. The tensile stress in the prestressing steel shall not exceed the values specified in ACI 318.~~

~~**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.~~

$$\cancel{\rho_s = 0.04(f'_c/f_{yh})[2.8 + 2.34P/f'_c A_g)]}$$ ~~**(Equation 18-5)**~~

~~where:~~

~~A_g = Pile cross-sectional area 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 = Spiral reinforcement index or volumetric ratio (vol. spiral/vol. core).~~

~~Not less than one-half the volumetric ratio required by Equation 18-5 shall be provided below the upper 20 feet (6096 mm) of the pile.~~

> ~~**Exception:** The minimum spiral reinforcement index required by Equation 18-5 shall not apply in cases where the design includes full consideration of load combinations specified in ASCE 7, Section 2.3.6 and the applicable overstrength factor, Ω_0. In such cases, minimum spiral reinforcement index shall be as specified in Section 1810.3.8.1.~~

~~**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:~~

~~ρ s = 0 . 06 (f ′ c / f y h) [2 . 8 + 2 . 34 P / f ′ c A g)]~~ $\rho_s = 0.06(f'_c / f_{yh})[2.8 + 2.34P/f'_c A_g)]$ **~~(Equation 18-6)~~**

~~but not exceed:~~

~~ρ s = 0 . 021~~ $\rho_s = 0.021$ **~~(Equation 18-7)~~**

~~where:~~

~~A_g = Pile cross-sectional area, 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.~~

> **~~Exception:~~** ~~The minimum spiral reinforcement required by Equation 18-6 shall not apply in cases where the design includes full consideration of load combinations specified in ASCE 7, Section 2.3.6 and the applicable overstrength factor, Ω_0. In such cases, minimum spiral reinforcement shall be as specified in Section 1810.3.8.1.~~

~~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 s h = 0 . 3 s h c (f ′ c / f y h) (A g / A c h - 1 . 0) [0 . 5 + 1 . 4 P / (f ′ c A g)]~~ $A_{sh} = 0.3s\, h_c\, (f'_c / f_{yh})(A_g / A_{ch} - 1.0)[0.5 + 1.4P/(f'_c A_g)]$ **~~(Equation 18-8)~~**

~~but not less than:~~

~~A s h = 0 . 3 s h c (f ′ c / f y h) (A g / A c h - 1 . 0) [0 . 5 + 1 . 4 P / (f ′ c A g)]~~ $A_{sh} = 0.12s\, h_c\, (f'_c / f_{yh})\, [0.5 + 1.4P/(f'_c A_g)]$ **~~(Equation 18-9)~~**

~~where:~~

~~f_{yh} = yield strength of transversereinforcement ≤70,000 psi (483 MPa).~~

~~h_c = Cross-sectional dimension of pile core measured center to center of hoop reinforcement, inch (mm).~~

~~s = Spacing of transverse reinforcement measured along length of pile, inch (mm).~~

~~A_{sh} = Cross-sectional area of tranverse reinforcement, square inches (mm^2).~~

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

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

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

~~1810.3.8.3.4 Axial load limit in Seismic Design Categories C through F.~~ ~~For structures assigned to *Seismic Design Category* C, D, E, or F, the maximum factored axial load on precast prestressed piles subjected to a combination of seismic lateral force and axial load shall not exceed the following values:~~

~~1.0.2$f'_c A_g$ for square piles~~
~~2.0.4$f'_c A_g$ for circular or octagonal piles~~

Revise as follows:

1810.3.9 Cast-in-place deep foundations. Cast-in-place deep foundation elements shall be designed and detailed in accordance with Sections 1810.3.9.1 through ~~1810.3.9.6.~~ 1810.3.9.4.

Delete without substitution:

~~1810.3.9.1 Design cracking moment.~~ ~~The design cracking moment (ΦM_n) for a cast-in-place deep foundation element not enclosed by a~~

~~structural steel pipe or tube shall be determined using the following equation:~~

~~ϕ Mn = 3 f ' c S m~~ $$\phi M_n = 3\sqrt{f'_c}S_m$$ ~~**(Equation 18-10)**~~

~~For SI:~~

$$\phi M_n = 0.25\sqrt{f'_c}S_m$$

~~where:~~

~~f'_c = Specified compressive strength of concrete or grout, psi (MPa).~~

~~S_m = Elastic section modulus, neglecting reinforcement and casing, cubic inches (mm^3).~~

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

Revise as follows:

~~**1810.3.9.3**~~ **1810.3.9.1** **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 that the construction method can be demonstrated to the satisfaction of the *building official*.

~~**1810.3.9.4**~~ **1810.3.9.2** **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* C, D, E , or F, reinforcement shall be provided in accordance with Section ~~1810.3.9.4.2.~~ 18.13.5.7 of ACI 318.

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~~ ACI 318 exceeds the required moment strength determined using the load combinations with overstrength factor in Section 2.3.6 or 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~~ 18.3.5.7.1 of ACI 318 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.

Delete without substitution:

~~**1810.3.9.4.1 Seismic reinforcement in *Seismic Design Category* C.** For structures assigned to *Seismic Design Category* C, cast-in-place deep foundation elements shall be reinforced as specified in this section. Reinforcement shall be provided where required by analysis.~~

~~Not fewer than four longitudinal bars, with a minimum longitudinal reinforcement ratio of 0.0025, shall be provided throughout the minimum reinforced length of the element as defined in this section starting at the top of the element. The minimum reinforced length of the element shall be taken as the greatest of the following:~~

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

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

~~Exceptions:~~

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

~~1810.3.9.4.2 Seismic reinforcement in Seismic Design Categories D through F.~~ ~~For structures assigned to *Seismic Design Category* D, E or F, cast-in-place deep foundation elements shall be reinforced as specified in this section. Reinforcement shall be provided where required by analysis.~~

~~Not fewer than four longitudinal bars, with a minimum longitudinal reinforcement ratio of 0.005, shall be provided throughout the minimum reinforced length of the element as defined in this section starting at the top of the element. The minimum reinforced length of the element shall be taken as the greatest of the following:~~

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

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

~~1.12 longitudinal bar diameters.~~
~~2.One-half the least dimension of the element.~~
~~3.12 inches (305 mm).~~

~~Exceptions:~~

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

~~1810.3.9.4.2.1 Site Classes A through D.~~ ~~For *Site Class* A, B, C or D sites, transverse confinement reinforcement shall be provided in the element in accordance with Sections 18.7.5.2, 18.7.5.3 and 18.7.5.4 of ACI 318 within three times the least element dimension of the bottom of the pile cap. A transverse spiral reinforcement ratio of not less than one-half of that required in Section 18.7.5.4(a) of ACI 318 shall be permitted.~~

~~1810.3.9.4.2.2 Site Classes E and F.~~ ~~For *Site Class* E or F sites, transverse confinement reinforcement shall be provided in the element in accordance with Sections 18.7.5.2, 18.7.5.3 and 18.7.5.4 of ACI 318 within seven times the least element dimension of the pile cap and within seven times the least element dimension of the interfaces of strata that are hard or stiff and strata that are liquefiable or are composed of soft- to medium-stiff clay.~~

Reason: This Code change includes revisions and additions to the Code in an effort to eliminate conflicting provisions in ACI 318-14, ASCE 7-16 and IBC-2018 regarding design of deep foundations for earthquake resistant structures. Subcommittee F, Foundations, of ACI 318 has coordinated efforts with members from ASCE 7 to bring the concrete material design requirements for foundations to one location. ASCE 7 started this effort in their cycle ending in 2016. The changes to ACI 318 shown here is the continuation of that effort. A side-by-side comparison is provided, however, difficult to follow with all the changes and dissimilar format. For a more comprehensive look at the changes in ACI 318, please review the public comment version available at https://www.concrete.org/publications/standards/upcomingstandards.aspx
Summary of code change proposals:

- Section 1810.2.4.1 is updated to the latest version of ACI 318.

· The sections in Materials for the design and detailing of deep foundations were updated to the latest edition of ACI 318.

o Section 1810.3.2.1: A general reference to ACI 318 is made and the existing requirement is moved to 1810.3.2.1.1 as it is not covered in ACI 318.

o Section 1810.3.2.1.1: Is covered by Section 18.13.5.4 in ACI 318.

o Section 1810.3.2.1.2: Is covered by Section 25.7.3.3 in ACI 318.

o Section 1810.3.2.2: Is covered by Section 20.3 in ACI 318.

· Section 18.10.3.8, Precast Concrete pile, was adopted by ACI 318. The exception for minimum spiral reinforcement was retained from Sections 1810.3.8.3.2 and 1810.3.8.3.3 with the appropriate references to ACI 318. The requirements for 18.10.3.8 mostly went to Section 13.4.5 and 18.3.5 of ACI 318. A comparison is provided but for a full review please reference the public comment version of ACI 318.

o Section 1810.3.8.1→13.4.5.2 and 13.4.5.6

o Section 1810.3.8.2.1→13.4.5.3

o Section 1810.3.8.2.2→18.13.5.10.2

§ Exception remains

o Section 1810.3.8.2.3→18.13.5.10.3

§ Exception remains

o Section 1810.3.8.3.1→13.4.5.4 and 13.4.5.5

o Section 1810.3.8.3.2→18.13.5.10.4

o Section 1810.3.8.3.3→18.13.5.10.5

o Section 1810.3.8.3.4→18.13.5.10.6

· Section 18.10.3.9, Cast-in-place deep foundation, was adopted by ACI 318.

o Section 1810.3.9.1→13.4.4

o Section 1810.3.9.2→13.4.4

o Section 1810.3.9.3→Remains

o Section 1810.3.9.4→Remains, update reference

o Section 1810.3.9.4.1→18.13.5.7

§ Exception→18.13.5.8

o Section 1810.3.9.4.2→18.13.5.7

§ Exception→18.13.5.8

o Section 1810.3.9.4.2.1→18.13.5.5

o Section 1810.3.9.4.2.2→18.13.5.5

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<table>
<tr><td>1810.3.2.1.1 Seismic hooks. For structures assigned to Seismic Design Category C, D, E or F, the ends of hoops, spirals and ties used in concrete deep foundation elements shall be terminated with seismic hooks, as defined in ACI 318, and shall be turned into the confined concrete core.</td><td>18.13.5.4 For structures assigned to SDC C, D, E, or F, hoops, spirals, and ties in deep foundation members shall be terminated with seismic hooks.</td></tr>
<tr><td>1810.3.2.1.2 ACI 318 Equation (25.7.3.3). Where this chapter requires detailing of concrete deep foundation elements in accordance with Section 18.7.5.4 of ACI 318, compliance with Equation (25.7.3.3) of ACI 318 shall not be required.</td><td>25.7.3.3 Except for transverse reinforcement in deep foundations, the volumetric spiral reinforcement ratio ρ_s shall satisfy Eq. (25.7.3.3).</td></tr>
<tr><td>1810.3.2.2 Prestressing steel. Prestressing steel shall conform to ASTM A416.</td><td>20.3 Prestressing strands, wires, and bars

20.3.1 Material properties

20.3.1.1 Except as required in 20.3.1.3 for special moment frames and special structural walls, prestressing reinforcement shall conform to (a), (b), (c), or (d):

(a) ASTM A416 – strand

(b) ASTM A421 – wire

(c) ASTM A421 – low-relaxation wire including Supplementary Requirement S1, “Low-Relaxation Wire and Relaxation Testing”

(d) ASTM A722 – high-strength bar</td></tr>
<tr><td>1810.3.8 Precast concrete piles. Precast concrete piles shall be designed and detailed in accordance with Sections 1810.3.8.1 through 1810.3.8.3.

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

1. At not more than 1 inch (25 mm) for the first five ties or spirals at each end; then

2. At not more than 4 inches (102 mm), for the remainder of the first 2 feet (610 mm) from each end; and then

3. At not more than 6 inches (152 mm) elsewhere.

The size of ties and spirals shall be as follows:

1. For piles having a least horizontal dimension of 16 inches (406 mm) or less, wire shall not be smaller than 0.22 inch (5.6 mm) (No. 5 gage).

2. For piles having a least horizontal dimension of more than 16 inches (406 mm) and less than 20 inches (508 mm), wire shall not be smaller than 0.238 inch (6 mm) (No. 4 gage).

3. For piles having a least horizontal dimension of 20 inches (508 mm) and larger, wire shall not be smaller than 1/4 inch (6.4 mm) round or 0.259 inch (6.6 mm) (No. 3 gage).

1810.3.8.2 Precast nonprestressed piles. Precast</td><td>13.4.5 Precast concrete piles

13.4.5.1 Precast concrete piles supporting buildings assigned to SDC A or B shall satisfy the requirements of 13.4.5.2 through 13.4.5.6.

13.4.5.2 Longitudinal reinforcement shall be arranged in a symmetrical pattern.

13.4.5.3 For precast nonprestressed piles, longitudinal reinforcement shall be provided according to (a) and (b):

(a) Minimum of 4 bars

(b) Minimum area of $0.008A_g$

13.4.5.4 For precast prestressed piles, the effective prestress in the pile shall provide a minimum average compressive stress in the concrete in accordance with Table 13.4.5.4.

Table 13.4.5.4 Minimum compressive stress in precast prestressed piles

<table><tr><td>Pile length (ft)</td><td>Minimum compressive stress (psi)</td></tr><tr><td>Pile length ≤ 30</td><td>400</td></tr><tr><td>30 < Pile length ≤ 50</td><td>550</td></tr><tr><td>Pile length > 50</td><td>700</td></tr></table></td></tr>
</table>

nonprestressed concrete piles shall comply with the requirements of Sections 1810.3.8.2.1 through 1810.3.8.2.3.

1810.3.8.2.1 Minimum reinforcement. Longitudinal reinforcement shall consist of not fewer than four bars with a minimum longitudinal reinforcement ratio of 0.008.

...

1810.3.8.3 Precast prestressed piles. Precast prestressed concrete piles shall comply with the requirements of Sections 1810.3.8.3.1 through 1810.3.8.3.3

1810.3.8.3.1 Effective prestress. The effective prestress in the pile shall be not less than 400 psi (2.76 MPa) for piles up to 30 feet (9144 mm) in length, 550 psi (3.79 MPa) for piles up to 50 feet (15 240 mm) in length and 700 psi (4.83 MPa) for piles greater than 50 feet (15 240 mm) in length. Effective prestress shall be based on an assumed loss of 30,000 psi (207 MPa) in the prestressing steel. The tensile stress in the prestressing steel shall not exceed the values specified in ACI 318.

13.4.5.5 For precast prestressed piles, the effective prestress in the pile shall be calculated based on an assumed total loss of 30,000 psi in the prestressed reinforcement.

13.4.5.6 The longitudinal reinforcement shall be enclosed by transverse reinforcement according to Table 13.4.5.6(a) and shall be spaced according to Table 13.4.5.6(b):

Table 13.4.5.6(a) Minimum transverse reinforcement size

Least horizontal pile dimension-h (in.)	Minimum wire size transverse reinforcement[1]
$h \leq 16$	W4, D4
$16 < h < 20$	W4.5, D5
$h \geq 20$	W5.5, D6

[1] If bars are used, minimum of #3 bar applies to all values of h

Table 13.4.4.6(b) Maximum transverse reinforcement spacing

Reinforcement location in the pile	Maximum center-to-center spacing (in.)
First five ties or spirals at each end of pile	1
24 in. from each end of pile	4
Remainder of pile	6

1810.3.9 Cast-in-place deep foundations. Cast-in-place deep foundation elements shall be designed and detailed in accordance with Sections 1810.3.9.1 through 1810.3.9.6. **1810.3.9.1** Design cracking moment. The design cracking moment (ϕM_n) for a cast-in-place deep foundation element not enclosed by a structural steel pipe or tube shall be determined using the following equation: $\phi M_n = 3\sqrt{f_c'} S_m$ (Equation 18-10) For SI: $0.25\sqrt{f_c'} S_m$ where: f_c' = Specified compressive strength of concrete or grout, psi (MPa). S_m = Elastic section modulus, neglecting reinforcement and casing, cubic inches (mm^3). **1810.3.9.2** Required reinforcement. Where subject to uplift or where the required moment strength determined using the load combinations of Section 1605.2 exceeds the design cracking moment determined in accordance with Section 1810.3.9.1, cast-in-place deep foundations not enclosed by a structural steel pipe or tube shall be reinforced.	**13.4.4** Cast-in-place deep foundations **13.4.4.1** Cast-in-place deep foundations that are subject to uplift or where M_u is greater than 0.4 M_{cr} shall be reinforced, unless enclosed by a structural steel pipe or tube. Note $f_{cr} = 7.5\sqrt{f_c'}$
1810.3.9.4.1 Seismic reinforcement in Seismic Design Category C. For structures assigned to Seismic Design Category C, cast-in-place deep foundation elements shall be reinforced as specified in this section. Reinforcement shall be provided where required by analysis. Not fewer than four longitudinal bars, with a minimum longitudinal reinforcement ratio of 0.0025, shall be provided throughout the minimum reinforced length of the element as defined in this section starting at the top of the element. The minimum reinforced length of the element shall be taken as the greatest of the following: 1. One-third of the element length. 2. A distance of 10 feet (3048 mm). 3. Three times the least element dimension. 4. The distance from the top of the element to the point where the design cracking moment determined in accordance with Section 1810.3.9.1 exceeds the required moment strength determined using the load combinations of Section 1605.2. Transverse reinforcement shall consist of closed ties or spirals with a minimum 3/8 inch (9.5 mm) diameter. Spacing of transverse reinforcement shall not exceed the smaller of 6 inches (152 mm) or 8- longitudinal-bar diameters, within a distance of three times the least element dimension from the bottom of the pile cap. Spacing of transverse reinforcement shall not exceed 16 longitudinal bar diameters throughout the remainder of the	**18.13.5** Deep Foundations **18.13.5.1** This section shall apply to the following types of deep foundations (a) uncased cast-in-place concrete drilled or augered piles (b) metal cased concrete piles (c) concrete filled pipe piles (d) precast concrete piles **18.13.5.2** For structures assigned to SDC C, D, E, or F, piles, piers, or caissons resisting tension loads shall have continuous longitudinal reinforcement over their length resisting to resist design tension forces. **18.13.5.3** For structures assigned to SDC C, D, E, or F, the minimum longitudinal and transverse reinforcement required by 18.13.5.7 through 18.13.5.10 shall be extended over the entire unsupported length for the portion of pile in air or water, or in soil that is not capable of providing adequate lateral restraint to prevent buckling throughout this length. **18.13.5.4** For structures assigned to SDC C, D, E, or F, hoops, spirals, and ties in deep foundation members shall be terminated with seismic hooks.

reinforced length.

Exceptions:

1. The requirements of this section shall not apply to concrete cast in structural steel pipes or tubes.

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

1810.3.9.4.2 Seismic reinforcement in Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E or F, cast-in-place deep foundation elements shall be reinforced as specified in this section. Reinforcement shall be provided where required by analysis. Not fewer than four longitudinal bars, with a minimum longitudinal reinforcement ratio of 0.005, shall be provided throughout the minimum reinforced length of the element as defined in this section starting at the top of the element. The minimum reinforced length of the element shall be taken as the greatest of the following:

1. One-half of the element length.

2. A distance of 10 feet (3048 mm).

3. Three times the least element dimension.

4. The distance from the top of the element to the point where the design cracking moment determined in accordance with Section 1810.3.9.1 exceeds the required moment strength determined using the load combinations of Section 1605.2.

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

1. 12 longitudinal bar diameters.

2. One-half the least dimension of the element.

3. 12 inches (305 mm).

Exceptions:

1. The requirements of this section shall not apply to concrete cast in structural steel pipes or tubes.

2. A spiral-welded metal casing of a thickness not less than manufacturer's standard No. 14 gage (0.068 inch) is permitted to provide concrete confinement in lieu of the

18.13.5.5 For structures assigned to SDC D, E, or F or located in Site Class E or F, concrete piles shall have transverse reinforcement in accordance with 18.7.5.2, 18.7.5.3, and Table 18.7.5.4(e) within seven pile diameters above and below the interfaces between strata that are hard or stiff and strata that are liquefiable or soft.

18.13.5.6 For structures assigned to SDC D, E, or F, in foundations supporting one- and two-story stud bearing wall construction, concrete piles, piers or caissons, and foundation ties are exempt from the transverse reinforcement requirements of 18.13.5.3 through 18.13.5.5.

18.13.5.7 Uncased cast-in-place drilled or augered concrete piles or piers

18.13.5.7.1 For structures assigned to SDC C, D, E, or F, reinforcement shall be provided in uncased cast-in-place drilled or augered concrete piles where required by analysis and in accordance with the requirements in Table 18.13.5.7.1.

18.13.5.7.2 Minimum longitudinal and transverse reinforcement shall be provided along minimum reinforced lengths measured from the top of the pile in accordance with Table 18.13.5.7.1.

18.13.5.7.3 Longitudinal reinforcement shall extend at least the development length in tension beyond the flexural length of the pile, which is defined in Table 18.13.5.7.1 as the distance from the bottom of the pile cap to where $0.4M_{cr} > M_u$.

18.13.5.8 Metal-cased concrete piles

18.13.5.8.1 For structures assigned to SDC C, D, E, or F, longitudinal reinforcement requirements and minimum reinforced lengths for metal-cased concrete piles shall be the same as for uncased concrete piles in 18.13.5.7

18.13.5.8.2 Metal-cased concrete piles shall have a spiral-welded metal casing of a thickness not less than 0.0747 in. (No. 14 gauge) that is adequately protected from possible deleterious action due to soil constituents, changing water levels, or other factors indicated by boring records of site conditions.

18.13.5.9 Concrete-filled pipe piles

18.13.5.9.1 For structures assigned to SDC C, D, E or F, concrete-filled pipe piles shall have longitudinal reinforcement in the top of the pile with a total area of at least $0.01A_g$ and with a minimum length within the pile equal to two times the required embedment length into the pile cap, but not less than the development length in tension of the reinforcement.

closed ties or spirals. Where used as such, the metal casing shall be protected against possible deleterious action due to soil constituents, changing water levels or other factors indicated by boring records of site conditions. **1810.3.9.4.2.1** Site Classes A through D. For Site Class A, B, C or D sites, transverse confinement reinforcement shall be provided in the element in accordance with Sections 18.7.5.2, 18.7.5.3 and 18.7.5.4 of ACI 318 within three times the least element dimension of the bottom of the pile cap. A transverse spiral reinforcement ratio of not less than one-half of that required in Section 18.7.5.4(a) of ACI 318 shall be permitted. **1810.3.9.4.2.2** Site Classes E and F. For Site Class E or F sites, transverse confinement reinforcement shall be provided in the element in accordance with Sections 18.7.5.2, 18.7.5.3 and 18.7.5.4 of ACI 318 within seven times the least element dimension of the pile cap and within seven times the least element dimension of the interfaces of strata that are hard or stiff and strata that are liquefiable or are composed of soft- to medium-stiff clay.	

Table 18.13.5.7.1 Minimum reinforcement for uncased cast-in-place or augered concrete piles or pier:

Minimum Reinforcement		**SDC C –** **All Site Classes**	**SDC D, E, and F –** **Site Class A, B, C, and D**	**SDC D, E, and F –** **Site Class E and F**
Minimum Longitudinal Reinforcement Ratio (minimum number of bars)		0.0025 (minimum number of bars in accordance with 10.7.3.1)	0.005 (minimum number of bars in accordance with 10.7.3.1)	0.005 (minimum number of bars in accordance with 10.7.3.1)
Minimum Reinforced Pile Length		Longest of (a) through (d): (a) 1/3 pile length (b) 10 ft. (c) 3 times the pile diameter (d) flexural length of pile - distance from bottom of pile cap to where 0.4$\mathbf{M_{cr}}$ exceeds $\mathbf{M_u}$.	Longest of (a) through (d): (a) ½ pile length (b) 10 ft. (c) 3 times the pile diameter (d) flexural length of pile - distance from bottom of pile cap to where 0.4$\mathbf{M_{cr}}$ exceeds $\mathbf{M_u}$.	Full length of pile except in accordance with [1] or [2].
Transverse Confinement Reinforcement Zone	Length of Reinforcement Zone	3 times the pile diameter from the bottom of the pile cap	3 times the pile diameter from the bottom of the pile cap.	7 times the pile diameter from the bottom of the pile cap.
	Type of Transverse Reinforcement	Closed ties or spirals with a minimum 3/8 in. diameter.	Minimum of No. 3 closed tie or 3/8 in. diameter spiral for piles ≤ 20 in. diameter. Minimum No. 4 closed tie or 1/2 in. diameter spiral for piles > 20 in. diameter.	
			In accordance with 18.7.5.2	

	Spacing and Amount of Transverse Reinforcement	Spacing shall not exceed lesser of 6 in. or 8 longitudinal bar diameters	In accordance with 18.7.5.3 and not less than one-half the requirement of Table 18.7.5.4(e)	In accordance with 18.7.5.3 and not less than the requirement of Table 18.7.5.4(e).
Transverse Reinforcement in Remainder of Reinforced Pile Length	Type of Transverse Reinforcement	Closed ties or spirals with minimum 3/8 in. diameter.	Minimum of No. 3 closed tie or 3/8 in. diameter spiral for piles ≤ 20 in. diameter. Minimum of No. 4 closed tie or 1/2 in. diameter spiral for piles > 20 in. diameter.	
			In accordance with 18.7.5.2	
	Spacing and Amount of Transverse Reinforcement	Maximum spacing of 16 longitudinal bar diameters.	Spacing shall not exceed the least of (a) through (c): (a) 12 longitudinal bar diameters (b) ½ the pile diameter (c) 12 in.	

[1] For piles sufficiently embedded in firm soil or rock, reinforcement shall be permitted to be terminated a length above the tip equal to the lesser of 5 percent of the pile length and 33 percent of the length of the pile within rock or firm soil.

[2] In lieu of providing full length minimum flexural reinforcement, the deep foundation element shall be designed to withstand maximum imposed curvatures from the earthquake ground motions and structural 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. Minimum reinforced length shall not be less than the requirement for SDC D, E, or F; Site Class D.

1810.3.8.2 Precast nonprestressed piles. ... **1810.3.8.2.2** Seismic reinforcement in Seismic Design Categories C through F. For structures assigned to Seismic Design Category C, D, E or F, precast nonprestressed piles shall be reinforced as specified in this section. The minimum longitudinal reinforcement ratio shall be 0.01 throughout the length. Transverse reinforcement shall consist of closed ties or spirals with a minimum 3/8 inch (9.5 mm) diameter. Spacing of transverse reinforcement shall not exceed the smaller of eight times the diameter of the smallest longitudinal bar or 6 inches (152 mm) within a distance of three times the least pile dimension from the bottom of the pile cap. Spacing of transverse reinforcement shall not exceed 6 inches (152 mm) throughout the remainder of the pile. **1810.3.8.2.3** Additional seismic reinforcement in Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E or F, transverse reinforcement shall be in accordance with Section 1810.3.9.4.2. ... **1810.3.8.3** Precast prestressed piles. ...	**18.13.5.10** Precast concrete piles **18.13.5.10.1** For precast concrete driven piles, the length of transverse reinforcement provided shall be sufficient to account for potential variations in the elevation of pile tips. **18.13.5.10.2** Precast nonprestressed concrete piles for structures assigned to SDC C shall satisfy (a) through (d): (a) Minimum longitudinal steel reinforcement ratio shall be 0.01 (b) Longitudinal reinforcement shall be enclosed within a minimum of No. 3 closed ties or 3/8-in. diameter spirals, for up to 20-in. diameter piles, and No. 4 closed ties or ½-in. diameter spirals, for larger diameter piles (c) Spacing of transverse reinforcement within a distance of 3 times the least cross-sectional dimension of the pile from the bottom of the pile cap shall not exceed the lesser of 8 times the diameter of the smallest longitudinal bar and 6 in. (d) Transverse reinforcement shall be provided throughout the length of the pile at a spacing not exceeding 6 in.

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.

$\rho_s = 0.04(f_c'/f_{yh})[2.8 + 2.34P/ f_c'A_g)]$ (Equation 18-5)

where:

A_g = Pile cross-sectional area 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 = Spiral reinforcement index or volumetric ratio (vol. spiral/vol. core).

Not less than one-half the volumetric ratio required by Equation 18-5 shall be provided below the upper 20 feet (6096 mm) of the pile.

Exception: The minimum spiral reinforcement index required by Equation 18-5 shall not apply in cases where the design includes full consideration of load combinations specified in ASCE 7, Section 2.3.6 and the applicable overstrength factor, Ω_0. In such cases, minimum spiral reinforcement index shall be as specified in Section 1810.3.8.1.

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.

18.13.5.10.3 For structures assigned to SDC D, E, or F, precast nonprestressed concrete piles shall satisfy the requirements of 18.13.5.10.2 and the requirements for uncased cast-in-place or augered concrete piles in SDC D, E, or F in Table 18.13.5.7.1.

18.13.5.10.4 For structures assigned to SDC C, precast prestressed concrete piles shall satisfy (a) and (b):

(a) If the transverse reinforcement consists of spirals or circular hoops, the volumetric ratio of transverse reinforcement, ρ_s, in the upper 20 ft shall not be less than that calculated by Eq. (18.13.5.10.4a) or calculated from a more detailed analysis by Eq. (18.13.5.10.4b):

$0.15(f_c'/f_{yt})$ (18.13.5.10.4a)

$0.04(f_c'/f_{yt})[2.8 + 2.3P_u/ f_c'A_g)]$ (18.13.5.10.4b)

and f_{yt} shall not be taken greater than 100,000 psi

(b) A minimum of one-half of the volumetric ratio of spiral reinforcement required by Eq. (18.13.5.10.4a) or Eq. (18.13.5.10.4b) shall be provided for the remaining length of the pile.

18.13.5.10.5 For structures assigned to SDC D, E, or F, precast prestressed concrete piles shall satisfy (a) through (e) and the ductile pile region shall be defined as the length of pile measured from the bottom of the pile cap to the point of zero curvature plus 3 times the least pile dimension, but not less than 35 ft. If the total pile length in the soil is 35 ft or less, the ductile pile region shall be taken as the entire length of the pile:

(a) In the ductile pile region, the center-to-center spacing of spirals or hoop reinforcement shall not exceed the least of 0.2 times the least pile dimension, 6 times the diameter of the longitudinal strand, and 6 in.

(b) Spiral reinforcement shall be spliced by lapping one full turn, by welding, or by the use of a mechanical splice. If spiral reinforcement is lap spliced, the ends of the spiral shall terminate in a seismic hook. Mechanical and welded splices of deformed bars shall comply with 25.5.7.

(c) If the transverse reinforcement consists of spirals, or circular hoops, the volumetric ratio of transverse reinforcement, ρ_s, in the ductile pile region shall not be less than that calculated by Eq. (18.13.5.10.5a) or calculated from a more detailed analysis by Eq. (18.13.5.10.5b), and the required volumetric ratio shall be permitted to be obtained by providing an inner and outer spiral.

$0.2(f_c'/f_{yt})$ (18.13.5.10.5a)

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:

$\rho_s = 0.06(f_c'/f_{yh})[2.8 + 2.34P/ f_c'A_g)]$ (Equation 18-6) but not exceed: $\rho_s = 0.021$ (Equation 18-7)

where:

A_g = Pile cross-sectional area, square inches (mm2).

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.

Exception: The minimum spiral reinforcement required by Equation 18-6 shall not apply in cases where the design includes full consideration of load combinations specified in ASCE 7, Section 2.3.6 and the applicable overstrength factor, Ω_0. In such cases, minimum spiral reinforcement shall be as specified in Section 1810.3.8.1.

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:

$A_{sh} = 0.3sh_c(f_c'/f_{yh})(A_g/A_{ch} - 1.0)[0.5 + 1.4P/(f_c'A_g)]$ (Equation 18-8)

but not less than:

$A_{sh} = 0.12sh_c(f_c'/f_{yh})\ [0.5 + 1.4P/(f_c'A_g)]$ (Equation 18-9)

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).

$0.06(f_c'/f_{yt})[2.8 + 2.3P_u/ f_c'A_g)]$ (18.13.5.10.5b)

and f_{yt} shall not be taken as greater than 100,000 psi

(d) Outside of the ductile pile region, spiral or hoop reinforcement shall be provided with a volumetric ratio not less than one-half of that required within the ductile pile region, and the maximum spacing shall be in accordance with Table 13.4.4.6(b).

(e) If transverse reinforcement consists of rectangular hoops and crossties, the total cross-sectional area of lateral transverse reinforcement in the ductile region shall be the greater of Eq. (18.13.5.10.5c) and Eq. (18.13.5.10.5d). The hoops and crossties shall be equivalent to deformed bars not less than No. 3 in size, and rectangular hoop ends shall terminate at a corner with seismic hooks.

$A_{sh} = 0.3sb_c\ (f_c'/f_{yt})(A_g/A_{ch} - 1.0)[0.5 + 1.4P_u/(f_c'A_g)]$ (18.13.5.10.5c)

$A_{sh} = 0.12sb_c\ (f_c'/f_{yh})\ [0.5 + 1.4P_u/(f_c'A_g)]$ (18.13.5.10.5d)

and f_{yt} shall not be taken as greater than 100,000 psi

18.13.5.10.6 For structures assigned to SDC C, D, E, or F, the maximum factored axial load for precast prestressed piles subjected to a combination of earthquake lateral force and axial load shall not exceed the following values:

(a) 0.2 $f_c'A_g$ for square piles

(b) 0.4$f_c'A_g$ for circular or octagonal piles

A_{sh} = Cross-sectional area of transverse reinforcement, square inches (mm2). f_c' = Specified compressive strength of concrete, psi (MPa). The hoops and cross ties shall be equivalent to deformed bars not less than No. 3 in size. Rectangular hoop ends shall terminate at a corner with seismic hooks. Outside of the length of the pile requiring transverse confinement reinforcing, the spiral or hoop reinforcing with a volumetric ratio not less than one-half of that required for transverse confinement reinforcing shall be provided. **1810.3.8.3.4** Axial load limit in Seismic Design Categories C through F. For structures assigned to Seismic Design Category C, D, E, or F, the maximum factored axial load on precast prestressed piles subjected to a combination of seismic lateral force and axial load shall not exceed the following values: 1. $0.2f_c'A_g$ for square piles 2. $0.4f_c'A_g$ for circular or octagonal piles	

Cost Impact: The code change proposal will not increase or decrease the cost of construction
There is no cost increase or decrease associated with this code change proposal with eliminates requirements addressed in ACI 318 from the IBC to avoid confusion and potenital conflicts.

S123-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee expressed concerns over the uncoordinated terminology utilized in the proposal specifically the inconsistences between ACI 318 and IBC. Some on the committee stated that they recognized that the concept of the proposal satisfied the long-term intent of moving technical requirements from the code to the appropriate standards; however, this proposal still needs modifications to satisfy the inconsistences. (Vote: 8-6)

Assembly Action: **None**

S123-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1810.2.4.1, 1810.3.9

Proponents:
Stephen Szoke, representing American Concrete Institute (steve.szoke@concrete.org)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

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 ~~1613.2.2~~ 18.13.5.10.5 in ACI 318 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 18.13.5.10.5 in ACI 318.
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 Section 18.13.5.5 of ACI 318.

1810.3.9 Cast-in-place deep foundations. Cast-in-place deep foundation elements shall be designed and detailed in accordance with Sections 1810.3.9.1 through ~~1810.3.9.4~~ 1810.3.9.2 and ACI 318 Section 18.13.

Commenter's Reason: This modification includes revisions and additions to the Code in an effort to eliminate conflicting provisions in ACI 318-19, ASCE 7-16 and IBC-2018 regarding design of deep foundations for earthquake resistant structures. Subcommittee F, Foundations, of ACI 318 has coordinated efforts with members from ASCE 7 to bring the concrete material design requirements for foundations to one location. ASCE 7 started this effort in their cycle ending in 2016. The changes to ACI 318 shown here is the continuation of that effort. A side-by-side comparison was provided with the original code change proposal.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
There is no cost increase or decrease associated with this code change proposal which eliminates requirements addressed in ACI 318 from the IBC to avoid confusion and potential conflicts.

Public Comment# 1280

Public Comment 2:

Proponents:
Thomas Schaeffer, Structural Design Group, representing Self (toms@sdg-structure.com)

requests As Submitted

Commenter's Reason: As I viewed the hearing from the ICC website, I was disappointed to see that S123-19 did not pass. And as someone who worked on the foundation provisions in ACI 318-19 for the last 5 years I would like share my knowledge of this proposal and hopefully clear up some the of the misconceptions I heard stated during the hearing. This process was a large undertaking by ACI 318 to assemble and organize all of the concrete deep foundation provisions from ASCE, IBC, and ACI in one Code and it is understandable that all of the aspects of the proposal S123-19 could not be adequately discussed in only a couple of minutes. Also, because S123-19 covers so much material and not just one or two provisions it was apparent that a lot of the subjects presented by the people in opposition were not accurate because they had only looked at bits and pieces and not studied S123- 19 as a whole. As an engineer who has been designing structures for almost 40 years, I think it will be a great improvement if all of the provisions related to the design of concrete foundations can be located in one Code. In this Public Discussion Comment I will do my best to address each of the negatives presented by the opposition at the hearing that led to the motion to disapprove, and hopefully you will reconsider that motion and approve S123- 19 as originally submitted.
I was chairman of the ACI 318 subcommittee on foundations that was responsible for writing the foundation change proposals that the full 318 committee voted on and adopted into ACI 318-19. The subcommittee was formed because of a study that had been performed outside of ACI showing the inconsistencies that occurred in the current provisions for deep foundations in seismic areas between ASCE-7, IBC, and ACI 318. The task of the subcommittee was to assemble the provisions for the concrete deep foundations all in one place, and that would be ACI 318. Many of the provisions in the IBC originated from the previous model codes and are based on ATC 3-06 and NEHRP, and they have essentially remained unchanged for some time. The goal of the ACI subcommittee was not to develop new provisions, but to organize the provisions from the three documents and assemble them in one Code document. If the concrete foundation provisions can reside in ACI 318 they will be evaluated each code cycle by the members of ACI 318 Building Code Committee and updated or revised as necessary. The 318 committee consists of professional engineers, professors, contractors, and building officials, all proficient in the knowledge of concrete design and construction. The foundation subcommittee for this code cycle included practicing engineers, some of whom that also serve on other ACI technical committees that deal specifically with foundations; 336 – Footings, Mats, and Drilled Piers and 543 – Concrete Piles. We held subcommittee meetings twice a year at the ACI Conventions and the meetings were open to the public. It is true as stated in the hearing that no pile foundation contractor associations were formally contacted, however, Dale Biggers, who is a member of the Pile Driver Contractor Association and stated in the hearing that ACI did not contact them regarding the ACI foundation work, actually attended 5 of the ACI 318-F subcommittee meetings. The ACI foundation subcommittee

also worked closely with a number of PCI members and researchers to update the recommended practice for the design of precast piling and include it in 318-19. In addition, ACI 318 has a Public Discussion period where all submitted public comments are considered and voted on by the committee. In fact, one of the opposition speakers made several public comments on the 318-19 provisions, all of which were considered and some were incorporated into 318-19. Therefore, the deep foundation provisions that are now included in ACI 318 have been fully vetted and successfully completed the consensus process.

The deep foundation provisions in 318-19 are essentially exactly the same as the related provisions that are currently in ASCE-7 and IBC. There are only very minor revisions where provisions needed to be updated. Opposition testimony stated that we are substituting a Code that works with one that is unproven, however, that can't be true because the related provisions in 318 are taken directly from ASCE 7 and the IBC.

There was discussion about the omission of SDC A and B in 318-19. In 1.4.7 of 318-19 it states the applicability of the Code, and that it includes Precast Piles in SDC A through F and Cast-in-place Concrete Piles in SDC C through F. The reason that SDC A and B for cast-in-place piles is not explicitly included is that it is not currently explicitly included in the IBC. In Section 1810.3.9 – Cast-in-place deep foundations, there is no provision that includes a statement for SDC A and B. It can be assumed that IBC Sections 1810.3.9.1 and 1810.3.9.2 include all of the Seismic Design Categories, and these two IBC provisions also appear in ACI 318-19 in Section 13.4.4, so a similar assumption can be made in ACI 318 that all SDC's are included. The remainder of the sections in IBC 1810.3.9 refer only to SDC C through F. The opposition testimony also stated that 1810.3.9.1 and 1810.3.9.2 are not included in ACI 318 and therefore must remain in the IBC, however this is incorrect, as stated above these provisions appear in 13.4.4 of ACI 318-19.

There was opposition testimony that ACI 318-19 mixes terminology for deep foundation members and the definitions are not consistent with IBC, which is not correct. The definitions in 318 are almost verbatim the definitions in IBC, except for the fact that "drilled shaft" in IBC is referred to as "drilled pier" in 318, and drilled pier is consistent with ASCE-7.

There was opposition testimony that stated that ACI 318-19 does not have provisions for reinforcement below the 1/3 pile length for a cast-in-place pile or pier in SDC. This is not correct, and the provisions for this pile or pier in ACI 318-19 [18.13.5.7] are exactly the same requirements as in IBC 1810.3.9.4.1. In addition, ACI 318 states that the longitudinal reinforcement shall extend at least the development length in tension beyond the flexural length of the pile, which is defined in ACI 318-19 [18.13.5.7].

Another statement made in opposition was that ACI 318-19 deviates from IBC with respect to provisions for piles/piers with casing. The provisions for cast-in-place concrete piles with spiral welded casing are the same in both IBC and 318. However, it should be noted that a pile with this type of casing is a mandrel driven pile that is unique and is typically known as a Raymond Pile, and since this unique type of pile is not used anymore, it should be considered to be removed from both Codes. With regards to concrete filled pipe piles or cast-in-place concrete piles with permanent casing, IBC Table 1810.3.2.6 contains allowable stress limitations for steel pipes and tubes in tension or compression. In this Table, the term "permanent casing" does not appear, but it can be assumed that permanent casing could be included as a steel pipe or tube. There was a statement made at the hearing that ACI 318-19 does not account for the presence of casing and that was a "hole" in the ACI provisions, but that is not correct. It is correct that for the maximum allowable compressive strength for deep foundation members ACI 318-19 Table 13.4.2.1 has a footnote that states that As in the equations does not include the steel in the casing, pipe, or tube; however, the commentary states that "Provisions for members designed to be composite with steel pipe or casing are covered in AISC 360". And this statement in the 318-19 Commentary is perfectly consistent with IBC 2205.1 that states "The design, fabrication and erection of structural steel elements in buildings, structures and portions thereof shall be in accordance with AISC 360". Therefore, ACI 318-19 and IBC are consistent with regards to provisions for the design of cast-in-place concrete piles in steel casing, pipe, or tube. However, this could be noted as a conflict in IBC in that 2205.1 states that steel design shall be in accordance with AISC 360 and that is not consistent with the allowable stresses given for structural steel elements in IBC Table 1810.3.2.6.

Therefore, based on the above discussion of the subjects presented by the opposition at the hearing I respectfully ask that S123-19 be reconsidered for adoption.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
S123-19, which is basically the incorporation of the deep foundation provisions from ASCE-7 and IBC into ACI 318-19, will not effectively increase or decrease the cost of construction.

Public Comment# 1134

S132-19

IBC®: 1810.3.5.3.1, 1810.3.11.2

Proposed Change as Submitted

Proponents: Jon-Paul Cardin, American Iron and Steel Institute, representing American Institute of Steel Construction (JCardin@steel.org)

2018 International Building Code

Revise as follows:

1810.3.5.3.1 Structural steel H-piles. Sections of structural steel H-piles shall comply with the requirements for HP shapes in ASTM A6, or 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 be not less than 80 percent of the depth of the section.
2. The nominal depth in the direction of the web shall be not less than 8 inches (203 mm).
3. Flanges and web shall have a minimum nominal thickness of $^3/_8$ inch (9.5 mm).

For structures assigned to Seismic Design Category D, E, or F, design and detailing of H-piles shall also conform to the requirements of AISC 341.

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 not less than 25 percent of the strength of the element in tension. Anchorage into the pile cap shall comply with the following:

1. In the case of uplift, the anchorage shall be capable of developing the least of the following:
1.1. The nominal tensile strength of the longitudinal reinforcement in a concrete element. .
1.2. The nominal tensile strength of a steel element. .
1.3. The frictional force developed between the element and the soil multiplied by 1.3.

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

2. In the case of rotational restraint, the anchorage shall be designed to resist the axial and shear forces, and moments resulting from the seismic load effects including overstrength factor in accordance with Section 2.3.6 or 2.4.5 of ASCE 7 or the anchorage shall be capable of developing the full axial, bending and shear nominal strength of the element.
3. The connection between the pile cap and the steel H-piles or unfilled steel pipe piles in structures assigned to Seismic Design Category D, E, or F shall be designed for a tensile force of not less than 10 percent of the pile compression capacity.

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

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

Reason: The purpose of this proposal is to correct an oversight and bring in modifications from ASCE 7-16, Section 14.1.8 to IBC Chapter 18. Interestingly, the language has been part of ASCE 7 since the 2005 edition, but not been brought forward to the IBC previously. Since ASCE 7 Chapter 14 is not typically adopted in the IBC for steel, it is necessary to add the language directly.
ASCE 7-16 Commentary states: “Steel piles used in higher SDCs are expected to yield just under the pile cap or foundation because of combined bending and axial load. Design and detailing requirements of AISC 341 for H-piles are intended to produce stable plastic hinge formation in the piles. Because piles can be subjected to tension caused by overturning moment, mechanical means to transfer such tension must be designed for the required tension force, but not less than 10% of the pile compression capacity.”

Bibliography: AISC (2016), *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341-16, American Institute of Steel Construction, Chicago, IL.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This proposal is not intended to make technical changes to the design or construction of H-piles. It is simply intended to clarify the currently accepted practice.

Staff Analysis: A review of the standard proposed for inclusion in the code, ASCE 4-16, 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 2, 2019.

S132-19

Public Hearing Results

Committee Action: **As Modified**

Committee Modification:

2018 International Building Code

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 not less than 25 percent of the strength of the element in tension. Anchorage into the pile cap shall comply with the following:

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

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

   1.2 The nominal tensile strength of a steel element.

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

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

2. In the case of rotational restraint, the anchorage shall be designed to resist the axial and shear forces, and moments resulting from the seismic load effects including overstrength factor in accordance with Section 2.3.6 or 2.4.5 of ASCE 7 or the anchorage shall be capable of developing the full axial, bending and shear nominal strength of the element.

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

~~Exception~~ Exceptions:

1. Connection tensile capacity need not exceed the strength required to resist seismic load effects including overstrength of ASCE 7 Section 12.4.3 or 12.14.3.2.

2. Connections need not be provided where the foundation or supported structure does not rely on the tensile capacity of the piles for stability under the design seismic force.

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

Committee Reason: Corrects a current code oversight by specifically allowing H-piles in the IBC for high seismic. The modification clarified the exceptions. (Vote: 13-0-1 abstaining)

Assembly Action: **None**

S132-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1810.3.11.2

Proponents:
Daniel Stevenson, representing GeoCoalition (dstevenson@berkelapg.com)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

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 not less than 25 percent of the strength of the element in tension. <u>For piles required to resist uplift forces or provide rotational restraint,</u> ~~A~~ <u>a</u>nchorage into the pile cap shall comply with the following:

1. In the case of uplift, the anchorage shall be capable of developing the least of the following:
   1.1. The nominal tensile strength of the longitudinal reinforcement in a concrete element. .

   1.2. The nominal tensile strength of a steel element. .

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

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

2. In the case of rotational restraint, the anchorage shall be designed to resist the axial and shear forces, and moments resulting from the seismic load effects including overstrength factor in accordance with Section 2.3.6 or 2.4.5 of ASCE 7 or the anchorage shall be capable of developing the full axial, bending and shear nominal strength of the element.
3. The connection between the pile cap and the steel H-piles or unfilled steel pipe piles in structures assigned to Seismic Design Category D, E, or F shall be designed for a tensile force of not less than 10 percent of the pile compression capacity.

   Exceptions:

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

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

Commenter's Reason: The existing code language is confusing and can appear contradictory. The requirements of "...not less than 25 percent of the strength of the element in tension.", and then later "The nominal tensile strength..." can appear contradictory of one does not realize that the more restrictive requirement is only required for piles required to resist uplift forces. The added phrase clarifies that the intent of the code is that the more restrictive requirements apply only to piles required to resist uplift forces or provide rotational restraint.

The added phrase is taken verbatim from ASCE-7 section 12.13.6.5, upon which this code section is based. ASCE-7 12.13.6 contains the sentence "For piles required to resist uplift forces or provide rotational restraint<u>,</u> anchorage into the pile cap shall comply with the following:" Following this sentence, ASCE-7 12.13.6.5 contains the same requirements as IBC 1810.3.11.2. Adding the phrase from ASCE-7 into the code will provide consistency between the code and referenced standard ASCE-7.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
This only clarifies existing code requirements.

S133-19

IBC: 1810.3.6

Proposed Change as Submitted

Proponents: Dale Biggers, P.E. GeoCoalition, representing GeoCoalition (dbiggers@bohbros.com); Daniel Stevenson, P.E., representing GeoCoalition (dstevenson@berkelapg.com); Lori Simpson, P.E., G.E., representing GeoCoalition (lsimpson@langan.com)

2018 International Building Code

Revise as follows:

1810.3.6 Splices. Splices shall be constructed so as to provide and maintain true alignment and position of the component parts of the deep foundation element during installation and subsequent thereto and shall be designed to resist the axial and shear forces and moments occurring at the location of the splice during driving and for design load combinations. Where deep foundation elements of the same type are being spliced, splices shall develop not less than 50 percent of the bending strength of the weaker section. Where deep foundation elements of different materials or different types are being spliced, splices shall develop the full compressive strength and not less than 50 percent of the tension and bending strength of the weaker section. Where structural steel cores are to be spliced, the ends shall be milled or ground to provide full contact and shall be full-depth welded.

Exception: Splices conforming to generally accepted engineering practices where approved by the *building official.*

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: 1. Section 1810.3.6 already requires that splices "…shall be designed to resist the axial and shear forces and moments occurring at the location of the splice…". Conformance with this requirement already ensures the structural integrity of the splice. Section 1810.3.6.1 contains more restrictive splice requirements for structures assigned to seismic design categories C through F.
2. The current specification precludes commonly available splices that would be acceptable in many design situations, such as a splice located at significant depth. (i.e., where significant tension or bending demands are not expected or possible. Load requirements at the splice diminish due to soil resistance above the splice as the splice is located deeper.).

3. The depth of the splice is known when you are driving to a predefined depth. For example where 240-ft long friction piles are driven to a predefined depth, the splice between two 120-ft sections will be 120-ft below grade.

4. The current code causes unnecessary costs.

Example a.) To make a welded splice on a 20-inch diameter pipe pile costs $ 1,015 in labor and equipment. To buy a drive-fit pipe-to-pipe splicer costs $ 495. For 211 piles at $520 extra, the added cost was $ 109,720.

Example b.) A tension splice for a 14-inch square prestressed concrete pile costs $ 553 to purchase. A drive-fit splice for that pile costs $ 201. For 2,420 piles at $ 352 extra, the added cost was $ 851,000.

These are real costs on real jobs, not hypothetical examples.

5. Drive-fit splices were used successfully on the New Orleans Superdome, 52-story Shell Square, 50-story Sheraton Hotel, and many other New Orleans structures. These buildings are more than 40 years old.

6. "Supporting data" may include a geotechnical investigation and/or a load test; this requirement is similar to Section 1810.3.2.8.

Click here to see the members of the GeoCoalition : http://www.piledrivers.org/2019-geocoalition-members/

Cost Impact: The code change proposal will decrease the cost of construction
The proposed change will decrease the cost of construction but only in some areas of the country.

S133-19

Public Hearing Results

Committee Action: **As Modified**

Committee Modification:
2018 International Building Code

1810.3.6 Splices. Splices shall be constructed so as to provide and maintain true alignment and position of the component parts of the deep foundation element during installation and subsequent thereto and shall be designed to resist the axial and shear forces and moments occurring at the location of the splice during driving and for design load combinations. Where deep foundation elements of the same type are being spliced, splices shall develop not less than 50 percent of the bending strength of the weaker section. Where deep foundation elements of different materials or different types are being spliced, splices shall develop the full compressive strength and not less than 50 percent of the tension and bending strength of the weaker section. Where structural steel cores are to be spliced, the ends shall be milled or ground to provide full contact and shall be full-depth welded.

Exception: Splices conforming to generally accepted engineering practices ~~and~~ where approved by the *building official* <u>for buildings assigned to Seismic Design Category A or B.</u>

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.

Committee Reason: This proposal recognizes the condition with lower stresses to engineer the splice. The modifications clarified that the exception is only for SDC A or B and deleted the word 'and' in the exception. (Vote: 14-0)

Assembly Action: **None**

S133-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1810.3.6 (New)

Proponents:
Dale Biggers, representing GeoCoalition (dbiggers@bohbros.com)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1810.3.6 Splices. Splices shall be constructed so as to provide and maintain true alignment and position of the component parts of the deep foundation element during installation and subsequent thereto and shall be designed to resist the axial and shear forces and moments occurring at the location of the splice during driving and for design load combinations. Where deep foundation elements of the same type are being spliced, splices shall develop not less than 50 percent of the bending strength of the weaker section. Where deep foundation elements of different materials or different types are being spliced, splices shall develop the full compressive strength and not less than 50 percent of the tension and bending strength of the weaker section. Where structural steel cores are to be spliced, the ends shall be milled or ground to provide full contact and shall be full-depth welded.

~~**Exception:** Splices conforming to generally accepted engineering practices where approved by the *building official* for buildings assigned to Seismic Design Category A or B.~~

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

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

Commenter's Reason: After the Committee Action Hearing, we received a comment that the Exception, as originally written and approved by the Committee, was overly broad. As previously written, it could be interpreted to mean that splices would not have to be designed for the forces at the splice location during driving or at the final splice location nor would the steel core requirements apply if approved by the building official. The intent was only to exempt splices in low seismic design categories from having to be designed to the 50 percent of the tension and bending strength of the pile material. All other requirements of this section should still apply.

The Committee approved the intent of the original proposal and this current change reflects the original intent.

Cost Impact: The net effect of the public comment and code change proposal will decrease the cost of construction
The proposed change will decrease the cost of construction in some areas. Additionally, the change is intended to eliminate any misinterpretations of the intent of the code.

Public Comment# 1268

S138-19

IBC®: 1810.4.1.3

Proposed Change as Submitted

Proponents: Daniel Stevenson, P.E., representing GeoCoalition (dstevenson@berkelapg.com); Dale Biggers, P.E. GeoCoalition, representing GeoCoalition (dbiggers@bohbros.com); Lori Simpson, P.E., G.E., representing GeoCoalition (lsimpson@langan.com)

2018 International Building Code

Revise as follows:

1810.4.1.3 Driving near uncased concrete. Deep foundation elements shall not be driven within six element diameters center to center in granular soils or within one-half the element length in cohesive soils of an uncased element filled with concrete less than 48 hours old unless *approved* by the *building official*. ~~If~~ During driving near uncased concrete, if the concrete surface in any completed element rises or drops significantly or bleeds additional water, the previously completed element shall be replaced. Driven uncased deep foundation elements shall not be installed in soils that could cause heave.

Reason: 1. Minor rises or drops are normal due to consolidation of the concrete, etc. Only significant changes in elevation are of concern.
2. There are other possible areas of concern in addition to a change of elevation of the top surface of a previously completed element. It is common to get some minimal bleed water due to concrete consolidation, but if there is excessive bleed water due to installation of another nearby pile then there is likely a problem.

3. In locations of high water table, installing piles can force ground water into previously installed piles.

4. The change clarifies the current guidelines and calls attention to conditions that should already be under consideration.

5. The proposal also clarifies that the previously completed element is the one to be replaced.

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Cost Impact: The code change proposal will not increase or decrease the cost of construction
These requirements are current industry standard quality control practice.

It will decrease costs in some cases since it may allow higher design loads where the geotechnical capacity is sufficiently higher than the structural strength.

S138-19

Public Hearing Results

Committee Action: **As Submitted**

Committee Reason: The committee agreed that the proposal clarifies the guidelines concerning 'what needs to be replaced'. The committee expressed concerns that re-wording maybe required during the public comment phase to clarify 'which previously completed elements' and 'who makes the call'. (Vote: 13-1)

Assembly Action: **None**

S138-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1810.4.1.3

Proponents:
Daniel Stevenson, representing GeoCoalition (dstevenson@berkelapg.com)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1810.4.1.3 Driving near uncased concrete. Deep foundation elements shall not be driven within six element diameters center to center in granular soils or within one-half the element length in cohesive soils of an uncased element filled with concrete less than 48 hours old unless *approved* by the *building official*. ~~During~~ If driving near uncased concrete elements~~,~~ causes ~~if~~ the concrete surface in any completed element to rise~~s~~ or drop~~s~~ significantly or bleed~~s~~ additional water, the ~~previously~~ completed element shall be replaced. Driven uncased deep foundation elements shall not be installed in soils that could cause heave.

Commenter's Reason: The proposed modifications do not change the requirements of the code section. They clarify this code section by using more concise language.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
This will clarify the code, but not change the code requirements.

Public Comment# 1928

S140-19

IBC®: 1810.4.5

Proposed Change as Submitted

Proponents: Dale Biggers, P.E. GeoCoalition, representing GeoCoalition (dbiggers@bohbros.com); Daniel Stevenson, P.E., representing GeoCoalition (dstevenson@berkelapg.com); Lori Simpson, representing GeoCoalition (lsimpson@langan.com)

2018 International Building Code

Revise as follows:

1810.4.5 Vibratory driving. Vibratory drivers shall only be used to install deep foundation elements where the element load capacity is verified by load tests in accordance with Section 1810.3.3.1.2. 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.

Exceptions:

1. The pile installation is completed by driving with an impact hammer in accordance with Section 1810.3.3.1.1.
2. The pile is to be used only for lateral resistance.

Reason: 1. Axial load tests are only needed when there are axial loads and the capacity is in doubt.
2. This proposal adds the exception for "the pile installation is completed by driving with an impact hammer…" because piles that are started using a vibratory hammer but completed using an impact hammer should be treated as piles that are installed by an impact hammer.

3. An impact hammer can be used to assure that you have achieved or exceeded the minimum required axial capacity. Section 1810.3.3.1.1. details how capacity might be determined from impact driving.

4. The exception for " the pile is to be used only for lateral resistance" is needed because a load test for axial capacity (as implied by 1810.3.3.1.2) is not needed for piles used only for lateral resistance. Lateral load capacity requirements are covered in Section 1810.3.3.2.

Click here to see the members of the GeoCoalition : http://www.piledrivers.org/2019-geocoalition-members/

Cost Impact: The code change proposal will decrease the cost of construction
Will not increase the cost of construction. In fact, it will likely decrease cost as an axial load test will not be required where piles are used only for lateral resistance or where the pile installation is completed using an impact hammer.

S140-19

Public Hearing Results

Committee Action: **As Submitted**

Committee Reason: The committee felt the new exceptions are appropriate and added clarification.
(Vote: 14-0)

Assembly Action: **None**

S140-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1810.4.5

Proponents:
Dale Biggers, representing GeoCoalition (dbiggers@bohbros.com)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1810.4.5 Vibratory driving. Vibratory drivers shall only be used to install deep foundation elements where the element load capacity is verified by load tests in accordance with Section 1810.3.3.1.2. ~~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.~~

Exceptions:

1. Load testing is not required where ~~T~~ the pile installation is completed by driving with an impact hammer in accordance with Section 1810.3.3.1.1.
2. Load testing is not required where ~~T~~ the vertical pile is to be used only for lateral resistance

   The installation of production elements shall be controlled according to power consumption, rate of penetration or other *approved* means that ensure element capacities equal or exceed those of the test elements.

Commenter's Reason: This change is made to improve the Exceptions by making full sentences. This also narrows the scope of the Exceptions. There is no change to the original intent of the Exceptions.

The last sentence of the code has been moved unchanged to a location following the Exception to clarify that the installation still must be controlled.

Cost Impact: The net effect of the public comment and code change proposal will decrease the cost of construction
This can save costs in some instances where a load test will not be required. An axial load test will not be required where piles are used only for lateral resistance or where the pile installation is completed using an impact hammer.

Public Comment# 1258

Public Comment 2:

IBC®: 1810.4.5

Proponents:
Daniel Stevenson, representing GeoCoalition (dstevenson@berkelapg.com)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1810.4.5 Vibratory driving. Vibratory drivers shall only be used to install deep foundation elements where the element load capacity is verified by load tests in accordance with Section 1810.3.3.1.2. ~~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.~~

Exceptions:

1. Load testing is not required when ~~T~~ the ~~pile~~ element installation is completed by driving with an impact hammer in accordance with Section 1810.3.3.1.1.
2. Load testing is not required when a vertical element ~~The pile~~ is to be used only for lateral resistance.

The installation of production elements shall be controlled according to power consumption, rate of penetration or other *approved* means that ensure element capacities equal or exceed those of the test elements.

Commenter's Reason: The second sentence has been moved to after the exceptions to clarify that the exceptions do not apply to the requirements contained in the second sentence.

The phrase "Load testing is not required when..." has been added to both exceptions to clarify that the exceptions only apply to the load test requirement, and to make both exceptions complete sentence.

The word "pile" has been replaced with the word "element" to be consistent with the terminology used throughout the code, including this code section.

The word "vertical" has been added to the second exception to clarify that the exception will not apply to a battered element that resists lateral load from the horizontal component of its axial load.

Cost Impact: The net effect of the public comment and code change proposal will decrease the cost of construction
The cost savings are less by not requiring unnecessary tests.

Public Comment# 1641

S144-19

IBC: 1901.2, SECTION 1907, 1907.1. 1907.2, 1907.3

Proposed Change as Submitted

Proponents: Stephen Szoke, American Concrete Institute, representing American Concrete Institute (steve.szoke@concrete.org); Scott Campbell, representing National Ready Mixed Concrete Association (scampbell@nrmca.org)

2018 International Building Code

Revise as follows:

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.

SECTION 1907
~~MINIMUM SLAB PROVISIONS~~ SLABS-ON-GROUND

1907.1 General. Slabs-on-ground not transmitting vertical loads or lateral forces from other parts of the structure to the soil shall be designed and constructed in accordance with section 1904 and this section. ~~The thickness of concrete floor slabs supported directly on the ground shall be not less than $3^1/_2$ inches (89 mm). A 6-mil (0.006 inch; 0.15 mm) polyethylene vapor retarder with joints lapped not less than 6 inches (152 mm) shall be placed between the base course or subgrade and the concrete floor slab, or other *approved* equivalent methods or materials shall be used to retard vapor transmission through the floor slab.~~

~~**Exceptions:** A vapor retarder is not required:~~

1. ~~For detached structures accessory to occupancies in Group R-3, such as garages, utility buildings or other unheated facilities.~~
2. ~~For unheated storage rooms having an area of less than 70 square feet (6.5 m^2) and carports attached to occupancies in Group R-3.~~
3. ~~For buildings of other occupancies where migration of moisture through the slab from below will not be detrimental to the intended occupancy of the building.~~
4. ~~For driveways, walks, patios and other flatwork that will not be enclosed at a later date.~~
5. ~~Where *approved* based on local site conditions.~~

Add new text as follows:

1907.1.1 Slabs-on-ground transmitting loads. Where slabs-on-ground transmit vertical loads or lateral forces from other parts of the structure to the soil all provisions in this Chapter shall be applicable.

1907.2 Thickness. The thickness of concrete floor slabs supported directly on the ground shall be not less than $3^1/_2$ inches (89 mm).

1907.3 Vapor retarder. A polyethylene vapor retarder having a minimum 6-mil (0.006 inch; 0.15 mm) polyethylene vapor retarder thickness and with joints lapped not less than 6 inches (152 mm) shall be placed between the base course or subgrade and the concrete floor slab, or other *approved* equivalent methods or materials shall be used to retard vapor transmission through the floor slab.

Exceptions: A vapor retarder is not required:

1. For detached structures accessory to occupancies in Group R-3, such as garages, utility buildings or other unheated facilities.
2. For unheated storage rooms having an area of less than 70 square feet (6.5 m^2) and carports attached to occupancies in Group R-3.
3. For buildings of other occupancies where migration of moisture through the slab from below will not be detrimental to the intended occupancy of the building.
4. For driveways, walks, patios and other flatwork that will not be enclosed at a later date.
5. Where *approved* based on local site conditions.

Reason: Reason: The current language is not clear. First the provisions are only applicable to slabs on ground and this should be more clearly stated. Further it is generally understood that all provisions of the IBC are minimum requirements. This code change places all provisions uniquely applicable to slabs-on-ground in one section rather than having provisions in sections 1901.2 and 1907.
Modifications shown as new section 1907.1.1. This portion of the proposed revision is editorial, deleting slab-on-ground provisions from Section 1901.2 (shown above as deleted text) and moving the provisions to the more appropriate section, 1907. This places provisions for concrete slabs-on-ground in one section.

Modifications shown as new section 1907.1.2. This portion of the proposed revision is editorial and clarifies that thickness criteria are for concrete slabs-on-ground.

Modifications shown as new section 1907.1.3. This portion of the proposed revision is editorial and appropriately assigns provisions for vapor retarders to vapor retarders and not to slabs-on-ground.

ACI, a 501.C.3 professional society, encourages the approval of this code change proposal to improve the IBC by more clearly advising the user that these provisions are only applicable to slabs-on-ground and relocates slab-on-ground provisions in one section.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
No change to cost of design or consturciton, change places slab related crtieria in one section.

S144-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee's majority opinion was that the proposed worded was less clear than the existing code wording (especially for section 1907.1.1).
(Vote: 10-4)

Assembly Action: **None**

S144-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1901.2, SECTION 1907, 1907.1, 1907.1.1 (New), 1907.2 (New), 1907.3 (New)

Proponents:
Stephen Szoke, representing American Concrete Institute (steve.szoke@concrete.org)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

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. 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.

SECTION 1907
SLABS-ON-GROUND

1907.1 General. Slabs-on-ground ~~not transmitting vertical loads or lateral forces from other parts of the structure to the soil shall be~~ designed and constructed in accordance with section 1904 and this section.

1907.1.1 Slabs-on-ground transmitting loads. Where slabs-on-ground transmit vertical loads or lateral forces from other parts of the structure to the soil <u>design and construction of slabs-on-ground shall comply with all applicable provisions of this chapter</u> ~~all provisions in this Chapter shall be applicable~~.

1907.2 Thickness. The thickness of concrete floor slabs supported directly on the ground shall be not less than $3^1/_2$ inches (89 mm).

1907.3 Vapor retarder. A polyethylene vapor retarder having a minimum 6-mil (0.006 inch; 0.15 mm) polyethylene vapor retarder thickness and with joints lapped not less than 6 inches (152 mm) shall be placed between the base course or subgrade and the concrete floor slab, or other *approved* equivalent methods or materials shall be used to retard vapor transmission through the floor slab.

Exceptions: A vapor retarder is not required:

1. For detached structures accessory to occupancies in Group R-3, such as garages, utility buildings or other unheated facilities.

2. For unheated storage rooms having an area of less than 70 square feet (6.5 m^2) and carports attached to occupancies in Group R-3.
3. For buildings of other occupancies where migration of moisture through the slab from below will not be detrimental to the intended occupancy of the building.
4. For driveways, walks, patios and other flatwork that will not be enclosed at a later date.
5. Where *approved* based on local site conditions.

Commenter's Reason: This public comment clarifies that provisions of the code for slabs-on-ground are not altered. This code change proposal corrects the IBC by clearly communicating that the provisions are only minimum requirements for slabs-on-ground and not applicable to all slabs, including interim floor slabs.
This change is not intended to alter any the requirements of the code but places vapor retard provisions in a section titled vapor retarders, thickness in a section title thickness, and design and construction requirements in the appropriate section.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
No change to the provisions, language changed and reorganized for clarity.

Public Comment# 1333

S146-19

IBC®: 1901.5

Proposed Change as Submitted

Proponents: Stephen Szoke, American Concrete Institute, representing American Concrete Institute (steve.szoke@concrete.org); Scott Campbell, National Ready Mixed Concrete Association, representing National Ready Mixed Concrete Association (scampbell@nrmca.org); Amy Trygestad, representing Concrete Reinforcing Steel Institute (atrygestad@crsi.org)

2018 International Building Code

Delete without substitution:

~~**1901.5 Construction documents.** The *construction documents* for structural concrete construction shall include:~~

~~1.The specified compressive strength of concrete at the stated ages or stages of construction for which each concrete element is designed.~~
~~2.The specified strength or grade of reinforcement.~~
~~3.The size and location of structural elements, reinforcement and anchors.~~
~~4.Provision for dimensional changes resulting from creep, shrinkage and temperature.~~
~~5.The magnitude and location of prestressing forces.~~
~~6.Anchorage length of reinforcement and location and length of lap splices.~~
~~7.Type and location of mechanical and welded splices of reinforcement.~~
~~8.Details and location of contraction or isolation joints specified for plain concrete.~~
~~9.Minimum concrete compressive strength at time of posttensioning.~~
~~10.Stressing sequence for posttensioning tendons.~~
~~11.For structures assigned to *Seismic Design Category* D, E or F, a statement if slab on grade is designed as a structural diaphragm.~~

Reason: This code change proposal removes an incomplete list of criteria necessary for the construction documents applicable to structural concrete. The list in the IBC is not as comprehensive as the list in referenced ACI documents. Many of the omissions from the IBC list are shown in the table below. Since the IBC supersedes referenced ACI documents the partial list in the IBC is all that would be required although ACI documents have significantly more extensive requirements. If the list in the IBC is to indicate what may be of particular importance to the building code official, then that list might be best included in the commentary to the IBC, but not provided as the applicable requirements for construction documents. Further maintaining duplicate lists becomes problematic and results code change proposals that would not alter the requirements. The list in the IBC is outdate and many important items recently added to ACI documents are not addressed, in particular note the requirements for anchors and qualifications for personnel.

IBC Requirements	ACI Requirements
	Loads used in design
	Design work delegated to contractor
	Cementitious materials and combinations
	Water cement ratio
	Aggregates
	Mixing water
	Admixtures
	If water reducing – amount of modification
	If retarding – modification in setting time
	Where expansive cements are used – admixture compatibility
	Steel fiber reinforcement
Compressive strength of concrete	Compressive strength of concrete
	Test age for compressive strengths
	Maximum water cement ratio
	Maximum size of aggregate
	Exposure Category F – air content
	Exposure Class C – chloride ion limits
	Exposure Class S – types of cement
	Density of lightweight aggregate

	Volumetric fracture of aggregates where required in design
	Where used for shear – requirements for steel-fiber reinforced concrete
	Exposure class at option of engineer
	Compressive strength as various phases at option of engineer.
	Concrete mix proportions
	Material storage
	Concrete batching, mixing, and transport/delivery
	Pump pipe requirements
	Concrete placement
	Vertical lift requirements
	Field cured specimens if required
	Temperature of high early strength concrete
	Accelerated curing requirements if employed
	Protection and curing concrete
	Cold weather concrete procedures if applicable
	Hot weather concrete procedures if applicable
	Locations where slab column interfaces are integrated
	Locations where steel-fiber reinforcement is required
	Saw cutting locations
Strength or grade of reinforcement	Designation and grade of reinforcement
Size and location of elements	Size and location of members
	Tolerance of members
Size and location of reinforcement	Size and location of reinforcement
	Tolerances for reinforcement
	Designation of protective coatings
	Mill reports
	Field bending of reinforcement
Provisions for dimensional change	Provisions for dimensional change
	Qualifications of anchors
	Type, size, location requirements, effective embedment depth, and installation requirements for anchors
	For adhesive anchors, minimum age of concrete, concrete temperature range, moisture condition of concrete at time of installation, type of lightweight concrete if applicable, and requirements for hole drilling and preparation
	Qualifications for anchor installers
	Corrosion protect for exposed anchors
	Type, size, details, and location of embedments
	Details of lifting devices, embedments, and related reinforcement required to resist temporary loads from handling, storage, transportation, and erection, where designed by the licensed design professional.
Magnitude and location of prestressing forces	Magnitude and location of prestressing forces
Anchor and lap splice lengths	Anchor and lap splice lengths
Type and location of welded and mechanical splices	Type and location of welded and mechanical splices
	Type and location of end-bearing splices

Details and location of construction joints	Details and location of construction joints
	Design of construction joints
	Surface preparation
	Shear transfer where required
Minimum compressive strength for post-tensioning	Minimum compressive strength for post-tensioning
Post-tensioning stressing sequence	Post-tensioning stressing sequence
	Tolerances for tendons
	Materials and details of corrosion protection for tendons, couplers, end fittings, post-tensioning anchorages, and anchorage regions.
	Requirements for grouting of bonded tendons, including maximum water-soluble chloride ion
	Formwork, including removal
	Qualifications of filed technicians
	Qualifications of inspectors
	Qualifications of testing agency and technicians
Slab on grade resisting seismic forces.	Identify if a slab-on-ground is designed as a structural diaphragm or part of the seismic-force-resisting system

ACI, a 501.C.3. professional technical society, recommends approval of this code change as submitted to assure that all relevant requirements for structural concrete as included on construction documents and to reduce confusion and eliminate the need to maintain duplicate lists.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This code change proposal removes potential conflicts between the IBC and ACI requirements for construction documents.

S146-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee expressed concerns that the proposal would make it harder to find requirements. The committee encourages ACI to update to the IBC list.
(Vote: 12-2)

Assembly Action: **None**

S146-19

Individual Consideration Agenda

Public Comment 1:

Proponents:
Stephen Szoke, representing American Concrete Institute (steve.szoke@concrete.org)

requests As Submitted

Commenter's Reason: The truncated list that appears in the IBC is substantially different from the requirements in ACI 318 creating a conflict within the IBC. Section 1901.1 requires compliance with ACI 318, yet the provisions of section 1901.5 of the IBC provides a less complete list of information required on the construction documents. Maintaining different lists in the IBC than the referenced documents is not appropriate and is

misleading when the intent is that all the requirements listed in ACI 318 are applicable.
As stated in the reason statement for the original proposal reason statement and in testimony, if a truncated list is important to highlight items of importance to the building official then this list should be in commentary and not the code.

One committee person suggested that ACI should duplicate the list in the IBC. There is no reason to duplicate the entire list in the IBC since the list is complete in ACI 318 which is a referenced standard. Further, it is not efficient use of anyone's time to maintain and coordinate duplicate lists.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
This public comment and the original proposal to not increase of decrease costs. The provisions of ACI 318 remain applicable.

Public Comment# 1336

S153-19

IBC®: 1907.1, ASTM Chapter 35 (New)

Proposed Change as Submitted

Proponents: Terry Kozlowski, representing Southern Nevada Chapter; Nenad Mirkovic, representing City of Las Vegas; Amanda Moss, representing SN-ICC Member; Cassidy Wilson, representing SN-ICC Member

2018 International Building Code

Revise as follows:

1907.1 General. The thickness of concrete floor slabs supported directly on the ground shall be not less than $3^1/_2$ inches (89 mm). A ~~6-mil (0.006~~ 10-mil (0.010 inch; ~~0.15~~ 0.254 mm) polyethylene vapor retarder conforming to ASTM E 1745 Class A requirements with joints lapped not less than 6 inches (152 mm) shall be placed between the base course or subgrade and the concrete floor slab, or other *approved* equivalent methods or materials shall be used to retard vapor transmission through the floor slab.

Exception: A vapor retarder is not required:

1. For detached structures accessory to occupancies in Group R-3, such as garages, utility buildings or other unheated facilities.
2. For unheated storage rooms having an area of less than 70 square feet (6.5 m^2) and carports attached to occupancies in Group R-3.
3. For buildings of other occupancies where migration of moisture through the slab from below will not be detrimental to the intended occupancy of the building.
4. For driveways, walks, patios and other flatwork that will not be enclosed and heated at a later date.
5. Where *approved* based on local site conditions.

Add new text as follows:

ASTM

ASTM International
100 Barr Harbor Drive, P.O. Box C700
West Conshohocken PA 19428-2959

E1745-17: Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs

Reason: By coordinating the requirements for the vapor retarder with the American Concrete Institute (ACI) recommendations, this proposal will promote consistency across codes and standards for various moisture conditions.

Bibliography: ACI 302.2R Section 9.3:
"…ACI 302.1R recommends a minimum 10 mil (0.25 mm) vapor retarder thickness when the retarder is protected with a granular fill. When the vapor retarder is not protected by a fill, some specifiers require a 15 mil (0.38 mm) thickness or greater…"

Cost Impact: The code change proposal will increase the cost of construction
This proposal will increase the cost of construction by an estimated $0.045/sq foot, based on cost analysis in current market conditions. For example, a 50,000 square foot commercial building will have an estimated increase of $2,250.

Staff Analysis: A review of the standard proposed for inclusion in the code, ASTM E1745-17, 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 2, 2019.

S153-19

Public Hearing Results

Committee Action: Disapproved

Committee Reason: The committee did not feel there was sufficient justification to increase the thickness from 6mil to 10mil.
(Vote: 14-0)

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

IBC®: 1907.1

Proponents:
Stephen Szoke, representing American Concrete Institute (steve.szoke@concrete.org)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

1907.1 General. The thickness of concrete floor slabs supported directly on the ground shall be not less than $3^1/_2$ inches (89 mm). A 6-mil ~~10~~ mil (0.006 ~~0.010~~ inch; 0.152 mm ~~mm~~) ~~polyethylene~~ vapor retarder conforming to at least the Class C requirements of ASTM E 1745 ~~Class A requirements~~ with joints lapped not less than 6 inches (152 mm) shall be placed between the base course or subgrade and the concrete floor slab, or other *approved* equivalent methods or materials shall be used to retard vapor transmission through the floor slab.

Exception: A vapor retarder is not required:

1. For detached structures accessory to occupancies in Group R-3, such as garages, utility buildings or other unheated facilities.
2. For unheated storage rooms having an area of less than 70 square feet (6.5 m^2) and carports attached to occupancies in Group R-3.
3. For buildings of other occupancies where migration of moisture through the slab from below will not be detrimental to the intended occupancy of the building.
4. For driveways, walks, patios and other flatwork that will not be enclosed and heated at a later date.
5. Where *approved* based on local site conditions.

Commenter's Reason: This code change proposal reduces the minimum thickness form 10 mil to 6 mil and reduces the specified class of materials conforming to ASTM E1745 from Class A to Class C.
These changes are intended to better align the provisions in the IRC with the recommendations of ACI Committee 302 on Construction of Concrete Floors as published in ACI 302.2R Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials which reads: "In the past, 4, 6, 8, and 10 mil (0.10, 0.15, 0.20, and 0.25 mm) low-density polyethylene sheets have been used as belowslab vapor retarder material. Any material used as a belowslab vapor retarder/barrier, however, should conform to the requirements of ASTM E 1745, 'Standard Specification for Water Vapor Retarders Used in Contact with Soil or Granular Fill Under Concrete Slabs.”

Since ACI 302.2R does not specify class, this public comment reduces the class to the minimum requirements of ASTM E1745 which is Class C.

Since ACI 302.2R does not specify thickness, as long as the material satisfies ASTM E1745 it would be preferable to not specify minimum thickness in the IBC. However, during the Committee Action Hearings arguments were made that 6 mil polyethylene sheet is not sufficiently durable for applications as belowslab vapor retarders. ASTM E1745 does not specify materials and thus arguments made that 6 mil polyethylene sheet might not be sufficiently durable may not be applicable to 6 mil membranes made of other materials. Since 6 mil was permitted in the 2018 IBC, this public comment reverts back to that as the minimum thickness. Regardless of thickness, the material must conform to ASTM E1745.

Bibliography: 302.2R-06 Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
By limiting the criteria of this provision to any material conforming to ASTM E1745 and allowing minimum thickness of 6 mil, this provision should not significantly increase the cost of construction as compared to the 2018 edition of the IBC, but could reduce costs compared to the the new provision presented as S153-19.

Public Comment# 1357

S154-19

IBC: 1907.2 (New)

Proposed Change as Submitted

Proponents: Amy Dowell, Post-Tensioning Institute, representing Post-Tensioning Institute (amy.dowell@post-tensioning.org); Stephen Szoke, representing American Concrete Institute (steve.szoke@concrete.org)

2018 International Building Code

1808.6.2 Slab-on-ground foundations. Moments, shears and deflections for use in designing slab-on-ground, mat or raft foundations on expansive soils shall be determined in accordance with *WRI/CRSI Design of Slab-on-Ground Foundations or PTI DC 10.5*. Using the moments, shears and deflections determined above, nonprestressed slabs-on-ground, mat or raft foundations on expansive soils shall be designed in accordance with *WRI/CRSI Design of Slab-on-Ground Foundations* and post-tensioned slab-on-ground, mat or raft foundations on expansive soils shall be designed in accordance with *PTI DC 10.5*. It shall be permitted to analyze and design such slabs by other methods that account for soil-structure interaction, the deformed shape of the soil support, the plate or stiffened plate action of the slab as well as both center lift and edge lift conditions. Such alternative methods shall be rational and the basis for all aspects and parameters of the method shall be available for peer review.

Add new text as follows:

1907.2 Post-tensioned concrete slabs-on-ground. Post-tensioned concrete slabs placed on expansive or stable soils shall be designed in accordance with PTI DC-10.5.

Reason: There are currently no provisions for designing post-tensioned slabs on stable soils in IBC. The updated PTI standard, PTI DC10.5-19 has been updated to include stable soils. This title of the reference document has been changed to: PTI DC10.5-19 Standard Requirements for Design and Analysis of Shallow Concrete Foundations on Expansive and Stable Soils.
Post-tensioned slabs are commonly used on stable soils for crack control as well as reduced slab thickness and nonprestressed steel use. This reduction in material use typically offsets the cost of the post-tensioning materials and labor.

Additional documentation can be viewed at http://ww2.post-tensioning.org/PDF_FILES/190102-DC10.5-Expansive and Stable Soils-Public Review.pdf.

Bibliography: .

Cost Impact: The code change proposal will not increase or decrease the cost of construction
Post-tensioned slabs are commonly used on expansive and stable soils for crack control as well as reduced slab thickness and nonprestressed steel use. This reduction in material use typically offsets the cost of the post-tensioning materials and labor.

S154-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee could find sufficient justification to add the provision to the code and unfortunately the committee could not question the proponent (not present).
(Vote: 14-0)

Assembly Action: **None**

S154-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 1907.1

Proponents:
Amy Dowell, Post-tensioning Institute, representing Post-Tensioning Institute (amy.dowell@post-tensioning.org); Stephen Szoke, representing

American Concrete Institute (steve.szoke@concrete.org); Kerry Sutton (kerry.sutton@concrete.org)

requests As Modified by Public Comment

Replace as follows:

2018 International Building Code

1907.1 General. The thickness of concrete floor slabs supported directly on the ground shall be not less than $3^1/_2$ inches (89 mm). Post-tensioned concrete slabs shall be designed in accordance with PTI DC10.5. A 6-mil (0.006 inch; 0.15 mm) polyethylene vapor retarder with joints lapped not less than 6 inches (152 mm) shall be placed between the base course or subgrade and the concrete floor slab, or other *approved* equivalent methods or materials shall be used to retard vapor transmission through the floor slab.

Exception: A vapor retarder is not required:

1. For detached structures accessory to occupancies in Group R-3, such as garages, utility buildings or other unheated facilities.
2. For unheated storage rooms having an area of less than 70 square feet (6.5 m^2) and carports attached to occupancies in Group R-3.
3. For buildings of other occupancies where migration of moisture through the slab from below will not be detrimental to the intended occupancy of the building.
4. For driveways, walks, patios and other flatwork that will not be enclosed at a later date.
5. Where *approved* based on local site conditions.

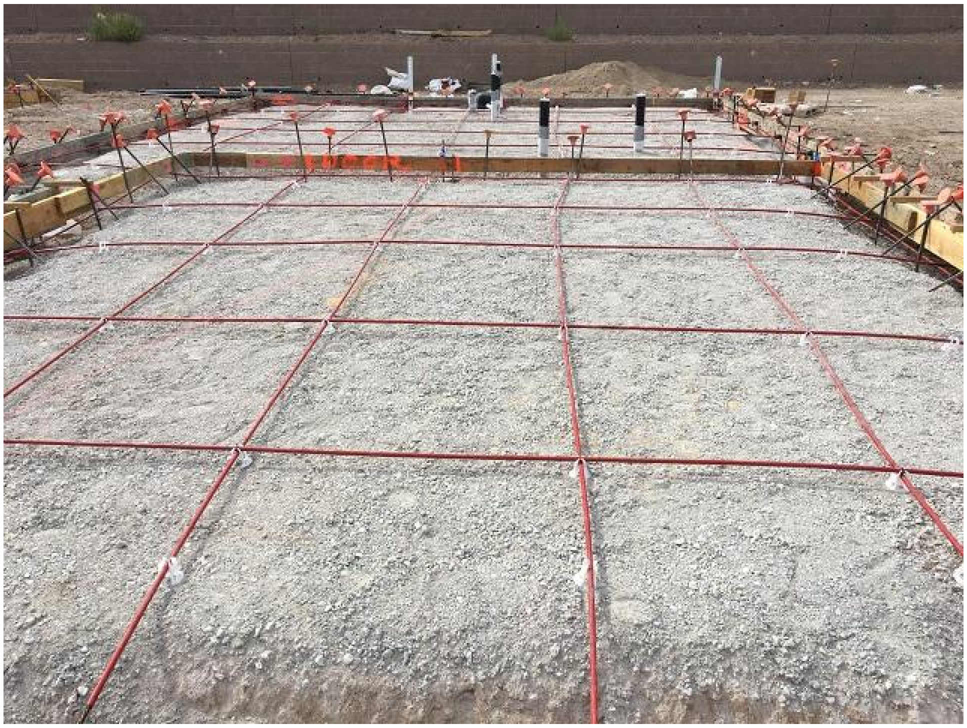
Commenter's Reason: Slab-on-ground foundations on stable and expansive soils are often post-tensioned and there is currently no design standard in the code to guide designers. The PTI DC10.5: Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive and Stable Soils is already referenced (Section 1808.6.2) for expansive soils and there is no parallel reference for stable soils. The added reference in this section clarifies that the standard shall be used for design of a post-tensioned concrete slab, not just for expansive soil sites.
The industry has a history of successful designs of post-tensioned slab-on-ground foundations on stable soil sites. Benefits include crack reduction, as well as reduced steel and concrete use. To demonstrate the history of post-tensioned slab-on-ground foundations, the following examples are two such regions where stable soils are common and post-tensioned slab-on-ground foundations have been successfully designed using PTI DC10.5 design principles:

1. Las Vegas, NV - one designer estimates that 50% of the post-tensioned slab-on-ground foundations constructed in the region are on stable soil sites and are designed using PTI DC10.5 design principles. Examples of slab-on-ground construction on stable sites are seen in the attached photos.

2. in the Southern US, one post-tensioning supplier reports the following statistics for slabs constructed on stable soil sites:

Projects on Stable Soil Sites - Florida Example

Year	# Contractors	#Projects	Total Project Square Footage
2014	3	10	1,113,693
2015	4	12	789,948
2016	4	23	2,640,620
2017	3	12	1,074,341
2018	4	16	1,113,693
2019 (ytd)	5	16	1,970,864



Bibliography: PTI DC10.5-19: Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive and Stable Soils
https://www.post-tensioning.org/publications/store/productdetail.aspx?ItemID=DC105&Language=English&Units=US_AND_METRIC

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
Post-tensioned slabs are commonly used on expansive and stable soils for crack control as well as reduced slab thickness and nonprestressed steel use. This reduction in material use typically offsets the cost of the post-tensioning materials and labor.

Public Comment# 1779

S156-19

IBC: 2109.2.4.8, 2109.2.4.8.1 (New), 2109.2.4.8.2 (New), 2109.2.4.8.3 (New), 2109.2.4.8.4 (New), 2109.2.4.8.5 (New), 2109.2.4.8.6 (New)

Proposed Change as Submitted

Proponents: K. Ben Loescher, AIA, Loescher Meachem Architects, representing Self (bloescher@lmarchitectsinc.com); Martin Hammer, representing Martin Hammer, Architect (mfhammer@pacbell.net); David Eisenberg, representing DCAT (strawnet@gmail.com)

2018 International Building Code

Delete and substitute as follows:

~~**2109.2.4.8 Exterior finish.** *Exterior walls* constructed of unstabilized adobe units shall have their exterior surface covered with not fewer than two coats of Portland cement plaster having a minimum thickness of $^3/_4$ inch (19.1 mm) and conforming to ASTM C926. Lathing shall comply with ASTM C1063. 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.~~

2109.2.4.8 Exterior finish. Exterior finishes applied to adobe masonry walls shall be of any type permitted by this code, and shall comply with the provisions of this section and with Chapter 14, except where stated otherwise in this section.

Add new text as follows:

2109.2.4.8.1 Purpose, and type. Unstabilized adobe masonry walls shall be finished on their exterior with a plaster of any type in this section to provide protection from weather in accordance with this code.

2109.2.4.8.2 Vapor retarders and vapor permeance. Class I and II vapor retarders shall not be used on any adobe masonry wall, nor shall any other material be used that has a vapor permeance rating of less than 5 perms.

2109.2.4.8.3 Plaster thickness and coats. Plaster applied to adobe masonry shall be not less than 7/8" (22 mm) and not greater than 2 inches (51 mm) thick. Plaster shall be applied in not less than two coats.

2109.2.4.8.4 Plaster application. Plaster shall be applied directly to adobe masonry walls without any type of membrane to facilitate transpiration of moisture from the masonry units, and to secure a mechanical bond between the masonry and plaster.

2109.2.4.8.5 Lath for plaster. Lath shall be provided for all plasters, except as otherwise not required in this section. Fasteners shall be spaced at 16 inches (406mm) on center maximum. Metal lath shall comply with ASTM C1063, as modified by this section, and shall be corrosion resistant. Plastic lath shall comply with ASTM C1788, as modified by this section.

2109.2.4.8.6 Cement plaster. Cement plaster shall conform to ASTM C926 and shall comply with Chapter 25, except that the proportion of lime in plaster coats shall not be less than 1 part lime to 6 parts cement to allow a minimum acceptable vapor permeability. The combined thickness of plaster coats shall not be more than 1 inch (25mm).

ASTM

ASTM International
100 Barr Harbor Drive, P.O. Box C700
West Conshohocken PA 19428-2959

C1788-14: Standard Specification for Non Metallic Plaster Bases (Lath) Used with Portland Cement Based Plaster in Vertical Wall Applications

Reason: Even more than wood frame or conventional masonry structures, adobe walls require vapor permeable finishes to ensure appropriate performance and service life; moisture that is trapped within adobe wall assemblies can cause failures due to finish separation, salt attack, coving and freeze-thaw related spalling. Although it is accepted that earthen walls require vapor permeable finishes to adequately manage moisture in the assembly and prevent various structural and finish pathologies, existing code language remains based on legacy language that predates current building science. Notably, while stabilized adobes do not require any exterior finishes, unstabilized adobes are required to be finished with conventional cement stucco, a finishing system that without modification has been shown to be insufficiently permeable. Research has shown that simply increasing the lime proportion in ordinary cement plasters can increase vapor permeability to acceptable levels.
Other comments related to this proposal:

- *Necessity: Unstabilized adobe masonry walls are subject to erosion from precipitation. As most of Section 2109 presumes that adobe masonry is used in structural applications, protective finishes are required to prevent structural failures from erosion, coving, and freeze/thaw related spalling.*
- *It is accepted that earthen building materials require exterior finishes that are vapor permeable in order to facilitate drying from moisture that may enter the wall assembly through roof or finish defects, condensation, plumbing failures, flooding, and capillary action from adjacent construction. In the presence of moisture and in the absence of vapor permeable finishes, earthen wall systems are subject to failure due to loss of integrity of the clay/sand matrix, liquification and/or salt-attack. (ASTM E2392)*

- *Plaster Thickness: The 7/8" minimum thickness requirement is identical to one that has existed successfully in the New Mexico Earthen Building Materials Code. Limits on the maximum thickness of applied plasters are required to ensure that the applied renders are securely bonded to the substrate. The New Mexico Earthen Building Materials Code includes no limit on the thickness of plasters; the 2" maximum proposed here is identical to that currently existing in IRC Appendix S.*
- *Vapor Retarders: Class I and II vapor retarders are prohibited here as they are effectively impermeable, having perm ratings of less than 1.*
- *Minimum Perm Rating: Although in many cases higher permeability would be desirable, for purposes of this proposal a minimum perm rating of 3.5 has been established as it allows the use of a 1:1:6 lime amended cement stucco with an applied siloxane water repellant (3.54 perms at 41 mm of thickness per Straube). 1:1:6 stuccos are applied with the same methods as 1:3 stuccos, at similar cost, and have similar durability. Surface applied siloxane based water repellents are effective at inhibiting water infiltration through plaster skins and desired by industry.*
- *Direct application is required as intermediate substrates may inhibit the beneficial outward movement of moisture, and introduces questions of mechanical attachment that cannot adequately be addressed within the scope of this proposal.*
- *Metallic laths are conventionally used for Portland cement based plasters. Requirements and conditions for their use need to be provided.*
- *ASTM C 1063: "Installation of Lathing and Furring to Receive Interior and Exterior Portland Cement-Based Plaster". This is the reference standard used elsewhere in the IBC to describe the material and practice requirements for the installation of metallic lathing.*
- *ASTM C926: This Standard Specification for the Application of Cement Stucco is the accepted reference standard for the materials and practices associated with cement stuccos.*
- *Lime requirement: Complimentary to the minimum vapor permeability requirements, this section requires lime to be added to cement stuccos. The consituents of conventional cement stuccos sometimes vary but are typically 1 part cement to 3 parts sand; based on Straube, this formulation yielded only 0.68 perms. The formulation proposed by this section yields 5.13 perms at 35 mm in thickness, or 3.54 perms at 41 mm of thickness when treated with siloxane, providing adequate (but not optimal) permeability while retaining desirable durability characteristics and application procedures of conventional cement stuccos. In both cases, permeability exceeds 5 perms at a 25mm (conventional applied thickness)*
- *Maximum Thickness: Limits on the thickness of applied plasters are required to ensure that the applied renders are securely bonded to the substrate. the 1 1/2" maximum proposed here is identical to that currently existing in IRC Appendix S, the 1" maximum for cement based plasters is required to achieve permeability of greater than 5 perms.*
- *Vapor Permeability of various finishes (per Straube):*

Sample	t [mm]	Permeance [ng/Pa s m^2]	Permeability [ng/Pa s m]	US Perms
Cement :Sand				
1:3 datum	43.5	39	1.7	0.68
1:3 elastomeric coating	39.5	40	--	0.70
1:3 siloxane	41.0	40	1.7	0.70
Cement:Lime:Sand				
1:1:6 datum	35	295	10.3	5.13
1:1:6 linseed	36	223	8.0	3.89
1:1:6 elastomeric	32.5	244	--	4.25
1:1:6 siloxane	41	203	8.3	3.54
1:1:6 calcium stearate	53.5	81	4.3	1.42
1:1:6 calcium stearate	44	142	6.2	2.47
1:1:6 calcium stearate	53.5	41	2.2	0.71
1:1:6 latex paint	36.5	203	--	3.54
1:1:6 oil paint	40	41	--	0.71
Cement:Lime:Sand				
1:2:9 datum	50.5	295	14.9	5.13
1:2:9 linseed	50.5	259	13.1	4.52
Lime:Sand				
1:3 Datum	33.5	565	18.9	9.85
1:3 Datum	35.5	529	18.8	9.22
1:3 Quicklime	32	459	14.7	8.00

Table 2.3: Results of Vapor Permeance Test Results [Straube, 2000]

Bibliography: 2015 New Mexico Earthen Building Materials Code
2015 *International Residential Code Appendix S - Strawbale Construction*

Adobe Conservation: A Preservation Handbook. Compiled by the Technical Staff of Cornerstones Community Partnership. Sunstone Press (Santa Fe, 2006)

ASTM E2392 / E2392M - 10(2016) Standard Guide for Design of Earthen Wall Building Systems

Building with Earth: Design and Technology of Sustainable Architecture. Gernot Minke, Birkhauser (Bern, 2009)

Earth Construction: A Comprehensive Guide. Hugo Houben and Hubert Guillaud, Practical Action Publishing (Rugby, 2008).

Moisture Properties of Plaster and Stucco for Strawbale Buildings. John Straube. (EBNET, 2019)

Sustainable Building with Earth. Horst Schroeder, Springer International Publishing (Switzerland, 2016)

Cost Impact: The code change proposal will decrease the cost of construction
In most cases, the proposed code language expands the options available to design professionals and contractors for the finishing of adobe wall systems without additional cost impact. The inclusion of earthen plasters in particular cases will decrease the cost of construction for some projects.

Staff Analysis: A review of the standard proposed for inclusion in the code, ASTM C1788-14, with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2019.

S156-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee found the proposal as written to be confusing and possibly more suited for an appendix.
(Vote: 11-3)

Assembly Action: **None**

S156-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 2109.2.4.8, 2109.2.4.8.1 (New), 2109.2.4.8.2 (New), 2109.2.4.8.3 (New), 2109.2.4.8.4 (New), 2109.2.4.8.5 (New), 2109.2.4.8.6 (New)

Proponents:
Ben Loescher, representing The Earthbuilders' Guild (bloescher@lmarchitectsinc.com); David Eisenberg, DCAT, representing DCAT (strawnet@gmail.com); Martin Hammer, representing Martin Hammer, Architect (mfhammer@pacbell.net)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

2109.2.4.8 Exterior finish. Exterior finishes applied to adobe masonry walls shall be of any type permitted by ~~this code and shall comply with the provisions of~~ this section ~~and~~ or ~~with~~ Chapter 14, except where stated otherwise in this section.

2109.2.4.8.1 ~~Purpose, and type~~ Where required. Unstabilized adobe masonry walls shall ~~be finished on their exterior with a plaster of any type in this section to provide protection from weather~~ receive a weather protective exterior finish in accordance with ~~this code~~ Section 2109.2.4.8.

2109.2.4.8.2 ~~Vapor retarders and v~~ Vapor permeance. ~~Class I and II vapor retarders shall not be used on any adobe masonry wall,. nor shall any other material be used that has a vapor permeance rating of less than 5 perms.~~ Plaster and finish assemblies shall have a vapor permeance of not less than 5 perms.

Exception: Insulation products applied to the exterior of stabilized adobe masonry walls in Climate Zones 2B, 3B, 4B and 5B shall have no vapor permeance requirement.

2109.2.4.8.3 Plaster thickness and coats. Plaster applied to adobe masonry shall be not less than 7/8" (22 mm) and not greater than 2 inches (51 mm) thick. Plaster shall be applied in not less than two coats.

2109.2.4.8.4 Plaster application. ~~Plaster shall be~~ Where plaster is applied directly to adobe masonry walls, no intermediate membrane shall be used. ~~any type of membrane to facilitate transpiration of moisture from the masonry units, and to secure a mechanical bond between the masonry~~

~~and plaster~~.

2109.2.4.8.5 Lath for plaster. Lath shall be provided for all plasters, except ~~as otherwise~~ where not required elsewhere in ~~this section~~ Section 2019.2.4.8. Fasteners shall be corrosion resistant and spaced at a maximum of 16 inches (406mm) on center ~~maximum~~ with a minimum 1-1/2 inches (38 mm) penetration into the adobe wall. Metal lath shall comply with ASTM C1063, as modified by this section, and shall be corrosion resistant. Plastic lath shall comply with ASTM C1788, as modified by this section. Wood substrates shall be protected with #15 asphalt felt, an *approved* wood preservative or other protective coating prior to lath application.

2109.2.4.8.6 Cement plaster. Cement plaster shall conform to ASTM C926 and shall comply with Chapter 25, except that the proportion of lime in plaster coats shall not be less than 1 part lime to ~~6~~ 4 parts cement ~~to allow a minimum acceptable vapor permeability.~~ The combined thickness of cement plaster coats shall not ~~be more than~~ exceed 1 inch (25mm).

Commenter's Reason: Proposal S156-19 was the first of a set of four proposals addressing finishes on adobe walls. This set of proposals was intended to address a serious flaw in the existing provisions related to the permeance of finishes on adobe walls, as well as add needed provisions for all finishes, and provide an appropriate place in the section for the addition of the finishes proposed in the following three proposals that were ultimately approved by the Committee.
Three factors resulted S156 being disapproved. Two were the result of confusion that was evident as the Committee heard proposal S156. The third was the result of concerns about wording specific to the proposal, which is addressed by this public comment.

First was the decision to separate these changes into four separate proposals, with the intention of making sure that the paramount concern, the permeance issue was addressed in S156, regardless of the potential outcome for the newly proposed plaster types that were in the other three proposals. Ironically, the outcome was the opposite when S156 was disapproved.

Second was the result of formatting and section numbering changes in the cdpACCESS process that made the four proposals appear as separate and independent from each other, rather than as S156 clearly being the overarching section under which the new sections for lime plasters, lime-cement plasters, and clay plasters proposed and approved in S157, S158, and S159 would exist. For clarity, we have included below, how the entire Adobe finishes section would appear if this public comment is approved, including the sections already approved from proposals S157-19, S158-19, and S159-19.

Third, were the Committee's specific concerns about language that needed improvement, resulting in S156 being disapproved. As the other three proposals were heard, greater clarity emerged about both the importance of addressing the permeance issue and added requirements for finishes, and the need for the structure that S156 provided for the other proposals. The Committee approved the other three proposals as submitted, in part to strengthen the case for approval of a public comment on S156. There were strong recommendations from committee members about the importance of addressing the permeance issue for finishes.

The specific changes made in response to the committee's comments are:

- eliminating ambiguity and removing language that was essentially commentary,

- clarifying language related to plaster thickness, permeance, substrates, the lathing exception, and the protection of wood substrates.

- clarifying language related to plaster thickness, permeance, substrates, the lathing exception, and the protection of wood substrates. An exception was added to allow less permeable insulation products in response to input from industry. Impermeable insulation products applied over asphalt-emulsion stabilized adobe walls have been used successfully for over thirty years in low humidity regions of the western United States. Their continued use is important to achieve energy code compliance until vapor permeable insulation products suitable for use beneath plaster or other finishes become readily available.

Note: In the Committee Reason for their vote to disapprove this proposal, there is mention of these provisions possibly being more suited for an appendix. Mention of an appendix was entirely the result of the mistaken comment of a person testifying in support having said these provisions were in an appendix. When it was pointed out that the Adobe provisions have been in the body of the IBC from the very first version, the person apologized and asked the Committee to disregard that part of his testimony. Nothing in the proposal, nor in other testimony or direct discussion among the Committee recommended moving this section to an appendix.

Background

Like traditional solid masonry walls, adobe masonry walls require vapor permeable finishes to ensure appropriate performance and service life. Moisture that is trapped within adobe wall assemblies behind impermeable finishes can lead to failures such as finish separation, salt attack, coving and freeze-thaw related spalling, and in extreme cases, structural collapse. Although this is widely known, existing language allowing non-permeable cement plasters remains in the code based on legacy language that predates current building science. Notably, while the current code does not require an exterior finish for stabilized adobe walls, it requires that unstabilized adobe walls be finished with conventional cement stucco, a finishing system that, without the modifications in this public comment, is vapor impermeable.

This proposal includes the addition of language for permeable finishes and is informed by code provisions and guidance from the 2015 *New Mexico*

Earthen Building Materials Code, ASTM E2392-10 *Standard Guide for Design of Earthen Wall Building Systems,* and IRC Committee approved changes for the 2021 IRC Appendix S - Strawbale Construction.

For overall clarity about how the four proposals (S156-19, S157-19, S158-19, and S159-19) were intended to relate to each other, below is how full Adobe Finishes section would look renumbered, and including the changes proposed in this public comment as well as those from the already approved proposals listed above, which are shown *italicized*:

2109.2.4.8 Exterior finish. Exterior finishes applied to adobe masonry walls shall be of any type permitted by this section or Chapter 14, except where stated otherwise in this section.

2109.2.4.8.1 Where required. Unstabilized adobe masonry walls shall receive a weather protective exterior finish in accordance with Section 2109.2.4.8.

2109.2.4.8.2 Vapor permeance. Plaster and finish assemblies shall have a vapor permeance of not less than 5 perms.

Exception: Insulation products applied to the exterior of stabilized adobe masonry walls in Climate Zones 2B, 3B, 4B and 5B shall have no vapor permeance requirement.

2109.2.4.8.3 Plaster thickness and coats. Plaster applied to adobe masonry shall be not less than 7/8" (22 mm) and not greater than 2 inches (51 mm) thick. Plaster shall be applied in not less than two coats.

2109.2.4.8.4 Plaster application. Where plaster is applied directly to adobe masonry walls, no intermediate membrane shall be used.

2109.2.4.8.5 Lath for plaster. Lath shall be provided for all plasters, except where not required elsewhere in section 2109.2.4.8. Fasteners shall shall be corrosion resistant and spaced at a maximum of 16 inches (406mm) on center with a minimum 1-1/2 inches (38 mm) penetration into the adobe wall. Metal lath shall comply with ASTM C1063, as modified by this section, and shall be corrosion resistant. Plastic lath shall comply with ASTM C1788, as modified by this section. Wood substrates shall be treated with #15 asphalt felt, an *approved* wood preservative or other protective coating prior to lath application.

2109.2.4.8.6 Cement plaster. Cement plaster shall conform to ASTM C926 and comply with Chapter 25, except that the proportion of lime in plaster coats shall not be less than 1 part lime to 4 parts cement. The combined thickness of cement plaster coats shall not exceed 1 inch (25 mm).

2109.2.4.8.7 Lime Plaster. *Lime plaster is any plaster with a binder composed of calcium hydroxide, including Type N or S hydrated lime, hydraulic lime, natural hydraulic lime, or slaked quicklime. Hydrated lime shall comply with ASTM C206. Hydraulic lime shall comply with ASTM C1707. Natural hydraulic lime shall comply with ASTM C141 and EN 459. Quicklime shall comply with ASTM C5.*

2109.2.4.8.8 Cement-lime plaster. *Cement-lime plaster shall be any plaster mix type CL, F or FL, as described in ASTM C926.*

2109.2.4.8.9 Clay Plaster. *Clay plaster shall comply with this section.*

2109.2.4.8.9.1 General. *Clay plaster shall be any plaster having a clay or clay subsoil binder. Such plaster shall contain sufficient clay to fully bind the aggregate, and shall be permitted to contain reinforcing fibers. Acceptable reinforcing fibers include chopped straw, sisal, and animal hair.*

2109.2.4.8.9.2 Clay subsoil requirements. *The suitability of clay subsoil shall be determined in accordance with the Figure 2 Ribbon Test and the Figure 3 Ball Test in the appendix of ASTM E2392/E2392M.*

2109.2.4.8.9.3 Weather exposed locations. *Clay plaster exposed to water from direct or wind-driven rain, or snow, shall be finished with an approved erosion-resistant finish. The use of clay plasters shall not be permitted on weather exposed parapets.*

2109.2.4.8.9.4 Prohibited finish coat. *Plaster containing Portland cement shall not be permitted as a finish over clay plaster.*

2109.2.4.8.9.5 Conditions where lathing is not required. *For unstabilized adobe walls finished with unstabilized clay plaster, lathing shall not be required.*

Bibliography: 2015 New Mexico Earthen Building Materials Code
Adobe Conservation: A Preservation Handbook. Compiled by the Technical Staff of Cornerstones Community Partnership. Sunstone Press (Santa Fe, 2006)

ASTM E2392 / E2392M - 10(2016) Standard Guide for Design of Earthen Wall Building Systems

Building with Earth: Design and Technology of Sustainable Architecture. Gernot Minke, Birkhauser (Bern, 2009)

Earth Construction: A Comprehensive Guide. Hugo Houben and Hubert Guillaud, Practical Action Publishing (Rugby, 2008).

Moisture Properties of Plaster and Stucco for Strawbale Buildings. John Straube. (EBNET, 2019)

Sustainable Building with Earth. Horst Schroeder, Springer International Publishing (Switzerland, 2016)

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
The proposed change has no impact on the cost of construction. In most cases, the proposed code language expands the options available to design professionals and contractors for the finishing of adobe wall systems without additional cost impact.

Public Comment# 1622

S162-19

IBC: SECTION 2209, 2209.2, 2209.3 (New), 2209.4 (New), 2209.5 (New), MHI Chapter 35 (New)

Proposed Change as Submitted

Proponents: Paul Armstrong, MHI, representing MHI

2018 International Building Code

Revise as follows:

SECTION 2209
STEEL ~~STORAGE RACKS~~ MATERIAL HANDLING STRUCTURES

2209.1 Storage racks. The design, testing and utilization of *storage racks* made of cold-formed or hot-rolled steel structural members shall be in accordance with RMI ANSI/MH 16.1. Where required by ASCE 7, the seismic design of *storage racks* shall be in accordance with Section 15.5.3 of ASCE 7.

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 of ASCE 7.

Add new text as follows:

2209.3 Industrial boltless steel shelving. The design and utilization of industrial boltless steel shelving shall be in accordance with ANSI/MH28.2.

2209.4 Industrial steel work platforms The design and utilization of industrial steel work platforms shall be in accordance with ANSI/MH28.3.

2209.5 Stairs, ladders and guards. The design and utilization of stais, ladders and op;en edge guards for use with material handling structures shall be in accordance with ANSI/MH32.1.

MHI

Material Handling Institute
8720 Red Oak Blvd. Suite 201
Charlotte NC 28217

MH28.3-2018: Design, Testing and Utilization of Industrial Steel Work Platforms

MHI

Material Handling Institute
8720 Red Oak Blvd. Suite 201
Charlotte NC 28217

MH28.2-2018: Design, Testing and Utilization of Industrial Boltless Steel Shelving

MHI

Material Handling Institute
8720 Red Oak Blvd. Suite 201
Charlotte NC 28217

MH32.1-2018: Stairs, Ladders, and Open-Edge Guards for Use with Material Handling Structures

Reason: SMA has developed new standards for the design, testing and installation of both steel work platforms and boltless steel shelving structures. They are ANSI accredited now and are included for review.

Cost Impact: The code change proposal will decrease the cost of construction
These standards will reduce the cost of construction by providing a uniform set of code regulations for the design and installation of such structures. Currently the imposed regulations seem to change based on the jurisdiction and/or plan reviewer.

Staff Analysis: A review of the standard proposed for inclusion in the code, MHI MH28.2-2018, MH28.3-2018 and MH32.1-2018, 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 2, 2019.

S162-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: Proponent requested disapproval. The committee felt the proposed list was incomplete and possibly being proposed for the wrong place in the code. (Vote: 10-4)

Assembly Action: **None**

S162-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 2209.3 (New), 2209.4 (New)

Proponents:
Paul Armstrong, MHI, representing MHI (paul.armstrong@pacodeservices.com)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

2209.3 Industrial boltless steel shelving. The design and utilization of industrial boltless steel shelving shall be in accordance with ANSI/MH28.2 as amended by 2209.3.1 through 2209.3.3.

2209.3.1 Section 1.1. Modify Section 1.1, by adding a concluding paragraph as follows: “Designers are cautioned when doing seismic design to obtain more detailed information, amendments and changes to adopted codes, including applicable design ground motions to use for design, by contacting the local jurisdiction where the racks are to be installed”.

2209.3.2 Section 4.6.2. Modify Section 4.6.2, by replacing the web site reference from http://earthquake.usgs.gov/designmaps/us/application.php to http://www.seaoc.org/page/seismicdesignmaptool.

2209.3.3 Section 4.6.6. Modify Section 4.6.6, by replacing b) to read as follows: “b) when the loss of seismic-resistance capacity in a single beam, post, or brace does not result in more than a 33% reduction in required seismic-resistance capacity of the structure (in the cross-aisle direction, this typically requires a minimum of two back-to-back rows interconnected); and”.

2209.4 Industrial steel work platforms The design and utilization of industrial steel work platforms shall be in accordance with ANSI/MH28.3 as amended by 2209.4.1 and 2209.4.2

2209.4.1 Section 1.1. Modify Section 1.1, by adding a concluding paragraph as follows: “Designers are cautioned when doing seismic design to obtain more detailed information, amendments and changes to adopted codes, including applicable design ground motions to use for design, by contacting the local jurisdiction where the racks are to be installed”.

2209.4.2 Section 4.6.3. Modify Section 4.6.3, by adding the following paragraph after “i) = 8 for a special moment frame;”, “The use of any of the above listed ‘R’ values is permissible provided that the seismic detailing requirements per AISC 341-16 are followed for the selected seismic force-resisting system (does not apply to those force-resisting systems not design to resist seismic loads). Detailing requirements for structural systems not specifically detailed for seismic resistance are found in AISC 360-16. Platforms not specifically detailed for seismic resistance using an R = 3 per ASCE 7 Table 12.2-1 are restricted to Seismic Design Categories A, B and C.

Commenter's Reason: During the Code Development Committee Hearings, feedback was received from FEMA's Seismic Code Support Committee. Unfortunately, there wasn't enough time at the hearings to development a modification. This Public Comment has been developed with their input from the hearings and subsequent conference calls. It was agreed that a few other items would be reviewed and considered for the next edition of each standard.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction

This just provides design standards for the systems identified in each standard.

Public Comment# 2058

S165-19

IBC: 2303.1.9, 2303.1.9.3 (New), 2303.2, 2303.2.5, 2306.1.3

Proposed Change as Submitted

Proponents: David P Tyree, American Wood Council, representing American Wood Council (dtyree@awc.org); Paul Coats, representing American Wood Council (pcoats@awc.org)

2018 International Building Code

2303.1.9 Preservative-treated wood. Lumber, timber, plywood, piles and poles supporting permanent structures required by Section 2304.12 to be preservative treated shall conform to AWPA U1 and M4. Lumber and plywood used in permanent wood foundation systems shall conform to Chapter 18.

Add new text as follows:

2303.1.9.3 Strength Adjustments. Design values for preservative-treated wood in accordance with Section 2303.1.9 do not need adjustment for the type of preservative used. Other adjustments in accordance with AWC NDS shall apply. Load duration factors for structural members pressure-treated with water-borne preservatives shall not exceed 1.6.

Revise as follows:

2303.2 Fire-retardant-treated wood. *Fire-retardant-treated wood* is any wood product that, when impregnated with chemicals by a pressure process or other means during manufacture, shall have, when tested in accordance with ASTM E84 or UL 723, a *listed* flame spread index of 25 or less and show no evidence of significant progressive combustion when the test is continued for an additional 20-minute period. Additionally, the flame front shall not progress more than $10^1/_2$ feet (3200 mm) beyond the centerline of the burners at any time during the test.

2303.2.5 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 in accordance with AWC NDS shall apply. Load duration factors for structural members pressure-treated with fire retardant chemicals shall not exceed 1.6.

Delete without substitution:

~~**2306.1.3 Treated wood stress adjustments.** The allowable unit stresses for *preservative-treated wood* need not be adjusted 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 with Section 2303.2.5 and can be deleted. Location of design value information in 2303.2.5 as opposed to 2306 on Allowable 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 wood. Additional description of specific revisions follows:
Section 2303.1.9.3. Sentence 1 clarifies that no adjustment is associated with the type of preservative used. The second sentence is consistent with AWC NDS Table 2.3.2 in which load duration adjustment is not to exceed 1.6 for structural members pressure treated with water-borne preservatives and is a smaller adjustment than the factor of 2.0 associated with impact load duration.

Section 2303.2.5. The second sentence is consistent with AWC NDS Table 2.3.2 in which load duration adjustment is not to exceed 1.6 for structural members pressure treated with fire-retardant chemicals and is a smaller adjustment than the factor of 2.0 associated with impact load duration.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
Clarification of current requirements and referenced standards.

S165-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee expressed apprehension that the proposal had not been vetted and coordinated throughout the industry.

(Vote: 14-0)

Assembly Action: **None**

S165-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 2303.1.9, 2303.1.9.3 (New), 2303.2, 2303.2.5, 2306.1.3

Proponents:
Christopher Athari, representing Hoover Treated Wood Products (cathari@frtw.com)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

2303.1.9 Preservative-treated wood. Lumber, timber, plywood, piles and poles supporting permanent structures required by Section 2304.12 to be preservative treated shall conform to AWPA U1 and M4. Lumber and plywood used in permanent wood foundation systems shall conform to Chapter 18.

2303.1.9.3 Strength Adjustments. Design values for preservative-treated wood in accordance with Section 2303.1.9 do not need adjustment for the type of preservative used. Other adjustments in accordance with AWC NDS shall apply. Load duration factors for structural members pressure-treated with water-borne preservatives shall not exceed 1.6.

2303.2 Fire-retardant-treated wood. *Fire-retardant-treated wood* is any wood product that, when impregnated with chemicals by a pressure process or other means during manufacture, shall have, when tested in accordance with ASTM E84 or UL 723, a *listed* flame spread index of 25 or less and show no evidence of significant progressive combustion when the test is continued for an additional 20-minute period. Additionally, the flame front shall not progress more than $10^1/_2$ feet (3200 mm) beyond the centerline of the burners at any time during the test.

2303.2.5 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 in accordance with AWC NDS shall apply. Load duration factors for structural members pressure-treated with fire retardant chemicals shall not exceed 1.6.~~ Duration of load factors, except impact load duration, in accordance AWC NDS shall apply.

2306.1.3
Treated wood stress adjustments.

Commenter's Reason: The reason statement implied that no technical changes were intended. The submission did include technical changes. This comment brings the language from 2306.1.3 into the section for FRTW.using the last snetence in the section and alerting the user to other load duration factors in the NDS.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
Updating for new material.

Public Comment# 1686

Public Comment 2:

IBC®: 2303.2.5, 2303.2.5.1, 2303.2.5.2

Proponents:
Jason Smart, American Wood Council, representing American Wood Council (jsmart@awc.org)

requests As Modified by Public Comment

Replace as follows:

2018 International Building Code

2303.2.5 Strength adjustments. Design values for ~~untreated~~ fire-retardant-treated lumber and wood structural panels shall be determined using published design values and adjustments for untreated lumber and wood structural panels, as specified~~ in Section 2303.1, shall be adjusted for *fire-retardant-treated wood*~~ elsewhere in this chapter, and further adjusted to account for the effects of treatment. Adjustments ~~to design values~~ for the effects of treatment 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.

2303.2.5.1 Wood structural panels. The effect of treatment, ~~and~~ the method of redrying after treatment, and any treatment-based degradation due to exposure to high temperatures and high humidities on the flexure properties of fire-retardant-treated softwood plywood shall be determined in accordance with ASTM D5516. The test data developed by ASTM D5516 shall be used to develop treatment adjustment factors~~, maximum loads and spans, or both, for untreated plywood design values~~ in accordance with ASTM D6305. Each manufacturer shall publish the allowable maximum loads and spans for service as floor and roof sheathing for its treatment.

2303.2.5.2 Lumber. For each species of wood that is treated, the effects of the treatment, the method of redrying after treatment, and any treatment-based degradation due to exposure to high temperatures and high humidities on the allowable design properties of fire-retardant-treated lumber shall be determined in accordance with ASTM D5664. The test data developed by ASTM D5664 shall be used to develop ~~modification~~ treatment adjustment factors for use at or near room temperature and at elevated temperatures and humidity in accordance with ASTM D6841. Each manufacturer shall publish the modification factors for service at temperatures of not less than 80°F (27°C) and for roof framing. The roof framing modification factors shall take into consideration the climatological location.

Commenter's Reason: Section 2303.2.5 is revised to clarify that design values for fire-retardant-treated wood products are based on design values for untreated wood products that have been adjusted for end-use conditions in accordance with Chapter 23 provisions and also adjusted to account for the effect of the fire-retardant treatment. This clarification aligns with ASTM D5664/D6841 for lumber and ASTM D5516/D6305 for plywood. In both cases, the FRT adjustment factors isolate the additional effect of the fire-retardant treatment, but do not address how the constituent untreated wood materials themselves need to be adjusted for typical application conditions. For this reason, design values for FRT wood products must be adjusted by factors that are applicable to untreated wood as well as the treatment adjustment factors.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
The revisions in this public comment simply provide a clarification of what is already required for designs utilizing fire-retardant-treated wood products.

Public Comment# 2012

Public Comment 3:

IBC®: 2303.1.9, 2303.1.9.3 (New), 2303.2, 2303.2.5, 2303.2.5.1, 2303.2.5.2, 2306.1.3

Proponents:
Jason Smart, American Wood Council, representing American Wood Council (jsmart@awc.org)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

2303.1.9 Preservative-treated wood. Lumber, timber, plywood, piles and poles supporting permanent structures required by Section 2304.12 to be preservative treated shall conform to AWPA U1 and M4. Lumber and plywood used in permanent wood foundation systems shall conform to Chapter 18.

~~**2303.1.9.3 Strength Adjustments.** Design values for preservative-treated wood in accordance with Section 2303.1.9 do not need adjustment for the type of preservative used. Other adjustments in accordance with AWC NDS shall apply. Load duration factors for structural members pressure-treated with water-borne preservatives shall not exceed 1.6.~~

2303.2 Fire-retardant-treated wood. *Fire-retardant-treated wood* is any wood product that, when impregnated with chemicals by a pressure process or other means during manufacture, shall have, when tested in accordance with ASTM E84 or UL 723, a *listed* flame spread index of 25 or less and show no evidence of significant progressive combustion when the test is continued for an additional 20-minute period. Additionally, the flame front shall not progress more than $10^{1}/_{2}$ feet (3200 mm) beyond the centerline of the burners at any time during the test.

2303.2.5 Strength adjustments. Design values for ~~untreated~~ fire-retardant-treated lumber and wood structural panels shall be determined using published design values and adjustments for untreated lumber and wood structural panels, as specified ~~in Section 2303.1, shall be adjusted for *fire-retardant-treated wood*~~ elsewhere in this chapter, and further adjusted to account for the effects of treatment. Adjustments ~~to design values, including fastener values,~~ for the effects of treatment 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 in accordance with AWC NDS shall apply. Load duration factors for structural members pressure-treated with fire retardant chemicals shall not exceed 1.6.~~

2303.2.5.1 Wood structural panels. The effect of treatment, ~~and~~ the method of redrying after treatment, and any treatment-based degradation due to exposure to high temperatures and high humidities on the flexure properties of fire-retardant-treated softwood plywood shall be determined in accordance with ASTM D5516. The test data developed by ASTM D5516 shall be used to develop treatment adjustment factors~~, maximum loads and spans, or both, for untreated plywood design values~~ in accordance with ASTM D6305. Each manufacturer shall publish the allowable maximum loads and spans for service as floor and roof sheathing for its treatment.

2303.2.5.2 Lumber. For each species of wood that is treated, the effects of the treatment, the method of redrying after treatment, and any treatment-based degradation due to exposure to high temperatures and high humidities on the allowable design properties of fire-retardant-treated lumber shall be determined in accordance with ASTM D5664. The test data developed by ASTM D5664 shall be used to develop ~~modification~~ treatment adjustment factors for use at or near room temperature and at elevated temperatures and humidity in accordance with ASTM D6841. Each manufacturer shall publish the modification factors for service at temperatures of not less than 80°F (27°C) and for roof framing. The roof framing modification factors shall take into consideration the climatological location.

2306.1.3 Treated wood stress adjustments. The allowable unit stresses for preservative treated wood need not be adjusted 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.

Commenter's Reason: Section 2303.1.9.3, which was proposed as a new section in S165-19, is removed by this public comment. The net result is that no changes are proposed under Section 2303.1.9, based on the 2018 IBC.
Section 2303.2.5 is revised to clarify that design values for fire-retardant-treated wood products are based on design values for untreated wood products that have been adjusted for end-use conditions in accordance with Chapter 23 provisions and also adjusted to account for the effect of the fire-retardant treatment. This clarification aligns with ASTM D5664/D6841 for lumber and ASTM D5516/D6305 for plywood. In both cases, the FRT adjustment factors isolate the additional effect of the fire-retardant treatment, but do not address how the constituent untreated wood materials themselves need to be adjusted for typical application conditions. For this reason, design values for FRT wood products must be adjusted by factors that are applicable to untreated wood as well as the treatment adjustment factors.

Section 2306.1.3, which was proposed for deletion in S165-19, is reinstated by this public comment. The net result is that no changes are proposed to Section 2306.1.3, based on the 2018 IBC.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
The revisions in this public comment simply provide a clarification of what is already required for designs utilizing fire-retardant-treated wood products.

Public Comment# 2040

S166-19

IBC®: 2303.2

Proposed Change as Submitted

Proponents: Marcelo M Hirschler, GBH International, representing GBH International (mmh@gbhint.com)

2018 International Building Code

Revise as follows:

2303.2 Fire-retardant-treated wood. *Fire-retardant-treated wood* is any wood product that, when impregnated with chemicals by a pressure process or other means during manufacture, shall have, when tested in accordance with ASTM E84 or UL 723, a *listed* flame spread index of 25 or less . ~~and show no evidence of significant progressive combustion when the test is continued~~ Additionally, the ASTM E84 or UL 723 test shall be continued for an additional 20-minute period~~. Additionally,~~ and 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.~~ extended 30-minute test.

Reason: This issue has been under discussion for many years at the ICC codes, as well as at ASTM and at NFPA, but can now be resolved in the IBC code. Fire test labs have been surveyed and they all agree that there are only two fire test requirements: a flame spread index of not more than 25 in the standard ASTM E84 test and a flame front that does not progress more than 101/2 feet beyond the centerline of the burners when the ASTM E84 test is extended for a total test time of 30 minutes.
The ASTM E5 committee, responsible for ASTM E84, has now, for the first time, accepted incorporating requirements for conducting a 30 minute test. Until this change ASTM E84 did not contain any information other than that it is a 10 minute test. Consequently, until this change ASTM E84 did not provide any details on how to assess either "no evidence of significant progressive combustion" or "the flame front shall not progress more than 101/2 feet (3200 mm) beyond the centerline of the burners". The information for how to determine both of those characteristics is contained in ASTM E2768. The committee agreed that the next edition of ASTM E84 will state that a 30 minute test is to be conducted per ASTM E2768. In turn, ASTM E2768 explains that "no significant progressive combustion" is evidenced by lack of flame front progress beyond 10 1/2 feet. In fact ASTM E2768 states: "The flame front shall not progress more than 10.5 ft (3.2 m) beyond the centerline of the burners at any time during the 30 min test period. This is considered evidence of no significant progressive combustion in this test method." This IBC proposal incorporates the requirements from the ASTM E84 test into the IBC and ensures that the code does not require a duplicate (and confusing) measurement.

It is likely that information will be presented stating that "no significant progressive combustion" has been in the code since the legacy codes and that the flame front progress requirement was added later. That is exactly the reason that ASTM E2768 was developed to ensure that everyone understands what is to be measured, and that is what the testing laboratories have been doing for many years now.

This change appears to alter requirements but in fact simply recognizes what the ASTM E84 standard states and what the labs are doing (and have been doing for years) and, therefore, is really clarification.

The committee E05 (on fire standards) agreed at the December 2018 meeting that the scope of ASTM E84 should read as follows:

1. Scope

1.1 This fire-test–response standard for the comparative surface burning behavior of building materials is applicable to exposed surfaces such as walls and ceilings. The test is conducted with the specimen in the ceiling position with the surface to be evaluated exposed face down to the ignition source. The material, product, or assembly shall be capable of being mounted in the test position during the test. Thus, the specimen shall either be self-supporting by its own structural quality, held in place by added supports along the test surface,or secured from the back side.

1.2 Test Method E84 is a 10-minute fire-test response method. The following standards address testing of materials in accordance with test methods that are applications or variations of the test method or apparatus used for Test Method E84:

1.2.1 Materials required by the user to meet an extended 30-min duration tunnel test shall be tested per Test Method E2768.

1.2.2 Wires and cables for use in air-handling spaces shall be tested per NFPA 262.

1.2.3 Pneumatic tubing for control systems shall be tested per UL 1820.

1.2.4 Combustible sprinkler piping shall be tested per UL 1887.

1.2.5 Optical fiber and communications raceways for use in air handling spaces shall be tested per UL 2024.

1.3 The purpose of this test method is to determine the relative burning behavior of the material by observing the flame spread along the specimen. Flame spread and smoke developed index are reported. However, there is not necessarily a relationship between these two measurements.

1.4 The use of supporting materials on the underside of the test specimen has the ability to lower the flame spread index from those which might be obtained if the specimen could be tested without such support. These test results do not necessarily relate to indices obtained by testing materials without such support.

1.5 Testing of materials that melt, drip, or delaminate to such a degree that the continuity of the flame front is destroyed,results in low flame spread indices that do not relate directly to indices obtained by testing materials that remain in place.

1.6 The values stated in inch-pound units are to be regarded as standard. The values given in parentheses are mathematical conversions to SI units that are provided for information only and are not considered standard.

1.7 The text of this standard references notes and footnotes that provide explanatory information. These notes and footnotes, excluding those in tables and figures, shall not be considered as requirements of the standard.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This proposal recognizes what the fire test labs have been doing for years and what ASTM committee E05 has recently agreed to do in the scope of ASTM E84.

S166-19

Public Hearing Results

Committee Action: **As Modified**

Committee Modification:
2018 International Building Code

2303.2 Fire-retardant-treated wood. *Fire-retardant-treated wood* is any wood product that, when impregnated with chemicals by a pressure process or other means during manufacture, shall have, when tested in accordance with ASTM E84 or UL 723, a *listed* flame spread index of 25 or less. Additionally, the ASTM E84 or UL 723 test shall be continued for an additional 20-minute period and 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 ~~extended 30-minute~~ test.

Committee Reason: The committee felt that the proposal cleaned up the language and makes the code consistent with current test methods. The modification simplified the language. (Vote: 11-2)

Assembly Action: **None**

S166-19

Individual Consideration Agenda

Public Comment 1:

Proponents:
David Tyree, representing American Wood Council (dtyree@awc.org)

requests As Modified by Committee

Commenter's Reason: As recommended by the Code Development Committee at the Committee Action Hearings, this change cleans up and simplifies the language of 2303.2, while making it more consistent with the referenced test methods.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
Proposal represents a simple cleaning up of existing language.

Public Comment# 1678

Public Comment 2:

Proponents:
Manny Muniz, representing Representing self (mannymuniz.mm@gmail.com)

requests Disapprove

Commenter's Reason: This proposal is similar to RB255-19 which was disapproved by a vote of 11-0. The residential committee disapproved that proposal for several reasons and stated ***"The proposal has a lower safety standard. There was a debate on the technical justification in testing. The standard in the reason statement, ASTM E05, is not referenced in the ICC."***

The lower safety standard is the deletion of a 55-year-old prescriptive requirement for showing *"no evidence of significant progressive combustion"* following the test. Such a deletion is arbitrary as there was no testing done to see if safety was being lowered. That's why it was disapproved.

The Structural committee reason statement stated *"The committee felt that the proposal cleaned up the language and makes the code consistent with current test methods. The modification simplified the language."*

Making the code consistent with current test methods, as stated in the structural committee reason, is backwards as to how you change a prescriptive requirement in the code. If, in fact, a test lab felt that **"showing no evidence of significant progressive combustion"** following the test is equivalent to the additional prescriptive requirement that **"the flame front shall not progress more than 10 ½ feet beyond the centerline of the burners at any time during the test"**, they should have performed tests to support their theory and submitted a code change to ICC with that substantiation.

Instead, a code consultant representing GBH International (fire test equipment & consulting services) is the one submitting this with no substantiation. That should be a huge red flag. You just don't delete a 55-year-old prescriptive test requirement without showing why the deletion is scientifically justified.

If you combine the total votes of both committees who heard these two proposals, there was a total of 13 votes to disapprove and 11 to approve. A historical review of the requirement for fire-retardant-treated wood may be helpful.

Requirements for Fire-retardant-treated wood first appeared in the 1964 UBC and required testing for 30 minutes in the "Tunnel Test", have a flame spread of not over 25 and show no evidence of progressive combustion.

Then, 24 years later, in the 1988 UBC, the 10.5 ft flame front limitation was added ***"To maintain the same level of performance of the material achieved under the previous flame spread calculation method."***

It is important to note that the 10.5 ft flame front limitation is evaluated during the 30-minute test whereas "progressive combustion" is evaluated at the end of the 30-minute test. Similar language is contained in the International Wildland Urban Interface Code for ignition-resistant building material (503.2, 1.1 & 503.2.1.2).

"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 10 ½ feet (3200 mm) beyond the centerline of the burner at any time during the extended 30-minute test."

The proponent has previously stated that he and others did not understood what was meant by “Significant progressive combustion” even though it’s been in the code for 55 years.

“Combustion” is not defined in the IBC, the IFC or the IRC, but we all understand what constitutes combustion.

“Progressive” is what it means, combustion that progresses or continues following the 30-minute test.

“Significant”, while subjective, appears 9 times in the IRC, 8 times in the IBC and 5 times in the IFC.

Keep in mind that ASTM E2768, where this all originated from with the claim that the prescriptive **“showing no evidence of significant progressive combustion”** is equivalent to the additional prescriptive requirement that **“the flame front shall not progress more than 10 ½ feet beyond the centerline of the burners at any time during the test”**, has previously been disapproved in two previous code cycles and was not adopted by the Admin Committee during the current 2019 code cycle.

Overturning this proposal is necessary to be consistent with what the International Residential Code Committee-Building did when it unanimously voted (11-0) to disapprove the proposed deletion of significant progressive combustion (RB255-19) without scientific justification.

Overturning this proposal will maintain the current prescriptive requirements that Fire-retardant-treated wood as well as Ignition-resistant building materials must both show no evidence of significant progressive combustion (IBC 2303.2 and IWUIC 503.2).

Bibliography: No bibliography.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
No change to code.

Public Comment# 1895

S167-19

IBC: 2303.2.3, 2303.3.2.3.1 (New)

Proposed Change as Submitted

Proponents: Marcelo M Hirschler, GBH International, representing GBH International (mmh@gbhint.com)

2018 International Building Code

Revise as follows:

2303.2.3 Testing. For ~~wood products produced by other means during manufacture, other than a pressure process, all sides~~ fire retardant treated wood products the front and back faces 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:

2303.2.3.1 Fire testing of wood structural panels Wood structural panels shall be tested with a ripped or cut longitudinal gap of 1/8 inch (3.2 mm).

Reason: Note that the sections above require that fire retardant treated wood be "impregnated with chemicals" and provide permanent protection. That requirement applies to all FRTW products, whether produced by a pressure process or produced by other means during manufacture. Section 2303.2.2 is also explicit in stating that the use of paints or coatings is not an approved method to comply with this section. This proposal thus eliminates the requirement to test a particular type of fire retardant treated wood on "all sides", since the testing is never actually conducted on all sides (as pointed out often by multiple testifiers in previous code cycles) because all sides really means front and back (you literally cannot test the edges in the ASTM E84 other than by putting multiple edge pieces into the tunnel to make up the 24 feet by 2 feet specimen). In order to test "all sides" of a lumber product it would be necessary to fasten 864 small pieces together to make one specimen, which is not realistic.
The proposed new subsection will add fire safety because it recognizes an issue that was highlighted in the previous code cycle, and was also brought up in committee ASTM E05 and at the IWUIC: wood structural panels are typically installed in the field following industry practice. Industry recommendations for wood structural panels require a gap to accommodate dimensional changes caused by swelling due to changing moisture conditions. Therefore, installation in the field requires cutting and ripping of the panels and this results in the creation of "non-factory edges". Therefore, it is important to test wood structural panels with a rip or gap to ensure that the required fire test results from the charging paragraph are achieved when the interior of the panel is exposed.

Note that the IWUIC requires such a rip or gap for ignition resistant structural panels, and it sends FRTW products to this IBC section.

Cost Impact: The code change proposal will increase the cost of construction
This proposal will add fire safety and will require more testing for wood structural panels. The proposal will also require more testing for other FRTW products manufactured by a pressure process but apparently less testing for FRTW products that are manufactured by other means, except that typically just the front and back faces are tested anyway.

S167-19

Public Hearing Results

Committee Action: As Modified

Committee Modification:
2018 International Building Code

~~**2303.2.3 Testing.** For fire retardant treated wood products the front and back faces of the wood product shall be tested in accordance with and produce the results required in Section 2303.2.~~

~~**2303.2.3.1 Fire testing of wood structural panels.** Wood structural panels shall be tested with a ripped or cut longitudinal gap of 1/8 inch (3.2 mm).~~

Committee Reason: The proposed new subsection will add fire safety because it recognizes an issue that was highlighted in the previous code cycle, and was also brought up in committee ASTM E05 and at the IWUIC: wood structural panels are typically installed in the field following industry practice. The modification deletes unnecessary testing and therefore provides 'a level playing field'. (Vote: 11-3)

Assembly Action: None

Individual Consideration Agenda

Public Comment 1:

IBC®: 2303.2.3,

2303.2.3.1

(New)

Proponents:
Christopher Athari, representing Hoover Treated Wood Products (cathari@frtw.com)

requests As Modified by Public Comment

Further modify as follows:

2018 International Building Code

2303.2.3 Testing. For wood products produced by other means during manufacture, other than a pressure process, all sides of the wood product shall be tested in accordance with and produce the results required in Section 2303.2. Wood structural panels shall be permitted to test only the front and back faces.

2303.2.3.1
Fire testing of wood structural panels. Wood structural panels shall be tested with a ripped or cut longitudinal gap of 1/8".

Commenter's Reason: 2303.2.3 is necessary to ensure that products products products by methods that are not produced by a pressure process are equivalent to materials that are. It is needed to maintain life safety.

The 1/8" gap is representative of how the product is actually installed in the field.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
Returning language to code.

Public Comment# 1689

Public Comment 2:

IBC®: 2303.2.3

Proponents:
Christopher Athari, representing Hoover Treated Wood Products (cathari@frtw.com)

requests As Modified by Public Comment

Further modify as follows:

2018 International Building Code

2303.2.3 Fire Testing Of Wood Structural Panels Wood structural panels shall be tested with a ripped or cut longitudinal gap of 1/8" (3.2 mm).

Commenter's Reason: The 1/8" gap is representative of how the product is installed and used in the field and products should be tested accordingly.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
No cost impact, clarifying test conditions

Public Comment# 1694

Public Comment 3:

Proponents:
David Tyree, representing American Wood Council (dtyree@awc.org)

requests As Modified by Committee

Commenter's Reason: AWC supports the Committee-recommended modification. Section 2303.2 already addresses testing and performance requirements for FRTW produced either by a pressure process or by other means during manufacture, so the testing provisions of Section 2303.2.3 are redundant and unnecessary. Furthermore, the fact that Section 2303.2.3 is applicable only to FRTW produced by other means during manufacture creates a potential for double-standards when compared to the requirements for FRTW produced by a pressure process. Deletion of 2303.2.3 will remove these redundant provisions and help to ensure a 'level playing field' between FRTW product types.
With regards to the proposed new Section 2303.2.3.1, specific provisions regarding testing should be addressed in the applicable consensus-based test standard, rather than in the code.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
Testing provisions already addressed in code and therefore are redundant and unnecessary.

Public Comment# 1680

S174-19

IBC: TABLE 2304.10.1

Proposed Change as Submitted

Proponents: Rick Allen, International Staple, Nail and Tool Association, representing International Staple, Nail and Tool Association (rallen@isanta.org)

2018 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	4-8d box (2½" x 0.113"); or 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 ($2^{1}/_{2}$″ × 0.131″) 2-3″ × 0.131″ nails 2-3″ 14 gage staples	Each end, toenail
	2-16 d common ($3^{1}/_{2}$″ × 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	4-8d box (2½" x 0.113"); or 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 Stainless Steel Fasteners are not applicable in this connection	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 Stainless Steel Fasteners are not applicable in this connection	2 toenails on one side and 1 toenail on opposite side of rafter or truss[c] ~~Toenail[c]~~
7. Roof rafters to ridge valley or hip rafters; or roof rafter to 2-inch ridge beam	2-16d common ($3^{1}/_{2}$″ × 0.162″); or 3-16d box (3½" x 0.135"); 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 ($3^{1}/_{2}$″ × 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
	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 wallcorners (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″) ; or 5-8d box (2½" x 0.113)	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 12-16d box (3½" x 0.135"); 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)

	staples, $^7/_{16}''$ crown	each side of end joint)
14. Bottom plate to joist, rim joist, band joist or blocking (not at braced wall panels)	16d common ($3^1/_2'' \times 0.162''$); or	$16''$ o.c. face nail
	16d box ($3^1/_2'' \times 0.135''$); or $3'' \times 0.131''$ 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 ($3^1/_2'' \times 0.162''$); or 3-16d box ($3^1/_2'' \times 0.135''$); or 4-$3'' \times 0.131''$ nails; or 4-$3''$ 14 gage staples, $^7/_{16}''$ crown	$16''$ o.c. face nail
16. Stud to top or bottom plate	3-16d box (3½" x 0.135"); or 4-8d common ($2^1/_2'' \times 0.131''$); or 4-10d box ($3'' \times 0.128''$); or 4-$3'' \times 0.131''$ nails; or 4-8d box (2½" x 0.113"); or 4-$3''$ 14 gage staples, $^7/_{16}''$ crown; or	Toenail
	2-16d common ($3^1/_2'' \times 0.162''$); or 3-16d box (3½" x 0.135"); or 3-10d box ($3'' \times 0.128''$); or 3-$3'' \times 0.131''$ nails; or 3-$3''$ 14 gage staples, $^7/_{16}''$ crown	End nail
17. Top plates, laps at corners and intersections	2-16d common ($3^1/_2'' \times 0.162''$); or 3-10d box ($3'' \times 0.128''$); or 3-$3''$ $\times 0.131''$ nails; or 3-$3''$ 14 gage staples, $^7/_{16}''$ crown	Face nail
18. $1''$ brace to each stud and plate	3-8d box (2½" x 0.113"); or 2-8d common ($2^1/_2'' \times 0.131''$); or 2-10d box ($3'' \times 0.128''$); or 2-$3'' \times 0.131''$ nails; or 2-$3''$ 14 gage staples, $^7/_{16}''$ crown Stainless Steel Fasteners are not applicable in this connection	Face nail
19. $1'' \times 6''$ sheathing to each bearing	3-8d box (2½" x 0.113"); or 2-8d common ($2^1/_2'' \times 0.131''$); or 2-10d box ($3'' \times 0.128''$) ; or 2-1¾ 16 gage staples, 1" crown Stainless Steel Fasteners are not applicable in this connection	Face nail
20. $1'' \times 8''$ and wider sheathing to each bearing	3-8d common ($2^1/_2'' \times 0.131''$); or 3-8d box (2½" x 0.113"); or ~~3~~ 2-10d box ($3'' \times 0.128''$); or 3-1¾" 16 gage staples, 1" crown Stainless Steel Fasteners are not applicable in this connection	Face nail
	Wider than 1" x 8" 3-8d common (2½" x 0.131"); or 4-8d box (2½" x 0.113"); or 3-10d box (3" x 0.128"); or 4-1¾" 16 gage staples, 1" crown Stainless Steel Fasteners are not applicable in this connection	
Floor		
21. Joist to sill, top plate, or girder	4-8d box (2½" x 0.113"); or 3-8d common ($2^1/_2'' \times 0.131''$); or floor 3-10d box ($3'' \times 0.128''$); or3-$3'' \times 0.131''$ nails; or 3-$3''$ 14 gage staples, $^7/_{16}''$ crown	Toenail
22. Rim joist, band joist, or blocking to top plate, sill or other framing below	4-8d box (2½" x 0.113)	4" o.c. toenail
	8d common ($2^1/_2'' \times 0.131''$); or 10d box ($3'' \times 0.128''$); or $3'' \times 0.131''$ nails; or $3''$ 14 gage staples, $^7/_{16}''$ crown	$6''$ o.c., toenail
23. $1'' \times 6''$ subfloor or less to each joist	3-8d box (2½" x 0.113"); or 2-8d common ($2^1/_2'' \times 0.131''$); or ~~2~~ 3-10d box ($3'' \times 0.128''$) ; or 2-1¾" 16 gage staples 1" crown Stainless Steel Fasteners are not applicable in this connection	Face nail
24. 2 subfloor to joist or girder	3-16d box (3½x 0.135"); or 2-16d common ($3^1/_2'' \times 0.162''$)	Blind and Face nail
25. $2''$ planks (plank & beam – floor & roof)	3-16d box (3½: x 0.135"); or 2-16d common ($3^1/_2'' \times 0.162''$)	Each bearing, face nail
26. Built-up girders and beams, $2''$ lumber layers	20d common ($4'' \times 0.192''$)	$32''$ o.c., face nail at top and bottom staggered on opposite sides
	10d box ($3'' \times 0.128''$); or $3'' \times 0.131''$ 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 ($4'' \times 0.192''$); or 3-10d box ($3'' \times 0.128''$); or 3-$3'' \times 0.131''$ nails; or 3-$3''$ 14 gage staples, $^7/_{16}''$ crown	Ends and at each splice, face nail
	2-16d common ($3^1/_2'' \times 0.162''$); or 4-16d box (3½" x 0.135"); or 4-	

27. Ledger strip supporting joists or rafters	3-16d common ($3^1/_2$″ × 0.162″); or 4-16d box (3½″ x 0.135″); 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	
28. 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	
29. 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	
Wood structural panels (WSP), subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framing[a]			
		Edges (inches)	Intermediate supports (inches)
30. $^3/_8$″ – $^1/_2$″	6d common or deformed (2″ × 0.113″); or $2^3/_8$″ × 0.113″ nail (subfloor and wall)	6	12
	8d common or deformed ($2^1/_2$″ × 0.131″) (roof) or RSRS-01 ~~($2^3/_8$″ × 0.113″) nail (roof)[d]~~	~~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
31. $^{19}/_{32}$″ – $^3/_4$″	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) or RSRS-01 ($2^3/_8$″ × 0.113″) nail (roof)[d]	6	12
	$2^3/_8$″ × 0.113″ nail; or 2″ 16 gage staple, $^7/_{16}$″ crown	4	8
32. $^7/_8$″ – $1^1/_4$″	10d common (3″ × 0.148″); or ~~8d~~ deformed ($2^1/_2$″ × 0.131″)	6	12
Other exterior wall sheathing			
33. $^1/_2$″ fiberboard sheathing[b]	$1^1/_2$″ x 0.120", galvanized roofing nail ($^7/_{16}$″ head diameter); or $1^1/_4$″ 16 gage staple with $^7/_{16}$″ or 1″ crown	3	6
34. $^{25}/_{32}$″ fiberboard sheathing[b]	$1^3/_4$″ x 0.120" 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			
35. $^3/_4$″ and less	8d common ($2^1/_2$″ × 0.131″); or ~~6d~~ deformed (2″ × 0.113″) or deformed 2" x 0.120"	6	12
36. $^7/_8$″ – 1″	8d common ($2^1/_2$″ × 0.131″); or ~~8d~~ deformed ($2^1/_2$″ × 0.131″) ; or deformed 2½" x 0.120"	6	12
37. $1^1/_8$″ – $1^1/_4$″	10d common (3″ × 0.148″); or ~~8d~~ deformed ($2^1/_2$″ × 0.131″) ; or deformed 2½" x 0.120"	6	12
Panel siding to framing			
38. $^1/_2$″ or less	6d corrosion-resistant siding ($1^7/_8$″ × 0.106″); or 6d corrosion-resistant casing (2″ × 0.099″)	6	12
39. $^5/_8$″	8d corrosion-resistant siding ($2^3/_8$″ × 0.128″); or 8d corrosion-resistant casing ($2^1/_2$″ × 0.113″)	6	12
Wood structural panels (WSP), subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framing[a]			
		Edges (inches)	Intermediate supports (inches)
Interior paneling			
40. $^1/_4$″	4d casing ($1^1/_2$″ × 0.080″); or 4d finish ($1^1/_2$″ × 0.072″)	6	12
41. $^3/_8$″	6d casing (2″ × 0.099″); or 6d finish (2" x 0.092") (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: IBC Table 2304.10.1 and IRC Table R602.3(1) are essentially the same table in structural connections 1 through 39. Although the descriptions are closely align, there are fasteners prescribed in the IBC table that are not in the IRC table and fasteners prescribed in the IRC table that are not in the IBC table.
This proposal is written to harmonize the fasteners between the two tables. In addition, where additional information exists in one table and not the other, this too is being harmonized.

For connections # 2,6,18,19, 20 & 23 there was a code change proposal RB272-13 entered in by the American Wood Council and adopted for the 2015 IRC. The reference nail values for the nailing schedule in these connections were based on Reference Lateral Values <u>and</u> Reference Withdrawal values. All other connections in the table were based on Reference Lateral Design Values. In the 2018 NDS, the reference withdrawal values for stainless steel nails were tabulated in a new NDS table (12.2D). The withdrawal values for stainless steel are lower than the values for carbon steel (bright or galvanized) nails of equivalent diameters.

As such, the lower stainless steel withdrawal values combined with the publication date of the 2018 NDS and the 2015 code proposal date would indicate that the basis of the original code proposal is relevant to only carbon steel nails and not to stainless steel nails. The added note to these connections is to exclude stainless steel from these connections based on the lower withdrawal values.

Connection 1:

Added 8d box nails to match IRC R602.3(1)

Connection 2:

Added note regarding stainless steel fasteners

Added 8d box nails from IRC R602.3(1)

Connection 6:

Added note regarding stainless steel fasteners

Changed Fastener Spacing and Location note to match IRC R602.3(1)

Connection 7:

Added 16d Box nails to match IRC R602.3(1)

Connection 11

Added 8d Box nails to match IRC R602.3(1)

Connection 13

Added 16d Box nails to match IRC R602.3(1)

Connection 16

Added 16d Box and 8d Box nails to match IRC R602.3(1)

Connection 18

Added note regarding stainless steel fasteners

Added 8d Box nails to match IRC R602.3(1)

Connection 19

Added note regarding stainless steel fasteners

Added 8d Box nails to match IRC R602.3(1)

Added 16 gage staples to match IRC R602.3(1)

Connection 20

Added note regarding stainless steel fasteners

Added 8d Box nails to match IRC R602.3(1)

Added 16 gage staples to match IRC R602.3(1)

Added subcategory "wider than 1" x 8" to match IRC R602.3(1)

Connection 21

Added 8d Box nails to match IRC R602.3(1)

Connection 22

Added a subcategory of 4' o.c. to match IRC R602.3(1)

Connection 23

Added note regarding stainless steel fasteners

Added 8d Box nails to match IRC R602.3(1)

Added 16 gage staples to match IRC R602.3(1)

Connection 24

Added 16d box nails to match IRC R602.3(1)

Changed Spacing and Location notation to match IRC R602.3(1)

Connection 25

Added 16d box nails to match IRC R602.3(1)

Connection 27

Added 16d box nails to match IRC R602.3(1)

Connection 30:

All 6 and 12 subfloor and wall fasteners were moved into one line

Connection 31:

The description **6d deformed (2" x 0.113")** is an incorrect description. ASTM F1667 does not have a classification for 6d deformed nails. The correct description is <u>deformed (2" x 0.113")</u>

Connection 32:

The description **8d deformed (2" x 0.131")** is an incorrect description. ASTM F1667 does not have a classification for 8d deformed nails. The correct description is <u>deformed (2½" x 0.131")</u>

Connections 33 & 34:

The current nail description is incomplete and is missing a shank diameter. Addition of the diameters match AWC SDPWS

Connections 35:

The description **6d deformed (2" x 0.113")** is an incorrect description. ASTM F1667 does not have a classification for 6d deformed nails. The correct description is deformed (2" x 0.113")

Connection 36 & 37:

The description **8d deformed (2" x 0.131")** is an incorrect description. ASTM F1667 does not have a classification for 8d deformed nails. The correct description is deformed (2½" x 0.131")

Connection 41:

Dimension of a 6d finish nail has been added to be consistent

Cost Impact: The code change proposal will not increase or decrease the cost of construction
listing of additional fasteners should have no effect on cost of construction

S174-19

Public Hearing Results

Committee Action: As Modified

Committee Modification:
2018 International Building Code

TABLE 2304.10.1

FASTENING SCHEDULE

DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER[e]	SPACING AND LOCATION
	Roof	
1. Blocking between ceiling joists, rafters or trusses to top plate or other framing below	4-8d box ($2^1/_2$" x 0.113"; or 3-8d common ($2^1/_2$" x 0.131"); or 3-10d box (3" x 0.128"); or 3-3" x 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 ($2^1/_2$" x 0.131") 2-3" x 0.131" nails 2-3" 14 gage staples	Each end, toenail
	2-16d common ($3^1/_2$" x 0.162") 3-3" x 0.131" nails 3-3" 14 gage staples	End nail
Flat blocking to truss and web filler	16d common ($3^1/_2$" x 0.162") @ 6 o.c. 3" x 0.131" nails @ 6 o.c. 3" 14 gage staples @ 6 o.c	Face nail
2. Ceiling joists to top plate	4-8d box ($2^1/_2$" x 0.113") 3-8d common ($2^1/_2$ x 0.131"); or 3-10d box (3"x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, $^7/_{16}$"crown ~~Stainless Steel Fasteners are no applicable in this connection~~	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 ($3^1/_2$" x 0.162"); or 4-10d box (3" x 0.128"); or 4-3" x 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

DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION
5. Collar tie to rafter	3-10d common (3" x 0.148"); or 4-10d box (3" x 0.128"); or 4-3" x 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" x 0.148"); or 3-16d box ($3^{1}/_{2}$" x 0.135"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, $^{7}/_{16}$" crown ~~Stainless Steel Fasteners are not applicable in this connection~~	2 toenails on one side and 1 toenail on opposite side of rafter or truss[c]
7. Roof rafters to ridge valley or hip rafters; or roof rafter to 2-inch ridge beam	2-16d common ($3^{1}/_{2}$" x 0.162"); or 3-16d box ($3^{1}/_{2}$" x 0.135") or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, $^{7}/_{16}$" crown	End nail
	3-16d common ($3^{1}/_{2}$" x 0.148"); or 4-16d box ($3^{1}/_{2}$" x 0.135"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, $^{7}/_{16}$" crown	Toenail
Wall		
8. Stud to stud (not at braced wall panels)	16d common ($3^{1}/_{2}$" x 0.162");	24 o.c. face nail
	10d box (3" x 0.128"); or 3" x 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 wallcorners (at braced wall panels)	16d common ($3^{1}/_{2}$" x 0.162")~~; or~~	16 o.c. face nail
	16d box ($3^{1}/_{2}$" x 0.135"); or 3" x 0.131" nails; or 3-3" 14 gage staples, $^{7}/_{16}$" crown	12 o.c. face nail
	~~3" x 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 ($3^{1}/_{2}$" x 0.162")~~; or~~	16 o.c. each edge, face nail
	16d box ($3^{1}/_{2}$" x 0.135")	12 o.c. each edge, face nail
11. Continuous header to stud	4-8d common ($2^{1}/_{2}$" x 0.131"); or 4-10d box (3" x 0.128"); or 5-8d box ($2^{1}/_{2}$" x 0.113")	Toenail
12. Top plate to top plate	16d common ($3^{1}/_{2}$" x 0.162")~~; or~~	16 o.c. face nail
	10d box (3" x 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 ($3^{1}/_{2}$" x 0.162"); or 12-16d box $3^{1}/_{2}$" x 0.135"); or 12-10d box (3" x 0.128); or 12-3" x 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 ($3^{1}/_{2}$" x 0.162")~~;or~~	16 o.c. face nail
	16d box ($3^{1}/_{2}$" x 0.135"); or 3" x 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 ($3^{1}/_{2}$" x 0.162"); or 3-16d box ($3^{1}/_{2}$" x 0.135); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, $^{7}/_{16}$" crown	16 o.c. face nail
16. Stud to top or bottom plate	3-16d box ($3^{1}/_{2}$" x 0.135"); or 4-8d common ($2^{1}/_{2}$" x 0.131"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131"; or 4-8d box ($2^{1}/_{2}$" x 0.113" nails; or 4-3" 14 gage staples, $^{7}/_{16}$" crown	Toenail
	2-16d common ($3^{1}/_{2}$" x 0.162"); or 3-16d box $3^{1}/_{2}$" x 0.135"; or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, $^{7}/_{16}$" crown	End nail
17. Top plates, laps at corners and intersections	2-16d common ($3^{1}/_{2}$" x 0.162"); or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, $^{7}/_{16}$" crown	Face nail
18. 1 brace to each stud and plate	3-8d box (2½" x 0.113"); or 2-8d common ($2^{1}/_{2}$" x 0.131"); or 2-10d box (3" x 0.128"); or 2-3" x 0.131" nails; or 2-3" 14 gage staples, $^{7}/_{16}$" crown ~~Stainless Steel Fasteners are not applicable in this connection~~	Face nail
19. 1 6 sheathing to each bearing	3-8d box (2½" x 0.113"; or 2-8d common ($2^{1}/_{2}$" x 0.131"); or 2-10d box (3" x 0.128"); or 2-1¾" 16 gage staples, 1" crown ~~Stainless Steel Fasteners are not applicable in this connection~~	Face nail

<table>
<tr><th>DESCRIPTION OF BUILDING ELEMENTS</th><th>NUMBER AND TYPE OF FASTENER</th><th colspan="2">SPACING AND LOCATION</th></tr>
<tr><td rowspan="2">20. 1 8 and wider sheathing to each bearing</td><td>3-8d common ($2^{1}/_{2}$"x 0.131"); or 3-8d box (2½" x 0.113"); or ~~3~~ 2-10d box (3" x 0.128") or 3-1¾" 16 gage staples, 1" crown

~~Stainless Steel Fasteners are not appliable in this connection~~</td><td colspan="2" rowspan="2">Face nail</td></tr>
<tr><td>Wider than 1" x 8"

3-8d common (2½" x 0.131"); or 4-8d box (2½" x 0.113"); or 3-10d box (3" x 0.128"); or 4-1¾" 16 gage staples, 1" crown

~~Stainless Steel Fasteners are not appliable in this connection~~</td></tr>
<tr><th colspan="4">Floor</th></tr>
<tr><td>21. Joist to sill, top plate, or girder</td><td>4 8d box (2½" x 0.113");or 3-8d common ($2^{1}/_{2}$" x 0.131"); or floor 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, $^{7}/_{16}$" crown</td><td colspan="2">Toenail</td></tr>
<tr><td rowspan="2">22. Rim joist, band joist, or blocking to top plate,sill or other framing below</td><td>4-8d box (2½" x 0.113")</td><td colspan="2">4" o.c. toenail</td></tr>
<tr><td>8d common ($2^{1}/_{2}$" x 0.131); or 10d box (3" x 0.128"); or 3" x 0.131" nails; or 3" 14 gage staples, $^{7}/_{16}$" crown</td><td colspan="2">6 o.c., toenail</td></tr>
<tr><td>23. 1 6 subfloor or less to each joist</td><td>3-8d box (2½" x 0.113") or 2-8d common (2½" x 0.131); or 3-10d box (3" x 0.128"); or 2-1¾" 16 gage staples 1" crown

~~Stainless Steel Fasteners are not applicable in this connection~~</td><td colspan="2">Face nail</td></tr>
<tr><td>24. 2 subfloor to joist or girder</td><td>3-16d box (3½" x 0.135"); or 2-16d common ($3^{1}/_{2}$" x 0.162")</td><td colspan="2">Blind and Face nail</td></tr>
<tr><td>25. 2 planks (plank & beam floor & roof)</td><td>3-16d box (3½" x 0.135"); or 2-16d common ($3^{1}/_{2}$" x 0.162")</td><td colspan="2">Each bearing, face nail</td></tr>
<tr><td rowspan="3">26. Built-up girders and beams, 2 lumber layers</td><td>20d common (4" x 0.192")</td><td colspan="2">32 o.c., face nail at top and bottom staggered on opposite sides</td></tr>
<tr><td>10d box (3" x 0.128"); or 3" x 0.131" nails; or 3" 14 gage staples, $^{7}/_{16}$" crown</td><td colspan="2">24 o.c. face nail at top and bottom staggered on opposite sides</td></tr>
<tr><td>And: 2-20d common (4" x 0.192"); or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, $^{7}/_{16}$" crown</td><td colspan="2">Ends and at each splice, face nail</td></tr>
<tr><td>27. Ledger strip supporting joists or rafters</td><td>3-16d common ($3^{1}/_{2}$" x 0.162"); or 4-16d box (3½" x 0.135"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, $^{7}/_{16}$" crown</td><td colspan="2">Each joist or rafter, face nail</td></tr>
<tr><td>28. Joist to band joist or rim joist</td><td>3-16d common ($3^{1}/_{2}$" x 0.162"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, $^{7}/_{16}$" crown</td><td colspan="2">End nail</td></tr>
<tr><td>29. Bridging or blocking to joist, rafter or truss</td><td>2-8d common ($2^{1}/_{2}$ " x 0.131"); or 2-10d box (3" x 0.128"); or 2-3" x 0.131" nails; or 2-3" 14 gage staples, $^{7}/_{16}$" crown</td><td colspan="2">Each end, toenail</td></tr>
<tr><th colspan="4">Wood structural panels (WSP), subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framing[a]</th></tr>
<tr><td></td><td></td><th>Edges (inches)</th><th>Intermediate supports (inches)</th></tr>
<tr><td rowspan="5">30. $^{3}/_{8}{}^{1}/_{2}$</td><td>6d common or deformed (2" x 0.113"); or $2^{3}/_{8}$" x 0.113" (subfloor and wall)</td><td>6</td><td>12</td></tr>
<tr><td>8d common or deformed ($2^{1}/_{2}$ x 0.131" x 0.281" head) (roof) or RSRS-01 ($2^{3}/_{8}$ " x 0.113") nail (roof)[d]</td><td></td><td></td></tr>
<tr><td>$1^{3}/_{4}$" 16 gage staple, $^{7}/_{16}$" crown (subfloor and wall)</td><td>4</td><td>8</td></tr>
<tr><td>$2^{3}/_{8}$" x 0.113" x 0.266" head nail (roof)</td><td>4</td><td>8</td></tr>
<tr><td>$1^{3}/_{4}$" 16 gage staple, $^{7}/_{16}$" crown (roof)</td><td>3</td><td>6</td></tr>
<tr><td rowspan="2"></td><td>8d common ($2^{1}/_{2}$" x 0.131"); or deformed (2" x 0.113")(subfloor and wall)</td><td>6</td><td>12</td></tr>
<tr><td></td><td></td><td></td></tr>
</table>

31. $^{19}/_{32}$ $^{3}/_{4}$ DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION	
	8d common or deformed ($2^{1}/_{2}$" x 0.131" x 0.281" head) (roof) or RSRS-01 ($2^{3}/_{8}$" x 0.113") nail (roof)[d]	6	12
	$2^{3}/_{8}$ " x 0.113" x 0.266" head nail; or 2" 16 gage staple, $^{7}/_{16}$" crown	4	8
32. $^{7}/_{8}$ $1^{1}/_{4}$	10d common (3" x 0.148"); or deformed ($2^{1}/_{2}$" x 0.131" x 0.281" head)	6	12
Other exterior wall sheathing			
33. $^{1}/_{2}$ fiberboard sheathing[b]	$1^{1}/_{2}$ x 0.120" galvanized roofing nail ($^{7}/_{16}$" head diameter); or $1^{1}/_{4}$" 16 gage staple with $^{7}/_{16}$" or 1" crown	3	6
34. $^{25}/_{32}$ fiberboard sheathing[b]	$1^{3}/_{4}$" x 0.120" 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			
35. $^{3}/_{4}$ and less	8d common ($2^{1}/_{2}$" x 0.131"); or deformed (2" x 0.113"); or deformed 2" x 0.120"	6	12
36. $^{7}/_{8}$ 1	8d common ($2^{1}/_{2}$ 0.131); or deformed ($2^{1}/_{2}$" x 0.131")	6	12
37. $1^{1}/_{8}$ $1^{1}/_{4}$	10d common (3" x 0.148"); or deformed ($2^{1}/_{2}$ " x 0.131"); or deformed 2½" x 0.120"	6	12
Panel siding to framing			
38. $^{1}/_{2}$ or less	6d corrosion-resistant siding ($1^{7}/_{8}$" x 0.106"); or 6d corrosion-resistant casing (2" x 0.099")	6	12
39. $^{5}/_{8}$	8d corrosion-resistant siding ($2^{3}/_{8}$ " x 0.128"); or 8d corrosion-resistant casing ($2^{1}/_{2}$ x 0.113")	6	12
Wood structural panels (WSP), subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framing[a]			
		Edges (inches)	**Intermediate supports (inches)**
Interior paneling			
40. $^{1}/_{4}$	4d casing ($1^{1}/_{2}$" x 0.080); or 4d finish ($1^{1}/_{2}$ " x 0.072")	6	12
41. $^{3}/_{8}$	6d casing (2" x 0.099"); or 6d finish 2" x 0.092" (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.

e. Nails and staples are carbon steel meeting the specifications of ASTM F1667

Committee Reason: This proposal harmonizes the IBC table with the IRC table. The modification provided coordination with the latest NDS standard (especially for stainless steel fasteners) (Vote: 14-0)

Assembly Action: None

S174-19

Individual Consideration Agenda

Public Comment 1:

IBC®: TABLE 2304.10.1 (New)

Proponents:
Kelly Cobeen, representing Federal Emergency Management Agency and Applied Technology Council Seismic Code Support Committee (FEMA/ATC SCSC) (kcobeen@wje.com); Michael Mahoney, Federal Emergency Management Agency, representing Federal Emergency Management Agency (mike.mahoney@fema.dhs.gov)

requests As Modified by Public Comment

Further modify as follows:

2018 International Building Code

TABLE 2304.10.1
FASTENING SCHEDULE

DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER[e]	SPACING AND LOCATION
	Roof	
1. Blocking between ceiling joists, rafters or trusses to top plate or other framing below	4-8d box (2½" x 0.113"); or 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 ($2^{1}/_{2}$″ × 0.131″) 2-3″ × 0.131″ nails 2-3″ 14 gage staples	Each end, toenail
	2-16 d common ($3^{1}/_{2}$″ × 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	4-8d box (2½" x 0.113"); or 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	2 toenails on one side and 1 toenail on opposite side of rafter or truss[c]
7. Roof rafters to ridge valley or hip rafters; or roof rafter to 2-inch ridge beam	2-16d common ($3^{1}/_{2}$″ × 0.162″); or 3-16d box (3½" x 0.135"); 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 ($3^{1}/_{2}$″ × 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
	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 wallcorners (at braced wall panels)	16d common ($3^{1}/_{2}$″ × 0.162″)	16″ o.c. face nail
	16d box ($3^{1}/_{2}$″ × 0.135″); or	12″ o.c. face nail or 3″ × 0.131″ nails; or 3-3″ 14 gage staples, ${}^{7}/_{16}$″ crown
10. Built-up header (2″ to 2″ header)	16d common ($3^{1}/_{2}$″ × 0.162″)	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″); or 5-8d box (2½" x 0.113)	Toenail
12. Top plate to top plate	16d common ($3^{1}/_{2}$″ × 0.162″)	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 12-16d box (3½" x 0.135"); 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″)	16″ o.c. face nail
	16d box ($3^{1}/_{2}$″ × 0.135″); or 3″ × 0.131″ nails; or 3″ 14 gage staples,	12″ o.c. face nail

	$^{7}/_{16}$″ crown		
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	3-16d box (3½" x 0.135"); or 4-8d common ($2^{1}/_{2}$″ × 0.131″); or 4-10d box (3″ × 0.128″); or 4-3″ × 0.131″ nails; or 4-8d box (2½" x 0.113"); or 4-3″ 14 gage staples, $^{7}/_{16}$″ crown; or	Toenail	
	2-16d common ($3^{1}/_{2}$″ × 0.162″); or 3-16d box (3½" x 0.135"); or 3-10d box (3″ × 0.128″); or 3-3″ × 0.131″ nails; or 3-3″ 14 gage staples, $^{7}/_{16}$″ crown	End nail	
17. 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	
18. 1″ brace to each stud and plate	3-8d box (2½" x 0.113"); or 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	
19. 1″ × 6″ sheathing to each bearing	3-8d box (2½" x 0.113"); or 2-8d common ($2^{1}/_{2}$″ × 0.131″); or 2-10d box (3″ × 0.128″); or 2-1¾ 16 gage staples, 1" crown	Face nail	
20. 1″ × 8″ and wider sheathing to each bearing	3-8d common ($2^{1}/_{2}$″ × 0.131″); or 3-8d box (2½" x 0.113"); or 2-10d box (3″ × 0.128″); or 3-1¾" 16 gage staples, 1" crown	Face nail	
	Wider than 1" x 8" 3-8d common (2½" x 0.131"); or 4-8d box (2½" x 0.113"); or 3-10d box (3" x 0.128"); or 4-1¾" 16 gage staples, 1" crown Stainless Steel Fasteners are not applicable in this connection		
Floor			
21. Joist to sill, top plate, or girder	4-8d box (2½" x 0.113"); or 3-8d common ($2^{1}/_{2}$″ × 0.131″); or floor 3-10d box (3″ × 0.128″); or3-3″ × 0.131″ nails; or 3-3″ 14 gage staples, $^{7}/_{16}$″ crown	Toenail	
22. Rim joist, band joist, or blocking to top plate, sill or other framing below	4-8d box (2½" x 0.113)	4" o.c. toenail	
	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	
23. 1″ × 6″ subfloor or less to each joist	3-8d box (2½" x 0.113"); or 2-8d common ($2^{1}/_{2}$″ × 0.131″); or 3-10d box (3″ × 0.128″); or 2-1¾" 16 gage staples 1" crown	Face nail	
24. 2 subfloor to joist or girder	3-16d box (3½x 0.135"); or 2-16d common ($3^{1}/_{2}$″ × 0.162″)	Blind and Face nail	
25. 2″ planks (plank & beam – floor & roof)	3-16d box (3½: x 0.135"); or 2-16d common ($3^{1}/_{2}$″ × 0.162″)	Each bearing, face nail	
26. 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 bot-tom 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	
27. Ledger strip supporting joists or rafters	3-16d common ($3^{1}/_{2}$″ × 0.162″); or 4-16d box (3½" x 0.135"); 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	
28. 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	
29. 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	
Wood structural panels (WSP), subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framing[a]			
		Edges (inches)	Intermediate supports (inches)
	6d common or deformed (2″ × 0.113″); or $2^{3}/_{8}$″ × 0.113″ nail	6	12

	(subfloor and wall)	6	12
30. $^3/_8$″ – $^1/_2$″	8d common or deformed ($2^1/_2$″ × 0.131″) (roof) or RSRS-01		
	$1^3/_4$″ 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
31. $^{19}/_{32}$″ – $^3/_4$″	8d common ($2^1/_2$″ × 0.131″); or deformed (2″ × 0.113″) (subfloor and wall)	6	12
	8d common or deformed (21/2" x 0.131" x 0.281" head) (roof) or RSRS-01 (23/8" x 0.113") nail (roof)d	6	12
	23/8 " x 0.113"x 0.266" head nail; or 2" 16 gage staple, 7/16" crown	4	8
32. $^7/_8$″ – $1^1/_4$″	10d common (3" x 0.148"); or deformed (21/2" x 0.131" x 0.281" head)	6	12
Other exterior wall sheathing			
33. $^1/_2$″ fiberboard sheathing[b]	$1^1/_2$″ x 0.120", galvanized roofing nail ($^7/_{16}$″ head diameter); or $1^1/_4$″ 16 gage staple with $^7/_{16}$″ or 1″ crown	3	6
34. $^{25}/_{32}$″ fiberboard sheathing[b]	$1^3/_4$″ x 0.120" 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			
35. $^3/_4$″ and less	8d common ($2^1/_2$″ × 0.131″); or deformed (2″ × 0.113″) or deformed 2" x 0.120"	6	12
36. $^7/_8$″ – 1″	8d common ($2^1/_2$″ × 0.131″); or deformed ($2^1/_2$″ × 0.131″); or deformed 2½" x 0.120"	6	12
37. $1^1/_8$″ – $1^1/_4$″	10d common (3″ × 0.148″); or deformed ($2^1/_2$″ × 0.131″); or deformed 2½" x 0.120"	6	12
Panel siding to framing			
38. $^1/_2$″ or less	6d corrosion-resistant siding ($1^7/_8$″ × 0.106″); or 6d corrosion-resistant casing (2″ × 0.099″)	6	12
39. $^5/_8$″	8d corrosion-resistant siding ($2^3/_8$″ × 0.128″); or 8d corrosion-resistant casing ($2^1/_2$″ × 0.113″)	6	12
Wood structural panels (WSP), subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framing[a]			
		Edges (inches)	Intermediate supports (inches)
Interior paneling			
40. $^1/_4$″	4d casing ($1^1/_2$″ × 0.080″); or 4d finish ($1^1/_2$″ × 0.072″)	6	12
41. $^3/_8$″	6d casing (2″ × 0.099″); or 6d finish (2" x 0.092") (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.

e. Nails and staples are carbon steel meeting the specifications of ASTM F1667. Connections using nails and staples of other materials or dimensions, such as stainless steel, shall be designed by accepted engineering practice or approved under Section 104.11.

Commenter's Reason: This public comment adds language to new footnote e in order to further clarify that the code change proposal, as approved at the CAH, deletes stainless steel nails and staples from this table. The added language is proposed because it is feared that users of the code will easily miss this change, and not necessarily understand that stainless steel is not carbon steel.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
The public comment provides clarification of provisions only.

Public Comment# 1807

Public Comment 2:

IBC®: TABLE 2304.10.1 (New)

Proponents:
J Daniel Dolan, representing Federal Emergency Management Agency/ Applied Technology Council Seismic Codes Support Committee (jddolan@wsu.edu)

requests As Modified by Public Comment

Further modify as follows:

2018 International Building Code

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	4-8d box (2½" x 0.113"); or 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 ($2^{1}/_{2}$″ × 0.131″) 2-3″ × 0.131″ nails 2-3″ 14 gage staples	Each end, toenail
	2-16 d common ($3^{1}/_{2}$″ × 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	4-8d box (2½" x 0.113"); or 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	2 toenails on one side and 1 toenail on opposite side of rafter or truss[c]
7. Roof rafters to ridge valley or hip rafters; or roof rafter to 2-inch ridge beam	2-16d common ($3^{1}/_{2}$″ × 0.162″); or 3-16d box (3½" x 0.135"); 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 ($3^{1}/_{2}$″ × 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
	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 wallcorners (at braced wall panels)	16d common ($3^{1}/_{2}$″ × 0.162″)	16″ o.c. face nail
	16d box ($3^{1}/_{2}$″ × 0.135″); or	12″ o.c. face nail or 3″ × 0.131″ nails; or 3-3″ 14 gage staples, $^{7}/_{16}$″ crown
10. Built-up header (2″ to 2″ header)	16d common ($3^{1}/_{2}$″ × 0.162″)	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″); or 5-8d box (2½" x 0.113)	Toenail
12. Top plate to top plate	16d common ($3^{1}/_{2}$″ × 0.162″)	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 12-16d box (3½" x 0.135"); 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″)	16″ o.c. face nail
	16d box ($3^{1}/_{2}$″ × 0.135″); or 3″ × 0.131″ nails; or 3″ 14 gage staples,	12″ o.c. face nail

	$^{7}/_{16}$″ crown		
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	3-16d box (3½" x 0.135"); or 4-8d common ($2^{1}/_{2}$″ × 0.131″); or 4-10d box (3″ × 0.128″); or 4-3″ × 0.131″ nails; or 4-8d box (2½" x 0.113"); or 4-3″ 14 gage staples, $^{7}/_{16}$″ crown; or	Toenail	
	2-16d common ($3^{1}/_{2}$″ × 0.162″); or 3-16d box (3½" x 0.135"); or 3-10d box (3″ × 0.128″); or 3-3″ × 0.131″ nails; or 3-3″ 14 gage staples, $^{7}/_{16}$″ crown	End nail	
17. 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	
18. 1″ brace to each stud and plate	3-8d box (2½" x 0.113"); or 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	
19. 1″ × 6″ sheathing to each bearing	3-8d box (2½" x 0.113"); or 2-8d common ($2^{1}/_{2}$″ × 0.131″); or 2-10d box (3″ × 0.128″); or 2-1¾ 16 gage staples, 1" crown	Face nail	
20. 1″ × 8″ and wider sheathing to each bearing	3-8d common ($2^{1}/_{2}$″ × 0.131″); or 3-8d box (2½" x 0.113"); or 2-10d box (3″ × 0.128″); or 3-1¾" 16 gage staples, 1" crown	Face nail	
	Wider than 1" x 8" 3-8d common (2½" x 0.131"); or 4-8d box (2½" x 0.113"); or 3-10d box (3" x 0.128"); or 4-1¾" 16 gage staples, 1" crown Stainless Steel Fasteners are not applicable in this connection		
Floor			
21. Joist to sill, top plate, or girder	4-8d box (2½" x 0.113"); or 3-8d common ($2^{1}/_{2}$″ × 0.131″); or floor 3-10d box (3″ × 0.128″); or3-3″ × 0.131″ nails; or 3-3″ 14 gage staples, $^{7}/_{16}$″ crown	Toenail	
22. Rim joist, band joist, or blocking to top plate, sill or other framing below	4-8d box (2½" x 0.113)	4" o.c. toenail	
	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	
23. 1″ × 6″ subfloor or less to each joist	3-8d box (2½" x 0.113"); or 2-8d common ($2^{1}/_{2}$″ × 0.131″); or 3-10d box (3″ × 0.128″); or 2-1¾" 16 gage staples 1" crown	Face nail	
24. 2 subfloor to joist or girder	3-16d box (3½x 0.135"); or 2-16d common ($3^{1}/_{2}$″ × 0.162″)	Blind and Face nail	
25. 2″ planks (plank & beam – floor & roof)	3-16d box (3½: x 0.135"); or 2-16d common ($3^{1}/_{2}$″ × 0.162″)	Each bearing, face nail	
26. 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	
27. Ledger strip supporting joists or rafters	3-16d common ($3^{1}/_{2}$″ × 0.162″); or 4-16d box (3½" x 0.135"); 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	
28. 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	
29. 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	
Wood structural panels (WSP), subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framing[a]			
		Edges (inches)	Intermediate supports (inches)
	6d common or deformed (2″ × 0.113″); or $2^{3}/_{8}$″ × 0.113″ nail	6	12

30. $^{3}/_{8}$″ – $^{1}/_{2}$″	(subfloor and wall)	6	12
	8d common or deformed ($2^{1}/_{2}$″ × 0.131″) (roof) or RSRS-01		
	$1^{3}/_{4}$″ 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
31. $^{19}/_{32}$″ – $^{3}/_{4}$″	8d common ($2^{1}/_{2}$″ × 0.131″); or deformed (2″ × 0.113″) (subfloor and wall)	6	12
	8d common or deformed (21/2" x 0.131" x 0.281" head) (roof) or RSRS-01 (23/8" x 0.113") nail (roof)d	6	12
	23/8 " x 0.113"x 0.266" head nail; or 2" 16 gage staple, 7/16" crown	4	8
32. $^{7}/_{8}$″ – $1^{1}/_{4}$″	10d common (3" x 0.148"); or deformed (21/2" x 0.131" x 0.281" head)	6	12
Other exterior wall sheathing			
33. $^{1}/_{2}$″ fiberboard sheathing[b]	$1^{1}/_{2}$″ x 0.120", galvanized roofing nail ($^{7}/_{16}$″ head diameter); or $1^{1}/_{4}$″ 16 gage staple with $^{7}/_{16}$″ or 1″ crown	3	6
34. $^{25}/_{32}$″ fiberboard sheathing[b]	$1^{3}/_{4}$″ x 0.120" 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			
35. $^{3}/_{4}$″ and less	8d common ($2^{1}/_{2}$″ × 0.131″); or deformed (2″ × 0.113″) or deformed 2" x 0.120"	6	12
36. $^{7}/_{8}$″ – 1″	8d common ($2^{1}/_{2}$″ × 0.131″); or deformed ($2^{1}/_{2}$″ × 0.131″); or deformed 2½" x 0.120"	6	12
37. $1^{1}/_{8}$″ – $1^{1}/_{4}$″	10d common (3″ × 0.148″); or deformed ($2^{1}/_{2}$″ × 0.131″); or deformed 2½" x 0.120"	6	12
Panel siding to framing			
38. $^{1}/_{2}$″ or less	6d corrosion-resistant siding ($1^{7}/_{8}$″ × 0.106″); or 6d corrosion-resistant casing (2″ × 0.099″)	6	12
39. $^{5}/_{8}$″	8d corrosion-resistant siding ($2^{3}/_{8}$″ × 0.128″); or 8d corrosion-resistant casing ($2^{1}/_{2}$″ × 0.113″)	6	12
Wood structural panels (WSP), subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framing[a]			
		Edges (inches)	Intermediate supports (inches)
Interior paneling			
40. $^{1}/_{4}$″	4d casing ($1^{1}/_{2}$″ × 0.080″); or 4d finish ($1^{1}/_{2}$″ × 0.072″)	6	12
41. $^{3}/_{8}$″	6d casing (2″ × 0.099″); or 6d finish (2" x 0.092") (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.

e. Nails and staples are carbon steel meeting the specifications of ASTM F1667. Connections using nails and staples of other materials, such as stainless steel, shall be designed by acceptable engineering practice or approved under Section 104.11.

Commenter's Reason: This public comment adds language to a new footnote e in order to clarify that the code change proposal, as approved at the CAH, deleted stainless steel nails and staples from this table. The added language is proposed because it is feared that users of the code will easily miss this change, and not necessarily understand that stainless steel in not carbon steel.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
The proposed comment does not add additions requirements, but rather clarifies that stainless steel (and other materials) required different considerations due to the differences in strength and withdrawal characteristics. Since stainless steel is being deleted in the approval in the CAH, this comment only provides clarification and does not add any cost effects to what is already accepted.

Public Comment# 1804

S187-19

IBC®: TABLE 2308.7.3.1

Proposed Change as Submitted

Proponents: Dennis Richardson, American Wood Council, representing American Wood Council (drichardson@awc.org); Philip Line (pline@awc.org)

2018 International Building Code

Revise as follows:

TABLE 2308.7.3.1
RAFTER TIE CONNECTIONS [~~g~~] [i]

~~RAFTER SLOPE~~	~~TIE SPACING (inches)~~	~~NO SNOW LOAD~~				~~GROUND SNOW LOAD (pound per square foot)~~							
						~~30 pounds per square foot~~				~~50 pounds per square foot~~			
		~~Roof span (feet)~~											
		~~12~~	~~20~~	~~28~~	~~36~~	~~12~~	~~20~~	~~28~~	~~36~~	~~12~~	~~20~~	~~28~~	~~36~~
		~~Required number of 16d common ($3^{1}/_{2}$" x 0.162") nails[a, b] per connection[c, d, e, f]~~											
~~3:12~~	~~12~~	~~4~~	~~6~~	~~8~~	~~10~~	~~4~~	~~6~~	~~8~~	~~11~~	~~5~~	~~8~~	~~12~~	~~15~~
	~~16~~	~~5~~	~~7~~	~~10~~	~~13~~	~~5~~	~~8~~	~~11~~	~~14~~	~~6~~	~~11~~	~~15~~	~~20~~
	~~24~~	~~7~~	~~11~~	~~15~~	~~19~~	~~7~~	~~11~~	~~16~~	~~21~~	~~9~~	~~16~~	~~23~~	~~30~~
	~~32~~	~~10~~	~~14~~	~~19~~	~~25~~	~~10~~	~~16~~	~~22~~	~~28~~	~~12~~	~~27~~	~~30~~	~~40~~
	~~48~~	~~14~~	~~21~~	~~29~~	~~37~~	~~14~~	~~32~~	~~36~~	~~42~~	~~18~~	~~32~~	~~46~~	~~60~~
~~4:12~~	~~12~~	~~3~~	~~4~~	~~5~~	~~6~~	~~3~~	~~5~~	~~6~~	~~8~~	~~4~~	~~6~~	~~9~~	~~11~~
	~~16~~	~~3~~	~~5~~	~~7~~	~~8~~	~~4~~	~~6~~	~~8~~	~~11~~	~~5~~	~~8~~	~~12~~	~~15~~
	~~24~~	~~4~~	~~7~~	~~10~~	~~12~~	~~5~~	~~9~~	~~12~~	~~16~~	~~7~~	~~12~~	~~17~~	~~22~~
	~~32~~	~~6~~	~~9~~	~~13~~	~~16~~	~~8~~	~~12~~	~~16~~	~~22~~	~~10~~	~~16~~	~~24~~	~~30~~
	~~48~~	~~8~~	~~14~~	~~19~~	~~24~~	~~10~~	~~18~~	~~24~~	~~32~~	~~14~~	~~24~~	~~34~~	~~44~~
~~5:12~~	~~12~~	~~3~~	~~3~~	~~4~~	~~5~~	~~3~~	~~4~~	~~5~~	~~7~~	~~3~~	~~5~~	~~7~~	~~9~~
	~~16~~	~~3~~	~~4~~	~~5~~	~~7~~	~~3~~	~~5~~	~~7~~	~~9~~	~~4~~	~~7~~	~~9~~	~~12~~
	~~24~~	~~4~~	~~6~~	~~8~~	~~10~~	~~4~~	~~7~~	~~10~~	~~13~~	~~6~~	~~10~~	~~14~~	~~18~~
	~~32~~	~~5~~	~~8~~	~~10~~	~~13~~	~~6~~	~~10~~	~~14~~	~~18~~	~~8~~	~~14~~	~~18~~	~~24~~
	~~48~~	~~7~~	~~11~~	~~15~~	~~20~~	~~8~~	~~14~~	~~20~~	~~26~~	~~12~~	~~20~~	~~28~~	~~36~~
~~7:12~~	~~12~~	~~3~~	~~3~~	~~3~~	~~4~~	~~3~~	~~3~~	~~4~~	~~5~~	~~3~~	~~4~~	~~5~~	~~7~~
	~~16~~	~~3~~	~~3~~	~~4~~	~~5~~	~~3~~	~~4~~	~~5~~	~~6~~	~~3~~	~~5~~	~~7~~	~~9~~
	~~24~~	~~3~~	~~4~~	~~6~~	~~7~~	~~3~~	~~5~~	~~7~~	~~9~~	~~4~~	~~7~~	~~10~~	~~13~~
	~~32~~	~~4~~	~~6~~	~~8~~	~~10~~	~~4~~	~~8~~	~~10~~	~~12~~	~~6~~	~~10~~	~~14~~	~~18~~
	~~48~~	~~5~~	~~8~~	~~11~~	~~14~~	~~6~~	~~10~~	~~14~~	~~18~~	~~9~~	~~14~~	~~20~~	~~26~~
~~9:12~~	~~12~~	~~3~~	~~3~~	~~3~~	~~3~~	~~3~~	~~3~~	~~3~~	~~4~~	~~3~~	~~3~~	~~4~~	~~5~~
	~~16~~	~~3~~	~~3~~	~~3~~	~~4~~	~~3~~	~~3~~	~~4~~	~~5~~	~~3~~	~~4~~	~~5~~	~~7~~
	~~24~~	~~3~~	~~3~~	~~5~~	~~6~~	~~3~~	~~4~~	~~6~~	~~7~~	~~3~~	~~6~~	~~8~~	~~10~~
	~~32~~	~~3~~	~~4~~	~~6~~	~~8~~	~~4~~	~~6~~	~~8~~	~~10~~	~~5~~	~~8~~	~~10~~	~~14~~
	~~48~~	~~4~~	~~6~~	~~9~~	~~11~~	~~5~~	~~8~~	~~12~~	~~14~~	~~7~~	~~12~~	~~16~~	~~20~~
~~12:12~~	~~12~~	~~3~~	~~3~~	~~3~~	~~3~~	~~3~~	~~3~~	~~3~~	~~3~~	~~3~~	~~3~~	~~3~~	~~4~~
	~~16~~	~~3~~	~~3~~	~~3~~	~~3~~	~~3~~	~~3~~	~~3~~	~~4~~	~~3~~	~~3~~	~~4~~	~~5~~
	~~24~~	~~3~~	~~3~~	~~3~~	~~4~~	~~3~~	~~3~~	~~4~~	~~6~~	~~3~~	~~4~~	~~6~~	~~8~~
	~~32~~	~~3~~	~~3~~	~~4~~	~~5~~	~~3~~	~~5~~	~~6~~	~~8~~	~~4~~	~~6~~	~~8~~	~~10~~
	~~48~~	~~3~~	~~4~~	~~6~~	~~7~~	~~4~~	~~7~~	~~8~~	~~12~~	~~6~~	~~8~~	~~12~~	~~16~~

		NO SNOW LOAD[g]			GROUND SNOW LOAD (pound per square foot)					
					30 pounds per square foot			50 pounds per square foot		
		Roof span (feet)								
RAFTER SLOPE	TIE SPACING (inches)	12	24	36	12	24	36	12	24	36
		Required number of 16d common (3-1/2" x 0.162") nails per connection[a, b, c, d, e, f, h]								
	12	3	5	8	3	6	9	5	9	13
	16	4	7	10	4	8	12	6	12	17
	19.2	4	8	12	5	10	14	7	14	21
3:12	24	5	10	15	6	12	18	9	17	26
	32	7	13	20	8	16	24	12	23	34
	48	10	20	29	12	24	35	17	34	51
	12	3	4	6	3	5	7	4	7	10
	16	3	5	8	3	6	9	5	9	13
	19.2	3	6	9	4	7	11	6	11	16
4:12	24	4	8	11	5	9	13	7	13	19
	32	5	10	15	6	12	18	9	17	26
	48	8	15	22	9	18	26	13	26	38
	12	3	3	5	3	4	6	3	6	8
	16	3	4	6	3	5	7	4	7	11
	19.2	3	5	7	3	6	9	5	9	13
5:12	24	3	6	9	4	7	11	6	11	16
	32	4	8	12	5	10	14	7	14	21
	48	6	12	18	7	14	21	11	21	31
	12	3	3	4	3	3	4	3	4	6
	16	3	3	5	3	4	5	3	5	8
	19.2	3	4	5	3	4	6	3	6	9
7:12	24	3	5	7	3	5	8	4	8	11
	32	3	6	9	4	7	10	5	10	15
	48	5	9	13	5	10	15	8	15	22
	12	3	3	3	3	3	3	3	3	5
	16	3	3	4	3	3	4	3	4	6
	19.2	3	3	4	3	4	5	3	5	7
9:12	24	3	4	5	3	4	6	3	6	9
	32	3	5	7	3	6	8	4	8	12
	48	4	7	10	4	8	12	6	12	17
	12	3	3	3	3	3	3	3	3	4
	16	3	3	3	3	3	3	3	3	5
	19.2	3	3	3	3	3	4	3	4	6
12:12	24	3	3	4	3	3	5	3	5	7
	32	3	4	5	3	4	6	3	6	9
	48	3	5	8	3	6	9	5	9	13

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot = 47.8 N/m^2.

a. ~~40d box (5″ × 0.162″) or 16d sinker (3$^1/_4$″ × 0.148″) nails are~~ 10d common (3" x 0.148") nails shall be permitted to be substituted for 16d common (3$^1/_2$″ × ~~0.16″) nails.~~ 0.162″) nails where the required number of nails is taken as 1.2 times the required number of 16d common nails.

~~b. Nailing requirements are permitted to be reduced 25 percent if nails are clinche~~

~~c~~ b. Rafter tie heel joint connections are not required where the ridge is supported by a load-bearing wall, header or ridge beam.

~~d~~ c. Where intermediate support of the rafter is provided by vertical struts or purlins to a load-bearing wall, the tabulated heel joint connection requirements are permitted to be reduced proportionally to the reduction in span.

~~e~~ d. Equivalent nailing patterns are required for ceiling joist to ceiling joist lap splices.

~~f~~ e. Connected members shall be of sufficient size to prevent splitting due to nailing.

~~g~~ f. For snow loads less than 30 pounds per square foot, the required number of nails is permitted to be reduced by multiplying by the ratio of actual snow load plus 10 divided by 40, but not less than the number required for no snow load.

g. Applies to roof live load of 20 psf or less.

h. Tabulated heel joint connection requirments assume that ceiling joists or rafter ties are located at the bottom of the attic space. Where ceiling joists or rafter ties are located higher in the attic, heel joint connection requirements shall be increased by the following factors:

H_C/H_R	Heel Joint Connection Adjustment Factor
1/3	1.5
1/4	1.33
1/5	1.25
1/6	1.2
1/10 or less	1.11

where:

H_C = Height of ceiling joists or rafter ties measured vertically above the top of the rafter support walls.

H_R = Height of roof ridge measured vertically above the top of the rafter support walls.

i. Tabulated requirements are based on 10 psf roof dead load in combination with the specified roof snow load and roof live load.

Reason: Replace Table 2308.7.3.1 to be consistent with calculation basis of 2018 Wood Frame Construction Manual (WFCM) heal joint nailing requirements based on the 2018 National Design Specification for Wood Construction (NDS) provisions for nailed connections. The reduced number of 16d common nails required in rafter tie connections, by approximately 15%, are due to changes in penetration factor and load duration assumptions from those used to develop the existing table. The existing table used a 0.77 penetration factor (based on 1991 and 1997 NDS) for 16d common nails with less than 12d penetration in the main member and a load duration factor of 1.25 for all tabulated cells. The proposed revised nailing requirements are based on use of a 1.15 load duration factor for snow cases, 1.25 load duration factor for roof live load cases, and an effective penetration factor equal to 1.0 per 2001 NDS and later editions when nail lateral value calculations are based on the actual penetration in the wood member. The ratio of nail design values for snow cases originally used to develop nailing requirements to the current nail design values for snow cases is (Z x 0.77 x 1.25)/(Z x 1.0 x 1.15) = 0.84 and explains the reduced number of nails required by this proposal. Due to revised nail design provisions in the NDS, the benefit of a longer nail that is clinched is no longer recognized for this application and existing footnote b is removed. A 10d common nail option is added in new footnote “a.” based on NDS lateral nail calculations. The table heading clarifies the 10psf dead load basis of the tabulated nailing requirements. Also, adjustment factors for rafter tie height, consistent with WFCM and IRC, are added in footnote “h.” to increase connection requirements where the rafter tie not located in the bottom of the attic space (i.e. rafter ties located at the top of the support walls).

Bibliography: 1) 2018 Wood Frame Construction Manual (WFCM) for One- and Two-Family Dwellings. American Wood Council, Leesburg, VA. 2018. https://awc.org/pdf/codes-standards/publications/wfcm/AWC-WFCM2018-ViewOnly-1711.pdf
2) 2018 National Design Specification (NDS) for Wood Construction. American Wood Council, Leesburg, VA. 2018. https://awc.org/pdf/codes-standards/publications/nds/AWC-NDS2018-ViewOnly-171117.pdf

Cost Impact: The code change proposal will decrease the cost of construction
This code change proposal utilizes fewer nails from the wood frame construction manual at less cost.

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Public Hearing Results

Committee Action: **As Submitted**

Committee Reason: Replace Table 2308.7.3.1 to be consistent with calculation basis of 2018 Wood Frame Construction Manual (WFCM) heal joint nailing requirements based on the 2018 National Design Specification for Wood Construction (NDS) provisions for nailed connections.
(Vote: 14-0)

Assembly Action: **None**

S187-19

Individual Consideration Agenda

Public Comment 1:

IBC®: TABLE 2308.7.3.1

Proponents:
Paul Coats, representing American Wood Council (pcoats@awc.org)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

TABLE 2308.7.3.1
RAFTER TIE CONNECTIONS [i]

RAFTER SLOPE	TIE SPACING (inches)	~~NO SNOW LOAD~~ LIVE LOAD ONLY[g]			GROUND SNOW LOAD (pound per square foot)					
					30 pounds per square foot			50 pounds per square foot		
		Roof span (feet)								
		12	24	36	12	24	36	12	24	36
		Required number of 16d common (3-1/2" x 0.162") nails per connection[a, b, c, d, e, f, h]								
3:12	12	3	5	8	3	6	9	5	9	13
	16	4	7	10	4	8	12	6	12	17
	19.2	4	8	12	5	10	14	7	14	21
	24	5	10	15	6	12	18	9	17	26
	32	7	13	20	8	16	24	12	23	34
	48	10	20	29	12	24	35	17	34	51
4:12	12	3	4	6	3	5	7	4	7	10
	16	3	5	8	3	6	9	5	9	13
	19.2	3	6	9	4	7	11	6	11	16
	24	4	8	11	5	9	13	7	13	19
	32	5	10	15	6	12	18	9	17	26
	48	8	15	22	9	18	26	13	26	38
5:12	12	3	3	5	3	4	6	3	6	8
	16	3	4	6	3	5	7	4	7	11
	19.2	3	5	7	3	6	9	5	9	13
	24	3	6	9	4	7	11	6	11	16
	32	4	8	12	5	10	14	7	14	21
	48	6	12	18	7	14	21	11	21	31
7:12	12	3	3	4	3	3	4	3	4	6
	16	3	3	5	3	4	5	3	5	8
	19.2	3	4	5	3	4	6	3	6	9
	24	3	5	7	3	5	8	4	8	11
	32	3	6	9	4	7	10	5	10	15
	48	5	9	13	5	10	15	8	15	22
9:12	12	3	3	3	3	3	3	3	3	5
	16	3	3	4	3	3	4	3	4	6
	19.2	3	3	4	3	4	5	3	5	7
	24	3	4	5	3	4	6	3	6	9
	32	3	5	7	3	6	8	4	8	12
	48	4	7	10	4	8	12	6	12	17
12:12	12	3	3	3	3	3	3	3	3	4
	16	3	3	3	3	3	3	3	3	5
	19.2	3	3	3	3	3	4	3	4	6
	24	3	3	4	3	3	5	3	5	7
	32	3	4	5	3	4	6	3	6	9
	48	3	5	8	3	6	9	5	9	13

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot = 47.8 N/m^2.

a. 10d common (3" x 0.148") nails shall be permitted to be substituted for 16d common ($3^1/_2$″ × 0.162″) nails where the required number of nails is taken as 1.2 times the required number of 16d common nails , rounded up to the next full nail.

b. Rafter tie heel joint connections are not required where the ridge is supported by a load-bearing wall, header or ridge beam.

c. Where intermediate support of the rafter is provided by vertical struts or purlins to a load-bearing wall, the tabulated heel joint connection requirements are permitted to be reduced proportionally to the reduction in span.

d. Equivalent nailing patterns are required for ceiling joist to ceiling joist lap splices.

e. Connected members shall be of sufficient size to prevent splitting due to nailing.

f. For snow loads less than 30 pounds per square foot, the required number of nails is permitted to be reduced by multiplying by the ratio of actual snow load plus 10 divided by 40, but not less than the number required for no snow load.

g. Applies to roof live load of 20 psf or less.

h. Tabulated heel joint connection requirments assume that ceiling joists or rafter ties are located at the bottom of the attic space. Where ceiling joists or rafter ties are located higher in the attic, heel joint connection requirements shall be increased by the following factors:

H_C/H_R	Heel Joint Connection Adjustment Factor
1/3	1.5
1/4	1.33
1/5	1.25
1/6	1.2
1/10 or less	1.11

where:

H_C = Height of ceiling joists or rafter ties measured vertically ~~above~~ from the top of the rafter support walls to the bottom of the ceiling joists or rafter ties.

H_R = Height of roof ridge measured vertically ~~above~~ from the top of the rafter support walls to the bottom of the roof ridge.

When H_C / H_R exceeds 1/3, connections shall be designed in accordance with accepted engineering practice.

i. Tabulated requirements are based on 10 psf roof dead load in combination with the specified roof snow load and roof live load.

Commenter's Reason: Several clarifications were suggested by the Structural Committee, and they are contained in this public comment: 1) a column heading is changed to clarify that it applies to live loads only, with a limit of 20 lbs. per footnote "g"; 2) text is added to footnote "a" to clarify that results should be rounded to the next full nail; 3) a clarifying sentence is added beneath the table in footnote "h" to clarify that rafter tie connections higher than H_C / H_R = 1/3 in the attic space must be engineered; and 4) the definitions of H_C and H_R are clarified to show how they should be measured.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
The public comment clarifies the intent of the original proposal, which could represent some cost savings due to efficiencies in the table content, depending on application.

Public Comment# 1722

S190-19

IBC: 2405.1, 2405.3, 2405.3.1 (New), 2405.3.2 (New), 2405.3.3 (New)

Proposed Change as Submitted

Proponents: Jennifer Hatfield, representing American Architectural Manufacturers Association (jen@jhatfieldandassociates.com)

2018 International Building Code

Revise as follows:

2405.1 Scope. This section applies to the installation of glass and other transparent, translucent or opaque glazing material installed at a slope of more than 15 degrees (0.26 rad) from the vertical plane, including glazing materials in skylights, roofs and sloped walls.

2405.3 Screening. ~~Where used in monolithic glazing systems, heat-strengthened and fully tempered glass shall have screens installed below the glazing material. The screens and their fastenings~~ Broken glass retention screens, where required, shall be: capable of supporting twice the weight of the glazing; firmly and substantially fastened to the framing members; and installed within 4 inches (102 mm) of the glass. The screens shall be constructed of a noncombustible material not thinner than No. 12 B&S gage (0.0808 inch) with mesh not larger than 1 inch by 1 inch (25 mm by 25 mm). In a corrosive atmosphere, structurally equivalent noncorrosive screen materials shall be used. ~~Heat-strengthened glass, fully tempered glass and wired glass, where used in multiple-layer glazing systems as the bottom glass layer over the walking surface, shall be equipped with screening that conforms to the requirements for monolithic glazing systems.~~

~~**Exception:** In monolithic and multiple-layer sloped glazing systems, the following applies:~~

1. ~~Fully tempered glass installed without protective screens where glazed between intervening floors at a slope of 30 degrees (0.52 rad) or less from the vertical plane shall have the highest point of the glass 10 feet (3048 mm) or less above the walking surface.~~
2. ~~Screens are not required below any glazing material, including annealed glass, where the walking surface below the glazing material is permanently protected from the risk of falling glass or the area below the glazing material is not a walking surface.~~
3. ~~Any glazing material, including annealed glass, is permitted to be installed without screens in the sloped glazing systems of commercial or detached noncombustible greenhouses used exclusively for growing plants and not open to the public, provided that the height of the greenhouse at the ridge does not exceed 30 feet (9144 mm) above grade.~~
4. ~~Screens shall not be required in individual *dwelling units* in Groups R-2, R-3 and R-4 where fully tempered glass is used as single glazing or as both panes in an insulating glass unit, and the following conditions are met:~~

4.1. ~~Each pane of the glass is 16 square feet (1.5 m^2) or less in area.~~
4.2. ~~The highest point of the glass is 12 feet (3658 mm) or less above any walking surface or other accessible area.~~
4.3. ~~The glass thickness is $^3/_{16}$ inch (4.8 mm) or less.~~

5. ~~Screens shall not be required for laminated glass with a 15-mil (0.38 mm) polyvinyl butyral (or equivalent) interlayer used in individual *dwelling units* in Groups R-2, R-3 and R-4 within the following limits:~~

5.1. ~~Each pane of glass is 16 square feet (1.5 m^2) or less in area.~~
5.2. ~~The highest point of the glass is 12 feet (3658 mm) or less above a walking surface or other accessible area.~~

Add new text as follows:

2405.3.1 Screens under monolithic glazing Heat-strengthened glass, annealed glass, wired glass and fully tempered glass shall have screens installed below the full area of the glazing material.

2405.3.2 Screens under multiple-layer glazing. Heat-strengthened glass, fully tempered glass, annealed glass and wired glass, glazing used as the bottom glass layer.shall have retention screens installed below the full area of the glazing material.

2405.3.3 Screens not required. For all other types of glazing complying with Section 2405.2, retention screens shall not be required.

Exception: In monolithic and multiple-layer sloped glazing systems, the following apply; which includes laminated glass with a 30-mil interlayer.

1. Fully tempered glass shall not be required to be installed with retention screens where glazed between intervening floors at a slope of 30 degrees (0.52 rad) or less from the vertical plane, and having the highest point of the glass 10 feet (3048 mm) or less above the walking surface.
2. Retention screens shall not be required below any glazing material, including annealed glass, where the walking surface below the glazing material is permanently protected from the risk of falling glass or the area below the glazing material is not a walking surface.
3. Retention screens shall not be required below any glazing material, including annealed glass, the sloped glazing systems of commercial or detached noncombustible greenhouses used exclusively for growing plants and not open to the public, provided that the height of the greenhouse at the ridge does not exceed 30 feet (9144 mm) above grade.

<u>4.Retention screens shall not be required in individual dwelling units in Groups R-2, R-3 and R-4 where fully tempered glass is used as single glazing or as both panes in an insulating glass unit, and all of the following conditions are met:</u>
<u>4.1.Each pane of the glass is 16 square feet (1.5 m2) or less in area.</u>
<u>4.2.The highest point of the glass is 12 feet (3658 mm) or less above any walking surface or other accessible area.</u>
<u>4.3.The glass thickness is 3/16 inch (4.8 mm) or less.</u>
<u>5.Retention screens shall not be required for laminated glass with a 15-mil (0.38 mm) polyvinyl butyral (or equivalent) interlayer used in individual *dwelling units* in Groups R-2, R-3 and R-4, and both of the following conditions are met:</u>
<u>5.1.Each pane of glass is 16 square feet (1.5 m^2) or less in area.</u>
<u>5.2.The highest point of the glass is 12 feet (3658 mm) or less above a walking surface or other accessible area.</u>

Reason: The current code language that states when screens are required below unit skylights and sloped glazing, has frequently been difficult to interpret by jurisdictions, causing consumers and others great concern when they are incorrectly told they need to install a glass retention screen below conforming (30-mil interlayer) laminated glass. Skylight and sloped glazing system manufacturers are asked to intervene far too frequently to ensure that unsightly, unnecessary screens are not installed in these instances. Furthermore, it is believed that many times an optional skylight installation is removed from submitted plans due to misinterpretation at the plan check stage, where the supplier may never know that the issue was raised because the permit applicant may surrender rather than appeal.
The current code language addresses qualifying laminated glass by simple omission from the “screens required” section. It is this omission that seems to create the confusion within the industry, especially considering Exception 5, which mentions that screens may be required when non-qualifying (15-mil interlayer) laminated glass is used.

This proposed code change simply rewrites this section to state clearly that laminated glass with 30-mil interlayer does not require screens. Specifically addressing the inapplicability of screens under laminated glass in the new section 2405.3.3 should reduce the frequency of misinterpretations that have been experienced. Adding the modifier, “broken glass retention” fully describes the screen’s purpose. This is to ensure readers do not confuse them with insect screens or fall protection screens, which are physically different and will not serve as effective retention screens.

None of the proposed changes affect the current code requirements; rather, the intent and only expected outcomes of this proposal are simply for better clarity and more consistent enforcement.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
The proposal should have a nominal effect on the cost of construction as the changes presented are not meant to alter the current requirements but simply meant to provide better clarity and more consistent enforcement.

S190-19

<u>Public Hearing Results</u>

Committee Action: **Disapproved**

Committee Reason: The committee felt that, as written, the proposed reorganization of the code appears to cause more confusion than clarity. (Vote: 13-1)

Assembly Action: **None**

S190-19

<u>Individual Consideration Agenda</u>

Public Comment 1:

IBC®: 2405.1, 2405.2, 2405.3

Proponents:
Jennifer Hatfield, representing American Architectural Manufacturers Association (jen@jhatfieldandassociates.com)

requests As Modified by Public Comment

Replace as follows:

2018 International Building Code

2405.1 Scope. This section applies to the installation of glass and other transparent, translucent or opaque glazing material installed at a slope of more than 15 degrees (0.26 rad) from the vertical plane, including glazing materials in skylights, roofs and sloped walls.

2405.2 Allowable glazing materials and limitations. Sloped glazing shall be any of the following materials, subject to the listed limitations.

1. For monolithic glazing systems, the glazing material of the single light or layer shall be laminated glass with a minimum 30-mil (0.76 mm) polyvinyl butyral (or equivalent) interlayer, wired glass, light-transmitting plastic materials meeting the requirements of Section 2607, heat-strengthened glass or fully tempered glass.
2. For multiple-layer glazing systems, each light or layer shall consist of any of the glazing materials specified in Item 1.

Annealed glass is permitted to be used as specified in Exceptions 2 and 3 of Section 2405.3.

Laminated glass and plastic materials described above shall not require the screening or height restrictions provided in Section 2405.3.

For additional requirements for plastic skylights, see Section 2610. Glass-block construction shall conform to the requirements of Section 2110.1.

2405.3 Screening. Where used in monolithic glazing systems, annealed, heat-strengthened , ~~and~~ fully tempered and wired glass shall have broken glass retention screens installed below the glazing material. The screens and their fastenings shall be: capable of supporting twice the weight of the glazing; firmly and substantially fastened to the framing members; and installed within 4 inches (102 mm) of the glass. The screens shall be constructed of a noncombustible material not thinner than No. 12 B&S gage (0.0808 inch) with mesh not larger than 1 inch by 1 inch (25 mm by 25 mm). In a corrosive atmosphere, structurally equivalent noncorrosive screen materials shall be used. Annealed, ~~H~~ heat-strengthened ~~glass~~, fully tempered ~~glass~~ and wired glass, where used in multiple-layer glazing systems as the bottom glass layer over the walking surface, shall be equipped with screening that conforms to the requirements for monolithic glazing systems.

Exception: In monolithic and multiple-layer sloped glazing systems, the following applies:

1. Fully tempered glass installed without protective screens where glazed between intervening floors at a slope of 30 degrees (0.52 rad) or less from the vertical plane shall have the highest point of the glass 10 feet (3048 mm) or less above the walking surface.
2. Screens are not required below any glazing material, including annealed glass, where the walking surface below the glazing material is permanently protected from the risk of falling glass or the area below the glazing material is not a walking surface.
3. Any glazing material, including annealed glass, is permitted to be installed without screens in the sloped glazing systems of commercial or detached noncombustible greenhouses used exclusively for growing plants and not open to the public, provided that the height of the greenhouse at the ridge does not exceed 30 feet (9144 mm) above grade.
4. Screens shall not be required in individual *dwelling units* in Groups R-2, R-3 and R-4 where fully tempered glass is used as single glazing or as both panes in an insulating glass unit, and the following conditions are met:

    4.1. Each pane of the glass is 16 square feet (1.5 m^2) or less in area.

    4.2. The highest point of the glass is 12 feet (3658 mm) or less above any walking surface or other accessible area.

    4.3. The glass thickness is $^3/_{16}$ inch (4.8 mm) or less.

5. Screens shall not be required for laminated glass with a 15-mil (0.38 mm) polyvinyl butyral (or equivalent) interlayer used in individual *dwelling units* in Groups R-2, R-3 and R-4 within the following limits:

    5.1. Each pane of glass is 16 square feet (1.5 m^2) or less in area.

    5.2. The highest point of the glass is 12 feet (3658 mm) or less above a walking surface or other accessible area.

Commenter's Reason: The current code language that states when screens are required below unit skylights and sloped glazing, has frequently been difficult to interpret by jurisdictions, causing consumers and others great concern when they are incorrectly told they need to install a glass retention screen below conforming (30-mil interlayer) laminated glass. Skylight and sloped glazing system manufacturers are asked to intervene far too often to ensure that unsightly, unnecessary screens are not installed in these instances. Furthermore, it is believed that many times an optional skylight installation is removed from submitted plans due to misinterpretation at the plan check stage, where the supplier may never know that the issue was raised because the permit applicant may surrender rather than appeal.
The current code language addresses qualifying laminated glass by simple omission from the "screens required" section. It is the omission that seems to create the confusion within the industry, especially considering Exception 5, which mentions that screens may be required when non-qualifying (15-mil interlayer) laminated glass is used.

The initial code proposal submitted attempted to re-write the section to make it clear that laminated glass with 30-mil interlayer does not require screens. However, it was determined at the committee action hearings the initial attempt to reorganize and provide this clarity was actually more confusing. Taking that input this public comment attempts to once again make it clear that laminated glass with 30-mil interlayer does not require screens, but in what we think is a much more clear and direct manner. We believe this addresses the committee's concerns.

In instances where screens our required, the public comment adds the modifier, "broken glass retention" to fully describe the screen's purpose. This is to ensure readers do not confuse these type of screens with insect screens or fall protection screens, which are physically different and will not serve as effective retention screens. The public comment also provides minor "clean up" to ensure all types of glass addressed in Section 2405.3, are listed in the opening paragraph.

None of what is being proposed changes the current code requirements; rather, the intent and only expected outcome of the public comment is to simply provide better clarity and more consistent enforcement.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
The changes presented are not meant to alter the current code requirements but simply meant to provide clarity and more consistent enforcement.

Public Comment# 1345

S191-19

IBC®: 2405.2

Proposed Change as Submitted

Proponents: Jennifer Hatfield, representing American Architectural Manufacturers Association (jen@jhatfieldandassociates.com)

2018 International Building Code

Revise as follows:

2405.2 Allowable glazing materials and limitations. Sloped glazing shall be any of the following materials, subject to the listed limitations.

1. For monolithic glazing systems, the glazing material of the single light or layer shall be laminated glass with a minimum 30-mil (0.76 mm) polyvinyl butyral (or equivalent) interlayer, wired glass, light-transmitting plastic materials meeting the requirements of Section 2607, heat-strengthened glass or fully tempered glass.
2. For multiple-layer glazing systems, each light or layer shall consist of any of the glazing materials specified in Item 1.

Annealed glass is permitted to be used as specified in Exceptions 2 and 3 of Section 2405.3.

For additional requirements for plastic skylights, see Section 2610. ~~Glass-block construction shall conform to the requirements of Section 2110.1.~~

Reason: The removal of the reference in Section 2405.2 to the “Glass block” section is suggested as it removes a non-germane statement. That section contains no provisions that would apply on roofs or sloped walls, and Section 2405 offers no guidance on the use or protections needed for glass block. The reference is out of place here and should be removed. Perhaps there is a better section in Chapter 24 for it to appear, if it is needed at all.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This change will not have an effect on cost as it is not removing the requirements in Section 2110 but just removing the reference that is not germane within Section 2405.

S191-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee consensus was that the proposal was deleting a required pointer in the code.
(Vote: 11-3)

Note: the committee vote for 'as submitted' failed 7 for and 8 against.

Assembly Action: **None**

S191-19

Individual Consideration Agenda

Public Comment 1:

Proponents:
Jennifer Hatfield, representing American Architectural Manufacturers Association (jen@jhatfieldandassociates.com)

requests As Submitted

Commenter's Reason: The removal of the reference in section 2405.2 to the "Glass Block" section is suggested as it removes a non-germane statement. The glass-block construction section contains no provisions that would apply on roofs or sloped walls. Further, section 2405.2 offers no guidance on the use or protections needed for glass block. The reference is simply out of place here.

In addition, any concern that by removing this sentence one is eliminating a necessary pointer in Chapter 24 to the glass block provisions, is alleviated by the fact a reference is included in section 2406.1.3. The reference is more appropriate here in the safety glazing section where Glass Block is specifically called out in section 2406.1.3. Further, the reference here in section 2406.1.3 points you to the entire section on glass block, referring you to section 2110 whereas in Section 2405.2 it points you only to a subsection, 2110.1.

SECTION 2406
SAFETY GLAZING

2406.1 Human impact loads.

Individual glazed areas, including glass mirrors, in hazardous locations as defined in Section 2406.4 shall comply with Sections 2406.1.1 through 2406.1.4.

Exception: Mirrors and other glass panels mounted or hung on a surface that provides a continuous backing support.

2406.1.1 Impact test.

Except as provided in Sections 2406.1.2 through 2406.1.4, all glazing shall pass the impact test requirements of Section 2406.2.

2406.1.2 Plastic glazing.

Plastic glazing shall meet the weathering requirements of ANSI Z97.1.

2406.1.3 Glass block.

Glass-block walls shall comply with Section 2110.

2406.1.4 Louvered windows and jalousies.

Louvered windows and jalousies shall comply with Section 2403.5.

Bibliography: See section 2406.1.3 of the 2018 IBC

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction
The change will not have any effect on costs as its not removing or changing any code requirements. Rather it is simply removing a pointer that is not germane within section 2405 and that is already included in section 2406 where it is more appropriate.

Public Comment# 1305

S193-19

IBC®: 2407.1.1 (New)

Proposed Change as Submitted

Proponents: Anthony Barnes, Trex Commercial Products, representing self (tbarnes@trexcommercial.com); Bryan Wedan, Enclos, representing self

2018 International Building Code

Revise as follows:

2407.1.1 Loads. The glass panels ~~and their support system~~ shall be designed to withstand the loads specified in Section 1607.8. Glass ~~guard elements~~ panels shall be designed using a factor of safety of four applied to the modulus of rupture.

Reason: Allowable glass stress is traditionally determined by probabilistic methods (ASTM E1300) given particular load durations. The allowable stress decreases with a longer duration load (thus the factor of safety increases). Therefore the allowable stress calculated per ASTM E1300 effectively contains a factor of safety. This appears to be the intent of section 2407.1.1 and the code commentary does suggest this (see attachments) as it mentions probability of glass breakage and that 4x the load is not to be applied to a railing system. The factor of safety of four should only be applied to glass. All other components supporting glass should be designed using the factors of safety provided in relevant material codes (AISC 360 for steel, etc.). All other glass systems such as skylights and walls are designed in the same manner and carry no less risk than guards.
There are also inconsistencies and ambiguities with the current code language. The factor of safety does not define which supports the factor of safety of 4 is to be applied to (loads must be transferred to ground, so where does glass support end?). The language is inconsistent in that other railing types are not designed with the same factor of safety of 4 even though failure modes could be similar. For example, a factor of safety of 4 may be applied to a steel post-supported glass infill railing system, but if a steel mesh panel infill is substituted for the glass, this panel and its supports would be designed with lower factors of safety per the relevant material codes and thus failure modes (including panels falling out of supports catastrophically) would occur at much lower loads.

In summary, changing this language removes ambiguity, makes guard design more consistent with other similar systems, and saves money by lowering factors of safety for supports (to those that are used by the relevant material codes).

Cost Impact: The code change proposal will decrease the cost of construction
Glass is already designed with factor of safety of 4 per 2407.1.1 so no change there. Factor of safety for steel, stainless steel, aluminum and concrete supports will be per relevant material codes and those factors are generally less than 4 (less costly) and are familiar to designers (less costly).

S193-19

Public Hearing Results

Committee Action: **As Modified**

Committee Modification:
2018 International Building Code

2407.1.1 Loads. The glass panels and their support system shall be designed to withstand the loads specified in Section 1607.8. Glass panels shall be designed using a factor of safety of four applied to the modulus of rupture.

Committee Reason: The proposal draws attention to the fact that glass panels are to be designed using a factor of safety of 4. The modification clarifies the intent of the proposal. (Vote: 11-2)

Assembly Action: **None**

S193-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 2407.1.1 (New)

Proponents:
Anthony Barnes, representing Self (tbarnes@trexcommercial.com)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

2407.1.1 Loads. ~~The glass panels and their support system shall be designed to withstand the loads specified in Section 1607.8. Glass panels shall be designed using a factor of safety of four applied to the modulus of rupture.~~

The panels and their support system shall be designed to withstand the loads specified in Section 1607.8. When analyzed with these loads, the glass components of the handrails and guards shall be designed using the minimum allowable edge stresses as follows:

1. Fully tempered glass panels shall be designed using an allowable stress of 6.8 ksi.

2. Heat-strengthened glass panels used as guards shall be designed with an allowable stress of 3.1 ksi.

Exception: Alternate allowable stresses shall be permitted where justified by analysis using base stresses and methodology in ASTM E1300.

Commenter's Reason: The Structural Committee approved a stopgap measure (S193-19) that retained the load requirement for glass supports, but the above revised proposal enhances this approved modification is supported by technically correct, and accepted, methods for glass design. The differences between the above proposed language and the Committee-approved version (S193-19) are as follows:

1) This version uses accepted methods to set minimum allowable glass stresses (note that we arrive at basically the same values as were ambiguously *implied* in previous code versions using a now much more robust and technically appropriate analysis; previous code versions: 24 ksi MOR / 4 = 6 ksi)

2) It is explicitly stated that alternate allowable stresses are permitted using the methodology outlined in ASTM E 1300

These minimum allowable edge stresses are based upon the following parameters:

- Base allowable stress values given in ASTM E1300 Table X7.1

- Glass probability or breakage equals 1 lite in 1000 (typical for life-safety applications and overhead glazing – see GANA Glazing Manual for example)

- The loads of Section 1607.8 are assumed to have a duration of 1 hour

There are three key drivers for the proposed updated language:

1. No glass design values are specified or referenced to which the “design factor” is applied;

2. Using the “design factor” terminology is technically problematic as simple design factors do not apply to brittle materials such as glass (see AAMA CW-12-84 and NCSEA Engineering Structural Glass Design Guide)

3. Allow design professionals to derive, using ASTM E 1300, and use, alternate allowable stresses

1.1 Point 1

Current ***StopGap*** language specifies the use of a “design factor” on the modulus of rupture (MoR), however 2407.1.1 currently does not specify a value or source to which to refer. To address this gap and fulfill the code intent, a “floor” value for glass design stress based on current accepted practice and standards is proposed. This floor value is a simplified design value based on conservative design assumptions that, when used without further analysis, would result in a safe glass guard design. Further effort to increase this floor value by rigorous engineering analysis is accommodated by reference to acceptable industry practice and standards.

1.2 Point 2

The use of Modulus of Rupture with an applied design factor is an inaccurate and outdated method by which to determine allowable design stresses (ref. GANA). Current techniques to determine allowable glass stress utilize probabilistic methods (explained in AAMA CW-12-84, GANA Glazing Manual, and ASTM E1300) based on, among other factors, a given load duration (allowable stress is greater for a short duration, and lower for a longer duration load), and acceptable risk of breakage (“probability of breakage” or P(breakage)). These methods augment material data, obtained by experiment and documented in ASTM standards, to suit the specific design situation.

The proposed allowable stresses are derived using ASTM E1300-16 per the below calculations. For glass in handrails and guards, a P(breakage) = 1:1000 and a 1 hr load duration is assumed. Furthermore allowable glass stresses also vary with location of the stress on the glass panel, either on the surface or the edge: The glass edge is conservatively considered, as the allowable stresses are lower at edges than for glass surfaces.

(It is noted that typical vertical glass applications assume a 3s load duration with a P(breakage) = 8:1000: the proposed 1 hr at 1:1000 is a significant increase in conservatism for the specific case of glass handrails and guards to reflect the “critical application”.)

Following the proposed method, the design stresses proposed are similar in magnitude to the historical MoR/4 values but with a more robust derivation and clearer direction for the designer. This updated approach, based on documented ASTM methods of glass design, is particularly important as the codes are extended to the use of laminated glass and no top-cap in guards – design methods and techniques must keep pace with code requirements as higher performance is demanded from the material.

1.3 Point 3

Note that the approach used to derive the proposed allowable stresses is conservative when considering most guardrail applications: the proposed stress values address the balustrade condition of a single glass leaf, without top cap, cantilevering from an embedded shoe, subjected directly to crowd loading. Where not subject to these high demands, increased allowable stress limits could be used with more rigorous engineering analysis employed. For example, a shorter duration load may be appropriate (e.g. 3s duration for wind gust), or less demanding 4-side support conditions of infill panels ensure peak stresses occur on the glass surface. Extension of glass design stresses beyond the proposed floor values is limited to reference to associated design standards and outside of IBC scope.

Base stresses

Determine probability of breakage factor to convert from P(breakage) = 8:1000 stresses, as documented in ASTM E 1300, to equivalent P(breakage) = 1:1000 stresses

$P_{b1} := 0.001$

$$\sigma_{allowable} = \left(\frac{P_b}{[k\,(d/3)^{7/n} * A]}\right)^{1/7} \qquad (X6.1)$$

$P_{b2} := 0.008$

$$\psi_{Pb} := \frac{(P_{b1})^{\left(\frac{1}{7}\right)}}{P_{b2}^{\left(\frac{1}{7}\right)}} = 0.743$$

TABLE X7.1 Allowable Edge Stress

	Clean Cut Edges, MPa (psi)	Seamed Edges, MPa (psi)	Polished Edges, MPa (psi)
Annealed	16.6 (2400)	18.3 (2650)	20.0 (2900)
Heat-strengthened	N/A[A]	36.5 (5300)	$\boxed{36.5\ (5300)}$
Tempered	N/A	73.0 (10 600)	$\boxed{73.0\ (10\ 600)}$

[A] N/A–Not Applicable.

Use base stress of 10.6 ksi for probability of breakage of 8:1000 (from Table X7.1)

$\sigma_{3\_FT} := 10.6 \cdot ksi \cdot \psi_{Pb} = 7.876 \cdot ksi$

allowable edge stress for FT glass, 1:1000 probability of breakage, 3s load duration, ψ_{Pb} = 0.743 per E1300 X6.1

$$d_{FT} := \begin{pmatrix} 3\cdot s \\ 60\cdot s \\ \boxed{1\cdot hr} \\ 12\cdot hr \\ 10\cdot yr \end{pmatrix}$$

$$\sigma_{all\_FT} := \sigma_{3\_FT} \cdot \left(3\cdot\frac{s}{d_{FT}}\right)^{\left(\frac{1}{48}\right)}$$

per ASTM E1300-16 equation X5.1 using 'n' values found in Table 1 of ASTM E-2751

$$\sigma_{all\_FT} = \begin{pmatrix} 7.876 \\ 7.399 \\ \boxed{6.794} \\ 6.451 \\ 5.36 \end{pmatrix} \cdot ksi$$

$\sigma_{3\_HS} := 5.3 \cdot ksi \cdot \psi_{Pb} = 3.938 \cdot ksi$

allowable edge stress for HS glass, 1:1000 probability of breakage, 3s load duration

$$d_{HS} := \begin{pmatrix} 3\cdot s \\ 60\cdot s \\ \boxed{1\cdot hr} \\ 12\cdot hr \\ 10\cdot yr \end{pmatrix}$$

$$\sigma_{all\_HS} := \sigma_{3\_HS} \cdot \left(3\cdot\frac{s}{d_{HS}}\right)^{\left(\frac{1}{32}\right)}$$

per ASTM E1300-16 equation X5.1, where n = 32 per ASTM E2751 Table 1

$$\sigma_{all\_HS} = \begin{pmatrix} 3.938 \\ 3.586 \\ \boxed{3.155} \\ 2.92 \\ 2.211 \end{pmatrix} \cdot ksi$$

1 Additional response to committee comments:

Note that IBC Commentary includes the following clarification:

2407.1.1 Loads. The panels and their support system shall be designed to withstand the loads specified in Section 1607.7. A safety factor of four shall be used.

❖ This section requires that railing systems using glass balusters be designed based on a safety factor of four. Nominally identical panes of glass inherently have a wide variation in strength. The safety factor of four is used in the design to minimize the likelihood that breakage will occur below the design loads. It is not intended that an in-place glass railing system be tested for or capable of withstanding four times the design load.

This reads:

“This section requires that the support system for glass guard or handrail assemblies be designed based on a factor of safety of four. Nominally identical panes of glass inherently have a wide variation in strength. The safety factor of four is used in the design to minimize the likelihood that breakage will occur below the design loads. <u>It is not intended that an in-place glass guard or handrail system be tested for or capable of withstanding four times the design load</u>.”

Bibliography:

1) GANA - Glazing Manual 50th Anniversary

2) AAMA CW-12-84

3) ASTM E 1300

4) ASTM E 2751

Cost Impact: The net effect of the public comment and code change proposal will decrease the cost of construction
The code change proposal will decrease the cost of construction. Factors of safety for steel, stainless steel, aluminum and concrete supports will be per relevant material codes and those factors are generally less than 4 (less costly) and are familiar to designers (less costly). Costly and protracted discussion on code ambiguity with permitting jurisdictions is mitigated by clear language based on current design methods and material knowledge.

Public Comment# 1394

Public Comment 2:

Proponents:
Tom Zaremba, representing Glazing Industry Code Committee (GICC), a section of the National Glass Association (NGA) (tzaremba@ralaw.com)

requests Disapprove

Commenter's Reason: The Glazing Industry Code Committee (GICC) urges you to disapprove S193-19. The proposal as modified, would make Section 2407.1.1 inconsistent with changes made in S192-19 that were unanimously approved by the Committee and, more importantly, it would make 2407.1.1 inconsistent with the language of design requirements specified in Section 1607.8 of the IBC.

As originally proposed, S193-19 was fatally flawed. It deleted the words "and their support systems" from the first sentence of Section 2407.1.1. That deletion would have eliminated the requirement that support systems for glass guards be designed to the loads specified in Section 1607.8. That error was corrected by the Committee's adoption of a modification adding that language back into 2407.1.1. However, in addition to making that correction to S193-19, the Committee allowed two other changes from the original proposal to stand, namely, changing "guard elements" to "panels," and adding "modulus of rupture" to the safety factor required by 2407.1.1.

Adopting these two changes from the original proposal are not justified for several reasons. First, the Committee unanimously recommended the adoption of S192-19, which changed Section 2407.1.1 to read as follows: "*Glass* ***handrails and guards*** *and their support system shall be designed* *to withstand the loads specified in Section 1607.8. All glass* **handrails and guards** *shall be designed using a factor of safety of four.*" (Emphasis added). The reason for this change was to ensure consistency between Section 2407.1.1 and Section 1607.8. In that regard, 1607.8.1 provides that: "**Handrails and guards** *shall be designed to resist a linear load of 50 pounds per linear foot ... in accordance with ASCE 7. Glass* **handrails and guards** *shall comply with Section 2407.*" (Emphasis added).

Please note that in order to make 2407.1.1 consistent with 1608.1.1, S192-19 changed the word "panels" to "handrails and guards" in 2407.1.1. Then, after recommending the adoption of S192-19, the Committee recommended that "handrails and guards" be changed back to "panels" in S193-19. Adopting this change from S193-19 would restore the inconsistency between Sections 2407.1.1 and 1608.1 that S192-18 resolved.

Additionally, the Committee's recommendation as to S193-19 would add the term "modulus of rupture" to the safety factor required by 2407.1.1. However, the Committee's recommendation includes no information as to how to test for the "modulus of rupture," how to determine the modulus of rupture, or, for that matter what it means in this context. In that regard, the term "modulus of rupture" is not defined in the IBC. (Unlike here, where the term "modulus of rupture" is used in the IBC, the code specifies how it is to be determined. See, therefore, Section 2109.2.1.2.4.)

Section 1608.1 requires handrails and guards to "be designed to resist a linear load of 50 pounds per linear foot ... in accordance with ASCE 7." Section 1608.1 makes no mention of testing "modulus of rupture." Instead, it refers to testing in accordance with ASCE 7. If modulus of rupture is to be added to the design considerations applicable to handrails and guards, it should either be added to Section 1607.8.1 or to ASCE 7, but not to 2407.1.1. (And, if ASCE 7 requires modulus of rupture testing, then adding it to 2407.1.1 is unnecessary.)

The use of inconsistent and undefined terms inevitably leads to misinterpretations. Adopting S193-19 will result in inconsistent terms between Sections 1608 and 2407 of the IBC and should be disapproved. The Committee's modification restoring the "and their support systems" language is unnecessary since that language already appears in 2407.1.1. More importantly, however, the Committee got it wrong when it recommended changing the language of 2407.1.1 back to "panels" from "handrails and guards" as proposed in S192-19 since "handrails and guards" is the language used in 1608.1. The Committee again got it wrong when it recommended that "modulus of rupture" be added to the safety factor referenced in 2407.1.1. "Modulus of rupture" is not defined in the IBC and, if it belongs in the design considerations for handrails and guards, it should be included, if at all, either in Section 1608.1 or ASCE 7, but it should not be included in Section 2407.1.1.

The Glazing Industry Code Committee urges you to vote to disapprove S193-19.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction No change to code.

Public Comment# 1642

S194-19

IBC®: 2510.6

Proposed Change as Submitted

Proponents: Craig Conner, representing self (craig.conner@mac.com); Joseph Lstiburek, representing self (joe@buildingscience.com)

2018 International Building Code

Revise as follows:

2510.6 Water-resistive barriers. *Water-resistive barriers* shall be installed as required in Section 1403.2 and, where applied over wood-based sheathing, shall include a water-resistive ~~vapor-permeable~~ barrier with a ~~performance~~ water resistance at least equivalent to two layers of *water-resistive barrier* complying with ASTM E2556, 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 1404.4) intended to drain to the *water-resistive barrier* is directed between the layers.

Exceptions:

1. Where the water-resistive barrier that is applied over wood-based sheathing has a water resistance equal to or greater than that of a water-resistive barrier complying with ASTM E2556 ~~,~~ Type ~~II~~ I and is separated from the stucco by an intervening ~~, substantially nonwater-absorbing layer or drainage~~ foam plastic insulating sheathing layer or by a minimum 3/16 inch space.
2. Where the *water-resistive barrier* is applied over wood-based sheathing ~~in Climate Zone 1A, 2A or 3A, a ventilated air~~ where the annual mean rainfall as determined by the National Oceanic and Atmospheric Administration (NOAA) exceeds 20 inches, a minimum 3/16 inch space shall be provided between the stucco and *water-resistive barrier*.

Reason: Objective:

1. Define water resistance as the primary functional requirement of the WRB and remove reference to vapor permeable.
2. Enable a single layer of WRB complying with ASTM E2556 Type 1 with a drainage space.
3. Define depth drainage space.

The existing code language gives insufficient guidance for other approved materials. The added language addresses this issue and provides a specific performance requirement for water resistance and provides consistency with other sections of the code that relate specifically to water resistive barriers.

The size othe the drainage space needs to be specified. Type 1 is the appropriate water-resistive metric for the specified space. This logic is consistent with the body and intent of the text of Section 2510.6. The specified space and one layer of Type 1 provides equivalent performance to the two layers of Type 1 specified in the body of 2510.6.

Annual mean rainfall is the appropriate metric for risk not humidity.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This change gives better guidance for water-resistance.

S194-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee felt that this change only confused the code content. The committee did not find sufficient justification for undefined terms such as 'approved weather data'.
(Vote: 14-0)

Assembly Action: **None**

S194-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 2510.6

Proponents:
Craig Conner, representing self (craig.conner@mac.com); Joseph Lstiburek, representing self (joe@buildingscience.com)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

2510.6 Water-resistive barriers. *Water-resistive barriers* shall be installed as required in Section 1403.2 and, where applied over wood-based sheathing, shall include a water-resistive barrier with a water resistance at least equivalent to two layers of *water-resistive barrier* complying with ASTM E2556, 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 1404.4) intended to drain to the *water-resistive barrier* is directed between the layers.

Exceptions:

1. Where the water-resistive barrier that is applied over wood-based sheathing has a water resistance equal to or greater than that of a water-resistive barrier complying with ASTM E2556 Type I and is separated from the stucco by an intervening foam plastic insulating sheathing layer or by a minimum 3/16 inch space.
2. Where the *water-resistive barrier* is applied over wood-based sheathing in the moist or marine climate zones of Figure N1101.7 ~~where the annual mean rainfall as determined by the National Oceanic and Atmospheric Administration (NOAA) exceeds 20 inches~~, a minimum 3/16 inch space shall be provided between the stucco and *water-resistive barrier* or a drainage layer having a drainage efficiency of not less than 90%, as measured in accordance with ASTM E2273 or Annex A2 of ASTM E2925, shall be added to the exterior side of the water-resistive barrier.

Commenter's Reason: The committee issue was and is addressed in the proposed modification.
Cleans up language relating to the function of a WRB so that S194 is now consistent with RB232 that passed in the IRC.

The modification makes the climate requirement for drainage consistent with the language in RB242 that passed in the IRC

The modification makes the requirement defining drainage consistent with the language in RB243 and RB242 that passed in the IRC

More significantly it recognizes that the most important factor relating to addressing the issues with stucco are drainage not the resistance to hydrostatic pressure. In other words drainage is more important than requiring a Type II water resistive barrier. A Type I water resistive barrier with drainage significantly outperforms a Type II water resistive barrier without drainage. ASTM E2556 does not address drainage.

ASTM E2556 requires materials to resist a water column of over 20 inches of water…a hydrostatic pressure greater than 5,000 pascals (an equivalent wind speed of 200 hundred miles per hour). The requirement is disingenuous when it is understood that sheet membranes are tested under ASTM E2556 without fasteners. Nails are required to install such products…as well as other products. Cladding fasteners then penetrate all products. The key is to control the hydrostatic pressure so the holes don’t matter.

Requiring a Type II water resistive barrier creates an artificial barrier to entry for products and approaches that have been demonstrated to work. It excludes products such as OSB sheathings with integral water control layers manufactured by Georgia Pacific, Louisiana Pacific and Huber. It excludes many fluid applied water resistive barriers and it adds unnecessary expense to drainage mat and dimple matt drainage approaches where Type I water resistive barriers function well. Requiring Type II water resistive barriers favors mechanically attached sheet good based water resistive barriers despite evidence that they do not function adequately in stucco assemblies without a gap.

Cost Impact: The net effect of the public comment and code change proposal will decrease the cost of construction
Requiring materials to meet Type II requirements significantly increases costs relative to meeting Type I requirements. This requirement doubles the material per square foot cost of water resistive barriers resulting in cost increases on the order of thousands of dollars on multifamily and commercial projects. Therefore, this code change significantly reduces the cost of construction by thousands of dollars on multifamily and commercial projects.

Staff Analysis: ASTM E2925 is a new standard that was submitted to staff in accordance with CP28 in support of S196-19, in which the standard is referenced.
Note: Both S194-19 and S196-19 deal with the same section in different ways. If both are approved, please ensure the final intentions are clear.

Public Comment 2:

Proponents:
Jay Crandell, P.E., ARES Consulting, representing Foam Sheathing Committee of the American Chemistry Council (jcrandell@aresconsulting.biz)

requests Disapprove

Commenter's Reason: The committee recommended disapproval of S194 and this should be upheld for the reasons given by the IBC-S committee. This proposal included a number of problems including undefined terms such as “approved weather data”. There is also concern that water-resistance was being reduced by changing from a Type II to Type I WRB in accordance with ASTM E2556. There was not adequate justification given for this change. Finally, it should be noted that all of these issues were resolved in S196 which was recommended for approval (14-0) by the committee noting that it “provides update of existing provisions to the latest technology and the drainage for correct climate zones.” In addition, S196 is coordinated with RB242 which also was approved for the IRC. For these reasons, we request that S194-19 remain disapproved.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction No change to code.

Public Comment# 1746

S196-19

IBC: 2510.6, 2510.6.1 (New), 2510.6.2 (New), ASTM Chapter 35 (New)

Proposed Change as Submitted

Proponents: Mike Fischer, representing Self (mfischer@kellencompany.com); Jay Crandell, P.E., ARES Consulting, representing Foam Sheathing Committee of the American Chemistry Council

2018 International Building Code

Revise as follows:

2510.6 Water-resistive barriers. *Water-resistive barriers* shall be installed as required in Section 1403.2 and, where applied over wood-based sheathing, shall comply with Section 2510.6.1 or Section 2510.6.2. ~~include a water-resistive vapor-permeable barrier with a performance at least equivalent to two layers of *water-resistive barrier* complying with ASTM E2556, 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 1404.4) intended to drain to the *water-resistive barrier* is directed between the layers.~~

~~**Exceptions:**~~

~~1.Where the *water-resistive barrier* that is applied over wood-based sheathing has a water resistance equal to or greater than that of a *water-resistive barrier* complying with ASTM E2556, 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 wood-based sheathing in Climate Zone 1A, 2A or 3A, a ventilated air space shall be provided between the stucco and *water-resistive barrier*.~~

Add new text as follows:

2510.6.1 Dry climates. One of the following shall apply for dry (B) *climate zones*:

1. The *water-resistive barrier* shall be two layers of 10-minute Grade D paper or have a water resistance equal to or greater than two layers of *water-resistive barrier* complying with ASTM E2556, 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 1404.4 and intended to drain to the *water-resistive barrier*, is directed between the layers.
2. The *water-resistive barrier* shall be 60-minute Grade D paper or have a water resistance equal to or greater than one layer of *water-resistive barrier* complying with ASTM E2556, Type II. The water-resistive barrier shall be separated from the stucco by a layer of foam plastic insulating sheathing or other nonwater absorbing layer.

2510.6.2 Moist or marine climates. In moist (A) or marine (C) *climate zones*, water-resistive barrier shall comply with of one of the following:

1. In addition to complying with Item 1 or 2 of Section 2510.6.1, a minimum 3/16 inch (4.8 mm) space shall be added to the exterior side of the *water-resistive barrier.*
2. In addition to complying with Item 2 of Section 2510.6.1, a space with a minimum drainage efficiency of 90% as measured in accordance with ASTM E2273 or Annex A2 of ASTM E2925 is added to the exterior side of the *water-resistive barrier.*

ASTM

ASTM International
100 Barr Harbor Drive, P.O. Box C700
West Conshohocken PA 19428-2959

E2925-17: Standard Specification for Manufactured Polymeric Drainage and Ventilation Materials Used to Provide a Rainscreen Function

Reason: The proposal does two things. First, it reorganizes the provisions by deleting two exceptions (which are really a construction options or requirements) and replacing them with subsections that indicate different methods of complying with stucco water-resistive barrier requirements. Second, the proposal properly applies requirements in relation to climate zones (a defined term in Chapter 2) -- something that has been missing in the code and is needed to avoid higher risk of moisture problems in climates that are moist/rainy. The proposal will help resolve problems with stucco performance (e.g., moisture problems over wood-based sheathings) and avoid impacting cost or performance where stucco has a long-standing record of good performance (e.g., dry climates such as the southwestern region of the U.S.).

Cost Impact: The code change proposal will increase the cost of construction
The proposal will not increase cost for substrates other than wood-based sheathing. Also, it will not impact cost or change requirements in dry climates where stucco has a long record of successful performance. This also will not impact cost in moist or marine climates where similar actions are already being taken (e.g., a drainage space) to reduce risk of moisture damage.

Staff Analysis: A review of the standard proposed for inclusion in the code, ASTM E2925-17, 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 2, 2019.

Public Hearing Results

Committee Action: **As Modified**

Committee Modification:
2018 International Building Code

2510.6.1 Dry climates. One of the following shall apply for dry (B) *climate zones*:

1. The *water-resistive barrier* shall be two layers of 10-minute Grade D paper or have a water resistance equal to or greater than two layers of *water-resistive barrier*complying with ASTM E2556, 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 1404.4 and intended to drain to the *water-resistive barrier*, is directed between the layers.
2. The *water-resistive barrier* shall be 60-minute Grade D paper or have a water resistance equal to or greater than one layer of *water-resistive barrier* complying with ASTM E2556, Type II. The water-resistive barrier shall be separated from the stucco by a layer of foam plastic insulating sheathing or other nonwater absorbing layer , or a drainage space.

2510.6.2 Moist or marine climates. In moist (A) or marine (C) *climate zones*, water-resistive barrier shall comply with of one of the following:

1. In addition to complying with Item 1 or 2 of Section 2510.6.1, a space or drainage material not less than ~~minimum~~ 3/16 inch (4.8 mm) in depth ~~space~~ shall be applied~~added~~ to the exterior side of the *water-resistive barrier.*
2. In addition to complying with Item 2 of Section 2510.6.1, drainage on the exterior side of the water-resistive barrier shall have ~~a space with~~ a minimum drainage efficiency of 90% as measured in accordance with ASTM E2273 or Annex A2 of ASTM E2925 ~~is added to the exterior side of the *water-resistive barrier*~~.

Committee Reason: The proposal provides update of existing provisions to the latest technology and the drainage for correct climate zones. The modification adds additional options to satisfying the requirements. (Vote: 14-0)

Assembly Action: **None**

S196-19

Individual Consideration Agenda

Public Comment 1:

IBC®: 2510.6.1 (New)

Proponents:
Craig Conner, representing self (craig.conner@mac.com); Joseph Lstiburek, representing self (joe@buildingscience.com)

requests As Modified by Public Comment

Further modify as follows:

2018 International Building Code

2510.6.1 Dry climates. One of the following shall apply for dry (B) *climate zones*:

1. The *water-resistive barrier* shall be two layers of 10-minute Grade D paper or have a water resistance equal to or greater than two layers of *water-resistive barrier* complying with ASTM E2556, 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 1404.4 and intended to drain to the *water-resistive barrier*, is directed between the layers.
2. The *water-resistive barrier* shall be 60-minute Grade D paper or have a water resistance equal to or greater than one layer of *water-resistive barrier* complying with ASTM E2556, Type I or II. The water-resistive barrier shall be separated from the stucco by a layer of foam plastic insulating sheathing or other nonwater absorbing layer, or a drainage space.

Commenter's Reason: Requiring materials to meet Type II requirements significantly increases costs relative to meeting Type I requirements.

This requirement doubles the material cost per square foot of water resistive barriers resulting in cost increases on the order of thousands of dollars on multifamily and commercial projects. Therefore, this code change significantly reduces the cost of construction by thousands of dollars on multifamily and commercial projects.

Cost Impact: The net effect of the public comment and code change proposal will decrease the cost of construction
This code change decreases costs.
Requiring materials to meet Type II requirements significantly increases costs relative to meeting Type I requirements. This requirement doubles the material cost per square foot of water resistive barriers resulting in cost increases on the order of thousands of dollars on multifamily and commercial projects. Therefore, this code change significantly reduces the cost of construction by thousands of dollars on multifamily and commercial projects.

Staff Analysis: Note: Both S194 and S196 deal with the same section in different ways. If both are approved, please ensure the final intentions are clear.

Public Comment# 1934

Public Comment 2:

Proponents:
Jay Crandell, P.E., ARES Consulting, representing Foam Sheathing Committee of the American Chemistry Council (jcrandell@aresconsulting.biz)

requests As Modified by Committee

Commenter's Reason: We request this proposal be approved in accordance with the Committee Action at the CAH.
The proponents testified that:

- The structure of this proposal is useful in that the more restrictive provisions are located in the main body of the text instead of placing these provisions in the exception statement.
- The problem to be solved is moisture performance issues with stucco in moist and marine climate zones. The reason for the problem is that dry climate zone installation techniques, currently required by code, are inadequate to reduce risk of moisture damage and do not provide for adequate drainage behind the stucco.
- The proposal breaks the requirements into a dry and moist/marine climate zone solution. There is a prescriptive solution – a 3/16" gap or drainage material and a performance solution which is a drainage efficiency requirement in accordance with ASTM standards.
- The water-resistive barrier requirements are retained as currently prescribed by code, and are specified in accordance with ASTM E2556, Type II, which has been in the code since 2006 and a part of ICC-ES AC-11 requirements beforehand.
- Opposition to the proposal supported the air gap and drainage plane, but also wanted to lower the WRB moisture performance by changing from a Type II down to a Type I. This was the intent of the other stucco proposals RB243/S194. This was not our original intent, as we have no supporting data or long-term performance studies to support this approach, either on a material or an assembly basis. Furthermore, the IBC-S committee ruled against S194 on lowering the WRB requirement, but both committees supported our proposals (RB242/S196) to add the air gap/drainage plane.

For the above reasons and to maintain consistency between the IRC and IBC, we therefore request your support of the committee action for approval of S196 as modified by committee.

Cost Impact: The net effect of the public comment and code change proposal will increase the cost of construction
Refer to the cost impact statement with the original S196 proposal. There is no change.

Public Comment# 1974

S200-19

IBC: 202 (New), 1609.1.1, 1613.1, 3103.1.1, 3103.5 (New), 3103.6 (New), ASCE Chapter 35 (New), ESTA Chapter 35 (New)

Proposed Change as Submitted

Proponents: Ali Fattah, City of San Diego, representing City of San Diego (afattah@sandiego.gov)

2018 International Building Code

Add new definition as follows:

TEMPORARY SPECIAL EVENT STRUCTURE. Any temporary ground-supported structure, platform, stage, stage scaffolding or rigging, canopy, tower supporting audio or visual effects equipment or similar structures.

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. The type of opening protection required, the basic design wind speed, *V*, 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 that 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. *Temporary special event structures* complying with Section 3103.5..5.

The wind speeds in Figures 1609.3(1) through 1609.3(8) are basic design wind speeds, *V*, and shall be converted in accordance with Section 1609.3.1 to *allowable stress design* wind speeds, V_{asd}, when the provisions of the standards referenced in Exceptions 4 and 5 are used.

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 Chapters 11, 12, 13, 15, 17 and 18 of ASCE 7, as applicable. The *seismic design category* for a structure is permitted to be determined in accordance with Section 1613 or ASCE 7.

Exceptions:

1. Detached one- and two-family dwellings, assigned to *Seismic Design Category* A, B or C, or located where the mapped short-period spectral response acceleration, S_S, is less than 0.4 g.
2. The seismic force-resisting system of wood-frame buildings that conform to the provisions of Section 2308 are not required to be analyzed as specified in this section.
3. Agricultural storage structures intended only for incidental human occupancy.
4. Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances and nuclear reactors.
5. References within ASCE 7 to Chapter 14 shall not apply, except as specifically required herein.
6. *Temporary special event structures* complying with Section 3103.5.

3103.1.1 Conformance. Temporary structures and uses shall conform to the structural strength, durability, fire safety, *means of egress*, accessibility, light, *ventilation* and sanitary requirements of this code as necessary to ensure public health, safety and general welfare.

Add new text as follows:

3103.5 Structural. The structural design for temporary structures shall comply with the requirements in Chapter 16. *Temporary special event structures* erected outdoors for a period of not more than six consecutive weeks shall be designed and erected to comply with requirements ESTA ANSI E1.21 as well as the lateral forces in ASCE 37.

3103.6 Durability and maintenance. A qualified person shall inspect *temporary special event structures*, including components, when purchased or acquired and at least once per year, based on the requirements in ESTA ANSI E1.21. Inspection records shall be kept and shall be made available for verification by the building official. Additionally, *temporary special event structures* shall be inspected at regular intervals when in service to ensure that the structure continues to perform as designed and initially erected.

ASCE/SEI

American Society of Civil EngineersStructural Engineering Institute
1801 Alexander Bell Drive
Reston VA 20191-4400

37-14: **Design Loads on Structures during Construction**

Add new standard(s) as follows:

ESTA

Entertainment Services and Technology Association
630 Ninth Avenue, Suite 609
New York NY 10036
US

ANSI E1.21—2013: **Entertainment Technology: Temporary Ground Supported Overhead Structures Used to Cover the Stage Areas and Support Equipment in the Production of Outdoor Entertainment Event**

Reason: Temporary Special Event Structures are regulated in Section 3105 of the International Fire Code and pose challenges to Building Officials and Fire Code Officials due to their temporary nature and methods of construction. The regular provisions of the IBC and IFC regulate permanent buildings and structures constructed to remain in service for long periods of time and as a consequence it is conceivable that over a 50 to 100 year services live that such buildings and structures can be expected to experience high wind and seismic. As a result when the duration of service is short for 6 weeks for example such as a sporting event, or one day such as in a concert, it is reasonable to assume that the probability of an event will not be high. Furthermore, wind events can be predicted fairly accurately to allow for adjustments or dismantling of temporarty structures when an installation may be subjected to winds higher than assumed in the design. As a consequence the entertainment industry developed "ANSI E1.21—2013: Entertainment Technology: Temporary Ground Supported Overhead Structures Used to Cover the Stage Areas and Support Equipment in the Production of Outdoor Entertainment Events" to specifically address the unique issues posed by temporary structures used as a part of special events in light of the duration of use and the reuse of components used to erect the structures. Additionally, these temporary structures may be erected with scaffolding systems that were intended for the construction of permanent buildings.
The definition for Temporary Special Event Structure is proposed to be modified to delete references limiting its applicaiton to the IFC.

IFC Section 3105 adopts by reference ANSI E1.21 so this code change merely harmonizes the two codes. It is noteworthy that ANSI E1.21 was last updated in 2013 and includes out of date references to 2010 edition of ASCE 7 as well as the 2002 edition of ASCE 37.

This code change also references ASCE 37-14 Design Loads on Structures during Construction since this standard is referenced in ANSI E1.21 and since by publishing it ASCE recognizes the need for reduced seismic loads adjusted by duration. It is worth noting that ASCE 37 intends to provide the same level of safety as the IBC does through ASCE 7.

Bibliography: ANSI E1.21—2013: Entertainment Technology: Temporary Ground Supported Overhead Structures Used to Cover the Stage Areas and Support Equipment in the Production of Outdoor Entertainment Events
ASCE 37-14 DEsign Loads on Structures During Construction

Cost Impact: The code change proposal will decrease the cost of construction
Building Official are requested on a regulart basis to accespt structural designs for concert stages and structures used in sporting events based on load reductions permitted in the two new referenced standards. As a consequnce of theis code change it is expected that ballast materials used to provide overturing and sliding resistance to be reduced. These standards are already in use in the motion picture and entertainment industry for work not specifically regulated by the Building Official.

Staff Analysis: A review of the standard proposed for inclusion in the code, ESTA ANSI E1.21-2013 and ASCE 37-14, with regard to the ICC criteria for referenced standards (Section 3.6 of CP#28) will be posted on the ICC website on or before April 2, 2019.

S200-19

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee agreed with the need for provisions relative to 'temporary special event structures'; however, the committee could not agree with a proposal that relied on ASCE 37 for temporary loads when the type of structure being considered is outside the scope of ASCE 37. ASCE representatives specifically testified that ASCE 37 is inappropriately being referenced in this proposal. The committee expressed concerns over 'who is responsible' and 'who would do the inspections'. (Vote: 13-1)

Assembly Action: **None**

S200-19

Individual Consideration Agenda

Public Comment 1:

IBC®: (New), 1609.1.1, 1613.1, 3103.1, 3103.5 (New), ANSI Chapter 35 (New), ASCE/SEI Chapter 35 (New), ESTA (New)

Proponents:
Ali Fattah, City of San Diego, representing City of San Diego (afattah@sandiego.gov)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

~~**TEMPORARY SPECIAL EVENT STRUCTURE.** Any temporary ground-supported structure, platform, stage, stage scaffolding or rigging, canopy, tower supporting audio or visual effects equipment or similar structures.~~

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. The type of opening protection required, the basic design wind speed, *V*, 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 that 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. ~~*Temporary special*~~ <u>*Special*</u> *event structures* complying with Section 3103.5..5.

The wind speeds in Figures 1609.3(1) through 1609.3(8) are basic design wind speeds, *V*, and shall be converted in accordance with Section 1609.3.1 to *allowable stress design* wind speeds, V_{asd}, when the provisions of the standards referenced in Exceptions 4 and 5 are used.

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 Chapters 11, 12, 13, 15, 17 and 18 of ASCE 7, as applicable. The *seismic design category* for a structure is permitted to be determined in accordance with Section 1613 or ASCE 7.

Exceptions:

1. Detached one- and two-family dwellings, assigned to *Seismic Design Category* A, B or C, or located where the mapped short-period spectral response acceleration, S_S, is less than 0.4 g.
2. The seismic force-resisting system of wood-frame buildings that conform to the provisions of Section 2308 are not required to be analyzed as specified in this section.
3. Agricultural storage structures intended only for incidental human occupancy.

4. Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances and nuclear reactors.

5. References within ASCE 7 to Chapter 14 shall not apply, except as specifically required herein.

6. ~~*Temporary special*~~ *Special event structures* complying with Section 3103.5.

3103.1 General. The provisions of Sections 3103.1 through ~~3103.4~~ 3103.5 shall apply to structures erected for a period of less than 180 days. Tents, umbrella structures and other membrane structures erected for a period of less than 180 days shall comply with the International Fire Code . Those erected for a longer period of time shall comply with applicable sections of this code.

3103.5 Structural. The structural design for temporary structures shall comply with the requirements in Chapter 16. ~~*Temporary special*~~ *Special event structures* erected outdoors for a period of not more than six consecutive weeks shall be designed and erected to comply with requirements ~~ESTA~~ ANSI E1.21 ~~as well as the lateral forces in ASCE 37~~.

ANSI

American National Standards Institute
25 West 43rd Street, Fourth Floor
New York NY 10036

ANSI E1.21—2013:
Entertainment Technology: Temporary Ground Supported Overhead Structures Used to Cover the Stage Areas and Support Equipment in the Production of Outdoor Entertainment Events

ASCE/SEI

American Society of Civil
EngineersStructural Engineering Institute
1801 Alexander Bell Drive
Reston VA 20191-4400

~~37-14: Design Loads on Structures during Construction~~

ESTA

Entertainment Services and Technology
Association
630 Ninth Avenue, Suite 609
New York NY 10036
US

~~ANSI E1.21—2013: Entertainment Technology: Temporary Ground Supported Overhead Structures Used to Cover the Stage Areas and Support Equipment in the Production of Outdoor Entertainment Event~~

Commenter's Reason: The original proposal has been simplified from what was originally submitted and remains true to the original goal to correlate the IBC and the IFC. The IFC references the IBC numerous times in IFC Ch 31 and intends to not duplicate requirements in the IBC. Additionally the Section 108.1 of the IBC authorized the Building Official to issue a permit for temporary structures and temporary uses but does not require it. However IFC Section 3105.5 requires that certain documents (structural plans and calculations) be submitted to the fire code official and the building official for review before a permit is approved.
The definition for temporary special event structure was deleted since the definition of special event structure was added to the 2018 IBC in code change G147-18.

The reference to ASCE 37 was also removed from the proposal in response to comments from ASCE during testimony at the CAH. It is interesting that ASCE's objections are due to safety personnel being present on construction sites and the lack of public access yet ASCE 37 can be used to shore bridges constructed over freeways, used to support construction scaffolding and equipment above public sidewalks and to shore up portions of occupied buildings undergoing structural renovations.

The ANSI E1.21—2013 is adopted in the 2018 IBC and includes loading requirements for wind and earthquake. And as frequently occurs with standards mismatches ASCE 7-10 is referenced in the ANSI standard as well as ASCE 37-02. While the original proposal was trying to correct the mismatch ASCE opposed the reference to ASCE 37 directly through the IBC.

During the CAH hearing we spent a considerable amount of time discussing the code change with questions raised by almost all the committee members. While the committee voted to disapprove the code change the committee was receptive to the need to treat temporary structures differently. We had many speakers in support from numerous jurisdictions. The only speakers in opposition were from ASCE regarding the the reference to ASCE 37. One comment that resonated is that Building Officials permit temporary structures to be supported on ground without a permanent concrete, masonry or wood foundation and ballast is permitted to provide for sliding resistance through friction and for wind uplift.

Supporting the original proposal were structural engineers representing firms that work on almost 75% of the temporary structures in the United

States and they are designing to what is being proposed including updates to the ANSI standard and ASCE 7.

The public comment has simplified the code change to better correlate with the fire code and the results of the group B code change. Please vote to support approval through public comment # 4 we need 2/3 of the governmental voting members to vote in the affirmative for the public comment to pass.

and the system shall be installed in accordance with NFPA 70. The emergency system provided shall have a minimum duration of 90 minutes when operated at full design demand.

3103.12.7 Means of egress illumination. *Means of egress* shall be illuminated with light having an intensity of not less than 1 foot-candle (11 lux) at floor level while the structure is occupied. Fixtures required for *means of egress* illumination shall be supplied from a separate circuit or source of power.

3103.12.8 Maintenance of means of egress. The required width of *exits*, *aisles* and passageways shall be maintained at all times to a *public way*. Guy wires, guy ropes and other support members shall not cross a *means of egress* at a height of less than 8 feet (2438 mm). The surface of *means of egress* shall be maintained in an *approved* manner.

SECTION 3104
TEMPORARY AND PERMANENT TENTS AND MEMBRANE STRUCTURES

3104.1 General. Tents and membrane structures, both temporary and permanent, shall be in accordance with this section and Sections 3106 and 3107. Permanent tents and membrane structures shall also comply with the *International Building Code*.

3104.2 Flame propagation performance treatment. Before a permit is granted, the *owner* or agent shall file with the *fire code official* a certificate executed by an *approved* testing laboratory. The certificate shall indicate that the floor coverings, tents, membrane structures and their appurtenances, which include sidewalls, drops and tarpaulins, are composed of materials meeting the flame propagation performance of Test Method 2 of NFPA 701. Additionally, it shall indicate that the bunting and combustible decorative materials and effects are composed of material meeting the flame propagation performance criteria of Test Method 1 or Test Method 2 of NFPA 701, as applicable. Alternatively, the materials shall be treated with a flame retardant in an *approved* manner and meet the flame propagation performance criteria of the applicable test method of NFPA 701. The flame propagation performance criteria shall be effective for the period specified by the permit.

3104.3 Label. Membrane structures or tents shall have a permanently affixed label bearing the identification of size and fabric or material type.

3104.4 Certification. An affidavit or affirmation shall be submitted to the *fire code official* and a copy retained on the premises on which the tent or air-supported structure is located. The affidavit shall attest to all of the following information relative to the flame propagation performance criteria of the fabric:

1. Names and address of the *owners* of the tent or air-supported structure.
2. Date the fabric was last treated with flame-retardant solution.
3. Trade name or kind of chemical used in treatment.
4. Name of person or firm treating the material.
5. Name of testing agency and test standard by which the fabric was tested.

*

SECTION 3105
TEMPORARY SPECIAL EVENT STRUCTURES

3105.1 General. Temporary special event structures shall comply with Section 3104, Sections 3105.2 through 3105.9 and ANSI E1.21.

3105.2 Approval. Temporary special event structures in excess of 400 square feet ($37 m^2$) shall not be erected, operated or maintained for any purpose without first obtaining approval and a permit from the *fire code official* and the building official.

3105.3 Permits. Permits shall be required as set forth in Sections 105.6 and 105.7.

3105.4 Use period. Temporary special event structures erected in accordance with ANSI E1.21 shall not be erected for a period of more than six consecutive weeks.

3105.5 Required documents. The following documents shall be submitted to the *fire code official* and the building official for review before a permit is *approved*:

1. Construction documents: *Construction documents* shall be prepared by a *registered design professional* in accordance with the *International Building Code* and ANSI E1.21 where applicable. *Construction documents* shall include:
   - 1.1. A summary sheet showing the building code used, design criteria, loads and support reactions.
   - 1.2. Detailed construction and installation drawings.
   - 1.3. Design calculations.
   - 1.4. Operating limits of the structure explicitly outlined by the *registered design professional* including environmental conditions and physical forces.
   - 1.5. Effects of additive elements such as video walls, supported scenery, audio equipment, vertical and horizontal coverings.
   - 1.6. Means for adequate stability including specific requirements for guying and cross-bracing, ground anchors or ballast for different ground conditions.
2. Designation of responsible party: The *owner* of the temporary special event structure shall designate in writing a person to have responsibility for the temporary special event structure on the site. The designated person shall have sufficient knowledge of the construction documents, manufacturer's recommendations and operations plan to make judgments regarding the structure's safety and to coordinate with the *fire code official*.

G147-18

IBC: SECTION 202, 202 (New), 3103.1

Proponent: Richard Nix, representing Entertainment Services & Technology Association/Event Safety Alliance (rnix@zoomtown.com)

2018 International Building Code

SECTION 202 DEFINITIONS

Add new definition as follows:

SPECIAL EVENT STRUCTURE. Any ground-supported structure, platform, stage, stage scaffolding or rigging, canopy, tower or similar structure supporting entertainment-related equipment or signage.

Revise as follows:

3103.1 General. The provisions of Sections 3103.1 through 3103.4 shall apply to structures erected for a period of less than 180 days. ~~Tents~~Special event structures, tents, umbrella structures and other membrane structures erected for a period of less than 180 days shall also comply with the International Fire Code. ~~Those erected for a longer period of time shall comply with applicable sections of this code.~~

Reason:
These structures are covered under the scope of IBC Chapter 31, Special Construction. IBC Section 3103 addresses installations <180 days, which are considered "temporary". Temporary tents, any type of membrane covered structure and special events structures are therefore within the scope of IBC Chapter 31, section 3102 and/or section 3103. All of these structures except special event structures are referred to IFC Chapter 31. The IFC has added new requirements in Chapter 31 for special events structures, therefore special event structures must also be referred from IBC Chapter 31 to IFC Chapter 31.

Building Code Officials and others using the IBC as a primary code reference require the proper guidance and direction to IFC Chapter 31. In the last code change cycle, F308-16 replaced the IFC term temporary stage canopy with the term temporary special events structure. Therefore, Temporary special events structures are now covered under the purview of IFC Chapter 31. Coordination with IBC Chapter 31 was both implied and intended to occur as a result of F308-16, due to the special construction and temporary characteristics of these structures. However, that coordination did not occur. This CCP ensures proper coordination between IFC and IBC as intended in the last code change cycle.

This proposed definition for Special Events Structures in IBC is slightly different than that used in IFC, because the word "temporary" is implied by the corresponding IBC section 3103, where these structures are currently mentioned.

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

The proposed change adds a definition for clarity along with a pointer for code coordination.

Internal ID: 311

G325

Committee Action: **Disapproved**

Committee Reason: This requirement is very limiting and doesn't allow for the alternatives that are regularly applied in the field. The language in the 2018 code extends these requirements to occupant elevators. In buildings with fire service elevators, the owners often would like them to serve multiple uses and we cannot preclude the idea that there will be other occupants in them, which drags accessibility into the mix. (Vote: 14-0)

Assembly Motion: **NONE**

G145-18

Committee Action: **Disapproved**

Committee Reason: This provision for elevator hoistway pressurization are now voluntary in lieu of providing elevator lobbies. Putting these requirements in the smoke proof enclosure requirements would create confusion. The code is clear right now regarding what protection is required outside of elevator hoistways and machine rooms and those areas are already protected. This would lead to subjective application of the code. (Vote: 13-1)

Assembly Motion: **NONE**

G146-18

Committee Action: **Approved as Submitted**

Committee Reason: This is a necessary change for correlation based on previous committee actions on the tall wood provisions. (Vote: 14-0)

Assembly Motion: **NONE**

G147-18

Committee Action: **Approved as Modified**

Committee Modification:

3103.1 General. The provisions of Sections 3103.1 through 3103.4 shall apply to structures erected for a period of less than 180 days. ~~Tents~~Special event structures, tents, umbrella structures and other membrane structures erected for a period of less than 180 days shall also comply with the International Fire Code. Those erected for a longer period of time shall comply with applicable sections of this code.

Committee Reason: The committee approved the modification as it alleviated concerns regarding temporary structures. The committee approved the proposal based on the proponent's reason statement. (Vote: 14-0)

Assembly Motion: **NONE**

G148-18

Committee Action: **Approved as Submitted**

Committee Reason: Based on the proponents published reason and the fact that this proposal clarifies the code. (Vote: 13-1)

Assembly Motion: **NONE**

G149-18

Committee Action: **Disapproved**

Committee Reason: This proposal has some merit, but the language is too loose. "Public" could mean any building that is considered public in the Americans with Disabilities Act. "Governmental entities" may be a better term. (Vote: 9-

Bibliography: The ANSI standard is already referenced in the IFC and is widely available.

Cost Impact: The net effect of the public comment and code change proposal will not increase or decrease the cost of construction This code change merely correlate standards and reflects the State of the practice.

Public Comment# 1294

Public Comment 2:

IBC®: (New), 1609.1.1, 1613.1, 3103.1, 3103.1.1, 3103.5 (New), 3103.6 (New), ASCE/SEI Chapter 35 (New), ESTA (New)

Proponents:

Richard Nix, representing Entertainment Servises and Technology Association (ESTA), and Event Safety Alliance (ESA) (rnix@zoomtown.com)

requests As Modified by Public Comment

Modify as follows:

2018 International Building Code

~~**TEMPORARY SPECIAL EVENT STRUCTURE.** Any temporary ground-supported structure, platform, stage, stage scaffolding or rigging, canopy, tower supporting audio or visual effects equipment or similar structures.~~

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. The type of opening protection required, the basic design wind speed, *V*, 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 that 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. ~~*Temporary special*~~ *Special event structures* complying with Section 3103.5~~..5.~~

The wind speeds in Figures 1609.3(1) through 1609.3(8) are basic design wind speeds, *V*, and shall be converted in accordance with Section 1609.3.1 to *allowable stress design* wind speeds, V_{asd}, when the provisions of the standards referenced in Exceptions 4 and 5 are used.

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 Chapters 11, 12, 13, 15, 17 and 18 of ASCE 7, as applicable. The *seismic design category* for a structure is permitted to be determined in accordance with Section 1613 or ASCE 7.

Exceptions:

1. Detached one- and two-family dwellings, assigned to *Seismic Design Category* A, B or C, or located where the mapped short-period spectral response acceleration, S_S, is less than 0.4 g.
2. The seismic force-resisting system of wood-frame buildings that conform to the provisions of Section 2308 are not required to be analyzed as specified in this section.
3. Agricultural storage structures intended only for incidental human occupancy.
4. Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances and nuclear reactors.
5. References within ASCE 7 to Chapter 14 shall not apply, except as specifically required herein.
6. ~~*Temporary special*~~ *Special event structures* complying with Section 3103.5.

3103.1 General. The provisions of Sections 3103.1 through ~~3103.4~~ 3103.5 shall apply to structures erected for a period of less than 180 days. Tents, umbrella structures and other membrane structures erected for a period of less than 180 days shall comply with the International Fire Code . Those erected for a longer period of time shall comply with applicable sections of this code.

3103.1.1 Conformance. Temporary structures and uses shall conform to the structural strength, ~~durability,~~ fire safety, *means of egress*, accessibility, light, *ventilation* and sanitary requirements of this code as necessary to ensure public health, safety and general welfare.

3103.5 Structural. The structural design for temporary structures shall comply with the requirements in Chapter 16. ~~*Temporary special*~~ *Special event structures* erected outdoors for a period of not more than six consecutive weeks shall be designed and erected to comply with requirements of ~~ESTA~~ ANSI E1.21 ~~as well as the lateral forces in ASCE 37~~.

~~**3103.6 Durability and maintenance.** A qualified person shall inspect *temporary special event structures*, including components, when purchased or acquired and at least once per year, based on the requirements in ESTA ANSI E1.21. Inspection records shall be kept and shall be made available for verification by the building official. Additionally, *temporary special event structures* shall be inspected at regular intervals when in service to ensure that the structure continues to perform as designed and initially erected.~~

ASCE/SEI

American Society of Civil EngineersStructural Engineering Institute
1801 Alexander Bell Drive
Reston VA 20191-4400

~~37-14: Design Loads on Structures during Construction~~

ESTA

Entertainment Services and Technology Association
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New York NY 10036
US

ANSI E1.21—2013: ***Entertainment Technology: Temporary ~~Ground Supported Overhead~~ Structures Used ~~to Cover the Stage Areas and Support Equipment in the~~ for Technical Production of Outdoor Entertainment Event s.***

Commenter's Reason: 1. The proposed definition is not necessary because a new definition for "Special Event Structure" has already been approved for inclusion in the IBC as a result of CCP G147-18.

2. Revise the proposed change to 1609.1.1 exception 7 to use the defined term as per CCP G147-18, and to correct an editorial reference to the newly proposed section 3103.5

3. Revise the proposed change to 1613.1 exception 6 to use the defined term as per CCP G147-18.
4. Revise existing wording in section 3103.1 to be inclusive of the newly proposed section 3103.5
5. Revise the proposal such that NO CHANGE is made to the existing wording of section 3103.1.1
6. Revise the wording of the proposed new section 3103.5 to use the defined term approved as a result of CCP G147-18, and to remove the proposed reference to ASCE 37-14.
7. Delete the proposed section 3103.6 from the proposal. It is not necessary. Inspections are addressed in ANSI E1.21.
8. Delete the proposed reference to ASCE/SEI 37-14.

The single objection made during CAH indicated that ASCE 37-14 is not intended to be used for temporary structures, yet it is interesting to note that its scope and associated commentary for Chapter 6 are clear that it is for use with temporary structures, as shown in the ASCE 37-14 commentary text:

This section deals with special issues of construction ***and temporary structures for which the basic procedures of ASCE 7-10 (ASCE/SEI 2010) are to be modified.***
The environmental loads in this chapter are reduced from those in ASCE 7-10 in recognition of the anticipated lifespan of temporary structures *and temporary configurations of structures under construction. Reduction to the safety of individuals is not the intent of the committee.*
Reductions of loads to the levels stated in this standard ***are appropriate when loading situations can be managed through safety protocols*** *that limit access to hazardous locations when loadings exceed those used for temporary designs, and when loadings, including environmental loadings, can be limited (e.g., by timely snow removal) proactively. The knowledge and training of personnel in control of construction sites, the visible nature of construction elements, and the processes on construction sites are key components of protocols necessary to control of risk to personnel and property on the construction site. Risks to personnel and property adjacent to the construction site also warrant attention.*

There is a clear distinction between "construction" and "temporary structures", though the principles used to support load reductions based on duration of exposure are exactly the same. It is also interesting to note that the structures covered by ASCE 37-14 include scaffold and shoring structures, such as those used during construction of buildings and renovation of existing buildings, where they might be built over sidewalk areas that are accessible to the public. Arguments made during the CAH included assertions that ASCE 37-14 is to be used only when personnel are trained, which is also clearly addressed in ANSI E1.21 as part of its Operations Management Plan requirements. This comment asks to remove that reference from the proposal in order to alleviate that concern.

9. Revise the title of the referenced ANSI Standard as indicated. The ANSI designation is correct, but the title has been incorrectly transcribed into the ICC references. The standard (ANSI E1.21-2013) is an approved reference. (see staff note)

General commentary:
This proposal satisfies a stated ICC goal to attain better correlation between IFC and IBC. These structures have been important topics for the last two code cycles, as evidenced by the major overhaul of IFC Chapter 31 to accommodate special events and their associated structures. However, the IFC cannot adequately address structural requirements, so the simple changes requested by the proposal are necessary to better achieve overall coverage. Guidance for these structures has - until now - been essentially non-existent, so any review and approval under code purview can only be accomplished using the "Alternative Methods" approach, which drastically increases the amount of time, paperwork and resources necessary to validate conformance. This proposal reduces the amount of time and resources required to properly review and approve the structures, by introducing a recognized reference standard, ANSI E1.21, that has been an approved, consensus developed standard since 2006. It contains structural design requirements, it has been, and is now, widely used in the structural engineering community, among those who perform work on these types of structures. Many of the engineers who developed and use this standard also testified in support of the proposal at the CAH.

Questions arose during CAH testimony regarding inspections. The intent is to require inspections, however the further intent is to allow the code official the flexibility to designate who is authorized to perfrom such inspections. In cases where the jurisdiction has such resources, they would perform the inspection as usual, but in jurisdictions where they do not have such resources it is acceptable and reasonable to designate a 3rd-party inspector. This is addressed in ANSI E1.21. Questions also arose during CAH regarding "who is responsible". ANSI E1.21 is explicit about the responsibilities of the "designated person", who must meet the following criteria in accordance with the requirements of ANSI E1.21:

- *...has overall responsibility on-site for the temporary structure;*
- *...shall have knowledge of the engineering documentation for the temporary structure’s components and configurations in use;*
- *...shall develop a risk assessment plan for each use, and shall provide instruction for the safe erection, use and dismantling of the temporary structure.*

- *...shall prepare layout drawings consistent with the engineering documentation.*

It was clear from the number and types of questions asked during the CAH that the committee was generally receptive to the proposal's intent of treating temporary structures for special events differently, and agreed that there was a deficiency in the code relating to the proposal's subject matter. Of greater importance is the reality that the number and size of special events in general - particularly those with structures - is growing significantly. The impact of this growth is clearly recognized in the current IFC's expansion of its Chapter 31 scope and coverage.
For Reference CCP G147:

- IBC Definitions - Temporary Special Event structures IBC: 3103.1, 202 (New) Proponent: Richard Nix, representing Entertainment Services & Technology Association/Event Safety Alliance (rnix@zoomtown.com) 2018 International Building Code 3103.1 General. The provisions of Sections 3103.1 through 3103.4 shall apply to structures erected for a period of less than 180 days. TentsSpecial event structures, tents, umbrella structures and other membrane structures erected for a period of less than 180 days shall also comply with the International Fire Code. Those erected for a longer period of time shall comply with applicable sections of this code. Add new definition as follows: SPECIAL EVENT STRUCTURE. Any ground-supported structure, platform, stage, stage scaffolding or rigging, canopy, tower or similar structure supporting entertainmentrelated equipment or signage. Reason: These structures are covered under the scope of IBC Chapter 31, Special Construction. IBC Section 3103 addresses installations <180 days, which are considered "temporary". Temporary tents, any type of membrane covered structure and special events structures are therefore within the scope of IBC Chapter 31, section 3102 and/or section 3103. All of these structures except special event structures are referred to IFC Chapter 31. The IFC has added new requirements in Chapter 31 for special events structures, therefore special event structures must also be referred from IBC Chapter 31 to IFC Chapter 31. Building Code Officials and others using the IBC as a primary code reference require the proper guidance and direction to IFC Chapter 31. In the last code change cycle, F308-16 replaced the IFC term temporary stage canopy with the term temporary special events structure. Therefore, Temporary special events structures are now covered under the purview of IFC Chapter 31. Coordination with IBC Chapter 31 was both implied and intended to occur as a result of F308-16, due to the special construction and temporary characteristics of these structures. However, that coordination did not occur. This CCP ensures proper coordination between IFC and IBC as intended in the last code change cycle. This proposed definition for Special Events Structures in IBC is slightly different than that used in IFC, because the word "temporary" is implied by the corresponding IBC section 3103, where these structures are currently mentioned. Cost Impact The code change proposal will not increase or decrease the cost of construction. The proposed change adds a definition for clarity along with a pointer for code coordination.

Please support the change proposal and this public review comment.

Bibliography: ANSI E1.21 is already an approved reference standard in the IFC. It is available for free download at
https://tsp.esta.org/tsp/documents/published_docs.php

Cost Impact: The net effect of the public comment and code change proposal will decrease the cost of construction
This proposal decreases the cost of construction by reducing or eliminating the need for additional time and resources necessary to validate an Alternate Methods design approach, by providing clear guidance to appropriate reference standards.

Public Comment# 2092