IBC — Structural

2019 GROUP B PROPOSED CHANGES TO THE I-CODES
ALBUQUERQUE COMMITTEE ACTION HEARINGS

April 28 - May 8, 2019
Albuquerque Convention Center, Albuquerque, NM
Constandino (Gus) Sirakis, PE, Chair
Assistant Commissioner of Technical Affairs &
Code Development
New York City Department of Buildings
New York, NY

Edward Lisinski, PE, Vice Chair
Director, Department of Building Inspection &
Neighborhood Services
City of West Allis
West Allis, WI

Ronald Brendel, PE
Sr. Plan Reviewer/Code Development Specialist
City of Saint Louis, MO
St. Louis, MO

David D. Chang, PE, SE
Senior Structural Engineer
Building and Safety Department,
City of Los Angeles
Alta Loma, CA

Wanda D. Edwards, PE
Senior Director of Technical Services
RCI Inc.
Raleigh, NC

Cornelia Orzescu
Plans Examiner
Town of Parker
Parker, CO

Larry Anthony Paul, AIA
Principal Architect
L. A. Paul Associates
San Rafael, CA

Anne Payne
Senior Building Inspector
City of Poquoson
Poquoson, VA

Jay Richards, RA
Assistant Architect Administrator
State of Ohio-Board of Building Standards
Reynoldsburg, OH

Dwight “Sonny” M. Richardson, Jr., LTC, EN
Rep: NAHB
President
Richardson Home Builders Inc.
Tuscaloosa, AL

Gwenyth R. Searer, PE, SE
Principal
Wiss, Janney, Elstner Associates, Inc.
Emeryville, CA

Saul Shapiro, PE
Vice President
Langan Engineering
New York, NY

Jonathan C. Siu, PE, SE
Principal Engineer/Building Official
City of Seattle, Department of Construction and
Inspections
Seattle, WA

Paul A. Turner, AIA
Principal Architect
Stewart, Schaberg & Turner/Architects LLC
St. Louis, MO

Howard Zee, PE, SE
Structural Engineer
City & County of San Francisco,
Department of Building Inspection
San Francisco, CA

Staff Secretariats:
Lawrence C. Novak, SE, F.SEI, CERT, LEED AP
Chief Structural Engineer
Codes and Standards Development
International Code Council
Central Regional Office
County Club Hills, IL
The following is the tentative order in which the proposed changes to the code will be discussed at the public hearings. Proposed changes which impact the same subject have been grouped to permit consideration in consecutive changes.

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

| EB2-19 | EB154-19 | G17-19 | S35-19 |
| EB15-19 | EB155-19 | G18-19 | S36-19 |
| EB16-19 | EB156-19 | G19-19 | S37-19 |
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| EB150-19 | G13-19 | S31-19 | S69-19 |
| EB151-19 | G14-19 | S32-19 | S70-19 |
| EB152-19 | G15-19 | S33-19 Part I | S71-19 |
| EB153-19 | G16-19 | S34-19 Part I | S72-19 |
S1-19

IBC: 1511.1 (IEBC 705.1)

Proponent: 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 and meet the requirements of Section 1608.3.

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 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.

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.

Proposal # 5211
S2-19
IBC: 1511.1 (IEBC 705.1)

Proponent: 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

[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 1/4" 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

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.

Proposal # 5234

S2-19
IBC: 1511.6.1 (New)

Proponent: 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.
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.

Proposal # 2100
S5-19

IBC: 1511.3 (IEBC 705.3)

Proponent: 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 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.

   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.

Proposal # 5588

S5-19
2018 International Building Code

SECTION 1511
REROOFING

Revise as follows:

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

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

Reason: This code proposal is for clarification that when there are two or more roof coverings, a new roof covering cannot be installed until the coverings are removed to the roof deck. Often, the contractor does not remove coverings down to the deck and this will remind the contractor that it is required.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. This code proposal is a clarification of the current code requirements and will not affect the cost of construction.
2018 International Building Code

SECTION 1511
REROOFING

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

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

Revise as follows:

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

1. Where the existing roof or roof covering is water soaked or has deteriorated to the point that the existing roof or roof covering is not adequate as a base for additional roofing.
2. Where the existing roof covering is slate, clay, cement or asbestos-cement tile.
3. Where the existing roof has two or more applications of any type of roof covering.
4. Where the existing roof covering is wood shakes or shingles and the roof covering was not installed in accordance with Section 1511.4.

1511.4 Roof recovering. Where the application of a new roof covering over wood shingle or shake roofs creates a combustible concealed space, the entire existing surface shall be covered with gypsum board, mineral fiber, glass fiber or other approved materials securely fastened in place. The installation of a new roof covering over wood shakes or shingles shall require the entire existing surface be covered with gypsum board or other approved rigid materials to provide for secure fastening.

Reason: Most manufacturers recommend the installation of a rigid decking material over the wood shakes or shingles to provide a solid surface for the securement of the new roof cover. The roof is being recovered because of deterioration of the wood shake or shingles and may be rotten and unable to provide a solid surface for fasteners to maintain attachment. Without a rigid deck and rotten or decayed shakes or shingles the fasteners will not keep the new roof covering attached. The installation of rigid deck also prevents seeing undulations in the new roof covering.

Cost Impact: The code change proposal will increase the cost of construction. There may be some increase in the cost of construction if the manufacturers installation instructions do not require the installation of a rigid decking material over the wood shakes or shingles. If the manufacturers installation instructions require the rigid decking material there is no increase in cost.
S8-19

IBC: SECTION 1511 (IEBC 705), 1511.1 (IEBC 705.1)

Proponent: Mark Graham, representing National Roofing Contractors Association (NRCA) (mgraham@nrca.net)

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, this section and Sections 1503 through 1509.

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: This code change proposal is intended to clarify the code's intent regarding reroofing, including roof re-covering and roof replacement. A reroofing project is not intended to require the need to upgrade any rooftop structures (Section 1510-Rooftop Structures) to the edition of the code that is current at the time of reroofing. A literal interpretation of the code's current requirement in Section 1511.1-General can be interpreted to require any rooftop structures to be upgraded when reroofing.

Similarly, a reroofing project is not intended to require the need to upgrade the roof area's roof drainage (Section 1502-Roof Drainage) to the edition of the code that is current at the time of reroofing. This is already addressed, in part, in Section 1511.1, Exception 2.

Limiting the sections of Chapter 15 applicable to reroofing addresses these issues and allows for eliminating Section 1511.1's Exception 2.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This code change proposal clarifies the code's intent; it is not intended to increase or decrease the stringency of the 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.

Exception:

- 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.

- 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.


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.
2018 International Building Code

Revise as follows:

1511.5 Reinstallation of materials. Existing slate, clay or cement tile shall be permitted for reinstallation, except that damaged, cracked or broken slate or tile shall not be reinstalled. Existing vent flashing, metal edgings, drain outlets, collars and metal counterflashings shall not be reinstalled where rusted, damaged or deteriorated. Aggregate Existing ballast that is damaged, cracked or broken shall not be reinstalled. Existing aggregate surfacing materials from built-up roofs shall not be reinstalled.

Reason: This proposal is intended to clarify the intent of the code.
Small diameter aggregate, such as that used as surfacing on built-up roof membranes, is generally considered not appropriate for re-use because the aggregate is contaminated with the existing roof's bitumen flood coat; this is already addressed in the last sentence of Sec. 1511.5. However, it is recognized in the roof industry existing aggregate ballast and pavers, such as that used on ballasted single-ply membrane roof systems, is appropriate for re-use, provided the pavers are not damaged, cracked or broken. Since the code's current language prohibiting the re-use of aggregate surfacing can be interpreted as also applying to aggregate and paver ballast, aggregate and paver ballast is sometimes disposed of unnecessarily.

This proposal is intended to provide differentiation between aggregate and paver ballast, and aggregate surfacing using the code's already existing terminology and is intended to eliminate the need for unnecessarily disposing of roof ballast materials.

Cost Impact: The code change proposal will decrease the cost of construction
In situations where existing aggreate or paver ballast is re-used, the material cost of the aggregate or paver ballast is saved.
2018 International Building Code

Revise as follows:

1402.2 Weather protection. Exterior walls shall provide the building with a weather-resistant exterior wall envelope. The exterior wall envelope shall include flashing, as described in Section 1404.4. The exterior wall envelope shall be designed and constructed in such a manner as to prevent the accumulation of water within the wall assembly by providing a water-resistive barrier behind the exterior veneer, as described in Section 1403.2, and a means for draining water that enters the assembly to the exterior. Protection against condensation in the exterior wall assembly shall be provided in accordance with Section 1404.3.

Exceptions:

1. A weather-resistant exterior wall envelope shall not be required over concrete or masonry walls designed in accordance with Chapters 19 and 21, respectively.
2. Compliance with the requirements for a means of drainage, and the requirements of Sections 1403.2 and 1404.4, shall not be required for an exterior wall envelope that has been demonstrated through testing to resist wind-driven rain, including joints, penetrations and intersections with dissimilar materials, in accordance with ASTM E331 under the following conditions:
   2.1. Exterior wall envelope test assemblies shall include not fewer than one opening, one control joint, one wall/eave interface and one wall sill. Tested openings and penetrations shall be representative of the intended end-use configuration.
   2.2. Exterior wall envelope test assemblies shall be not less than 4 feet by 8 feet (1219 mm by 2438 mm) in size.
   2.3. Exterior wall envelope assemblies shall be tested at a minimum differential pressure of 6.24 pounds per square foot (psf) (0.297 kN/m²).
   2.4. Exterior wall envelope assemblies shall be subjected to a minimum test exposure duration of 2 hours.

The exterior wall envelope design shall be considered to resist wind-driven rain where the results of testing indicate that water did not penetrate control joints in the exterior wall envelope, joints at the perimeter of openings or intersections of terminations with dissimilar materials. Special Inspections of the weather-resistant exterior wall envelope shall comply with 1402.3.

3. Exterior insulation and finish systems (EIFS) complying with Section 1407.4.1.

Add new text as follows:

1402.3 Special inspections. Special Inspections of the weather-resistant exterior wall envelope shall be as required in Chapter 17.

1705.16 Weather-resistant exterior wall envelope. Special inspections and tests shall be based on the weather-resistant exterior wall envelope design as designated in the approved construction documents.

Reason: The weather-resistant exterior wall envelope is a site-assembled system that is often concealed when construction is complete. To repair or replace poor initial installation or workmanship of the water-resistive barrier assembly (1403.2) and flashing (1404.4) often requires extensive disassembly and replacement of the surrounding exterior cladding, fenestration and structural components.

This proposal allows the AHJ to designate a Special inspection for the critical water management building system when the design complexity calls for additional inspections through the construction documents.

Cost Impact: The code change proposal will increase the cost of construction.

Cost will only be increased where the design complexity in the construction documents calls for it. This is the condition where special inspection of the concealed system is critical.
S12-19

IBC: 1503.3, 1503.3.1 (New), 1503.3.2 (New)

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

2018 International Building Code

Revise as follows:

1503.3 Coping. Parapet walls. Parapet walls shall be properly coped with noncombustible, weatherproof materials of a width not less than the thickness of the parapet wall; coped or covered in accordance with Sections 1503.3.1 and 1503.3.2. The top surface of the parapet wall shall provide positive drainage.

Add new text as follows:

1503.3.1 Fire-resistance-rated parapet walls. Parapet walls required by Section 705.11 shall be coped or covered with non-combustible, weatherproof materials of a width not less than the thickness of the parapet wall.

Revise as follows:

1503.3.2 Other parapet walls. Parapet walls meeting one of the exceptions in Section 705.11 shall be coped or covered with weatherproof materials of a width not less than the thickness of the parapet wall.

Reason: The current language in this section is in dire need of an update, as it does not address current technologies or practices. This language is a carry over from the legacy code and was meant to apply to the coping of masonry parapet walls. The use of the word coping is also confusing, as it is often used interchangeably with the word covered. Depending on the type of roofing system that is being used, traditional metal or masonry copings are not always used to cap or cover a parapet wall.

This proposal provides the much needed clarity as to when and how parapet walls are to be properly coped or covered. The requirement has been broken out into 2 subsections for the two different parapet wall types. 1503.3.1 is for parapet walls that are required to comply with 705.11 must be coped or covered with weatherproof and noncombustible materials.

1503.3.2 is for parapet walls that do not have to comply with 705.11, are required to be coped or covered with weatherproof materials.

This revision will provide additional options for maintaining a continuous air barrier. For example, the roof membrane could be used to wrap the top of the parapet wall and extend down the exterior side of the wall. The membrane could then be tied into the wall air barrier system. See also Figures 1 through 4.

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 the BCAC website at: https://www.iccsafe.org/codes-tech-support/codes/code-development-process/building-code-action-committee-bcac/.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. No additional materials or detailing will be required based on this code change proposal; therefore it will not increase the cost of construction.

Proposal # 4127

S12-19
2018 International Building Code

Revise as follows:

1503.3 Coping, Parapet Walls. Parapet walls shall be properly coped with non-combustible, weatherproof materials of a width not less than the thickness of the parapet wall, or covered in accordance with Sections 1503.3.1 and 1503.3.2. The top surface of the parapet wall shall provide positive drainage.

Add new text as follows:

1503.3.1 Fire-resistance-rated parapet walls. Parapet walls required by section 705.11 shall be coped or covered with non-combustible, weatherproof materials of a width not less than the thickness of the parapet wall.

1503.3.2 Other parapet walls. Parapet walls meeting one of the exceptions in Section 705.11 shall be coped or covered with weatherproof materials of a width not less than the thickness of the parapet wall.

Reason: This proposal clarifies how to properly cope or cover the two different types of parapet wall types (those that must comply with Section 705.11 and those that do not).

The current language does not address current technologies or practices. This language is a carry over from the legacy code and was meant to apply to the coping of masonry parapet walls. The use of the word coping is also confusing, as it is often used interchangeably with the word covered. Depending on the type of roofing system that is being used, traditional metal or masonry copings are not always used to cap or cover a parapet wall.

This revision will provide additional options for maintaining a continuous air barrier. For example, the roof membrane could be used to wrap the top of the parapet wall and extend down the exterior side of the wall. The membrane could then be tied into the wall air barrier system.
Examples of covered parapets as required by 1503.3.2.

Examples of copped parapets as required by 1503.3.1.
Fascia on 6” wide by 4” high “Parapet”

Adelman Travel - Fascia on radius “parapet” @ 6” high x 6” wide

Hyvee Iowa Fascia on 18” parapet condition
Cost Impact: The code change proposal will decrease the cost of construction
The code change proposal will decrease the cost of construction. This proposal clarifies the difference between parapet wall types and how they should be covered or coped. Where metal coping is not required this proposal would lead to a decrease in the cost of construction by reducing material and labor. This could result in a cost reduction as much as $5-10 per foot.
S14-19

IBC: 1504.2.1, 1504.2.1.2, 1504.2.1.3 (New), ASTM Chapter 35

Proponent: Rob Brooks, Rob Brooks and Associates, LLC, representing DowDuPont (rob@rtbrooks.com)

2018 International Building Code

Revise as follows:

1504.2 Wind resistance of clay and concrete tile. Wind loads on clay and concrete tile roof coverings shall be in accordance with Section 1609.5.

1504.2.1 Testing. Testing of concrete and clay roof tiles shall be in accordance with Sections 1504.2.1.1, 1504.2.1.2 and 1504.2.1.3.

1504.2.1.1 Overtopping resistance. Concrete and clay roof tiles shall be tested to determine their resistance to overturning due to wind in accordance with Chapter 15 and either SBCCI SSTD 11 or ASTM C1568.

1504.2.1.2 Wind tunnel testing. Where concrete and clay roof tiles do not satisfy the limitations in Chapter 16 for rigid tile, a wind tunnel test shall be used to determine the wind characteristics of the concrete or clay tile roof covering in accordance with Chapter 15 and either SBCCI SSTD 11 or ASTM C1569.

Add new text as follows:

1504.2.1.3 Air permeability testing. The lift coefficient for concrete and clay tile shall be 0.2 or shall be determined in accordance with SBCCI SSTD 11 or ASTM C1570.

Add new standard(s) as follows:

ASTM


ASTM


Reason: Reason: In 2003, ASTM International Subcommittee C15.06 replicated SSTD 11-99 by subdividing the SBCCI standard into three different ASTM standards:

1) ASTM C1568-03, Standard Test Method for Wind Resistance of Concrete and Clay Roof Tiles (Mechanical Uplift Resistance Method),

2) ASTM C1569-03, Standard Test Method for Wind Resistance of Concrete and Clay Roof Tiles (Wind Tunnel Method), and


In the previous code cycle, ASTM C1568 for mechanical uplift resistance was added to Section 1504.2.1.1 as an alternate to SSTD 11-99. This code change adds ASTM C1569 to Section 1504.2.1.2 for wind tunnel testing.

The ASTM C1569 test method determines the uplift forces acting as a result of the simulated wind when tiles are attached to a section a roof deck in accordance with the manufacturer's instructions.

The cross-correlation of ASTM C1569 and SSTD 11 is as follows:

C1569 Section 5 relates to SSTD 11 Section 801
C1569 Section 7.2 relates to SSTD 11 Section 802
C1569 Section 7.5 relates to SSTD 11 Section 803
C1569 Section 7.6 relates to SSTD 11 Section 804
C1569 Section 7.7 relates to SSTD 11 Section 805
This code change also adds ASTM C1570 to Section 1504.2.1.3 for air permeability testing.

The ASTM C1570 test method measures the ability of the roof system to relieve wind-induced uplift pressures as a result of the overall air permeability of the roof assembly as it relates to the resistance of the roof system to damage induced by the wind. It serves to evaluate the uplift coefficient CL, referenced in IBC Section 1609.5.3, Equation 16-34, where the lift coefficient determination states: The lift coefficient for concrete and clay tile shall be 0.2 or shall be determined by test in accordance with Section 1504.2.1. That pointer has been modified to Section 1504.2.1.3.

The cross-correlation of ASTM C1570 and SSTD 11 is as follows:

C1570 Section 1.2 relates to SSTD 11 Section 901
C1570 Section 4.1 relates to SSTD 11 Section 902.1
C1570 Section 6 relates to SSTD 11 Section 902.2
C1570 Section 7 relates to SSTD 11 Section 902.3
C1570 Section 8 relates to SSTD 11 Section 902.4
C1570 Section 9 relates to SSTD 11 Section 902.5
C1570 Section 10 relates to SSTD 11 Section 902.6
C1570 Section 11 relates to SSTD 11 Section 902.7
C1570 Section 12 relates to SSTD 11 Section 903
C1570 Section 13 relates to SSTD 11 Section 904

There are no technical changes proposed with this code change request. ASTM C1569 and C1570 are simply a duplication of the relevant sections of SSTD 11-99 with regard to the wind tunnel and air-permeability test methods. This modification now references ASTM consensus standards that will have the capability to be updated in the future, as SSTD 11 has not been updated since 1999.

The chronology of the progression of these standardized test methods is found in Table 1 at [https://www.researchgate.net/publication/299487049_A_study_of_wind_load_interaction_for_roofing_field_tiles]
**Cost Impact:** The code change proposal will not increase or decrease the cost of construction.

The ASTM standards replicate the current requirements of SBCCI SSTD-99, and therefore will not increase the cost of construction.

**Staff Analysis:** A review of the standard proposed for inclusion in the code, ASTM C1569-03(2016) and C1570-03(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.
2018 International Building Code

Revise as follows:

1504.4 Ballasted low-slope single-ply roof systems. Ballasted low-slope (roof slope < 2:12) single-ply roof system coverings installed in accordance with Sections 1507.12 and 1507.13 shall be designed in accordance with Section 1504.8 and ANSI/SPRI RP-4.

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.

Exception: Ballasted single-ply roof systems complying with Section 1504.4

Reason: This proposal makes a much-needed correction to section 1504.4 for ballasted roof systems for low-slope single-ply roofs. This proposal revises Section 1504.4 so that ballasted roofs comply with ANSI/SPRI RP-4 and not 1504.8. The requirements in RP-4 were developed for the appropriate application, installation and to prevent ballast scour for this specific type of single-ply ballasted system. The scour wind speed is below that at which blowoff would occur. It also provides design options for various conditions.

Section 1504.8 is based on the wind speeds for blow-off and only deals with smaller aggregate used for surfacing of built up roofs (BUR) and sprayed polyurethane foam (SPUF) roofs, which are completely different systems than ballasted roofs. For this reason an exception has been added in Section 1504.8 for ballasted single-ply roof systems complying with Section 1504.4.

The requirements in ANSI/SPRI RP-4 are based on a complete set of wind tunnel tests conducted in the largest commercially available wind tunnel in North America located at the National Research Council Canada. In this test series all variables that would impact the wind performance of ballasted single ply roof assemblies were evaluated, including stone size and size distribution as specified in ASTM D7655 Standard Classification for Size of stone used as ballast for membrane roof systems.

In this series of tests three critical windspeeds were identified for each condition of parapet height and stone size, windspeed 1 is the speed at which the stone distribution first begins to move, windspeed 2 is the speed is that which if maintained would result in stone scouring, and windspeed three is the speed at which stone blow-off occurs. The requirements in the Design Table of ANSI/SPRI RP-4 are based on windspeed 2, or the windspeed at which stone scour would occur.

The requirements of this standard have been updated based on field performance and in the most recent edition the design tables have been revised to reflect current methodology for interpreting wind tunnel data. Section 1504.8 does not consider the critical variables of parapet height and stone size and should not be applicable to ballasted single ply roof systems.

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

This proposal only clarifies what design requirements are to be used for ballasted single-ply roof systems.
2018 International Building Code

Revise as follows:

1504.5 Edge securement systems for low-slope roofs. Low-slope metal edge systems, except gutters, installed on built-up, modified bitumen and single-ply roof system metal edge securement, except gutters systems having a slope less than 2:12, shall be designed and installed for wind loads in accordance with Chapter 16 and tested for resistance in accordance with Test Methods RE-1, RE-2 and RE-3 of ANSI/SPRI ES-1, except basic design wind speed, V, shall be determined from Figures 1609.3(1) through 1609.3(8) as applicable.

Reason: KULIK: This proposal is intended to clarify that regardless if the roof membrane is either independently or dependently terminated, the edge metal system needs to be properly tested to the appropriate standard. Metal edge systems prevent water infiltration, and in many cases to also secure the roof membrane. Loss of the edge system or components of the edge system during a high wind event could allow for water infiltration even if the roof membrane remains secure. Furthermore, any component of the edge system that becomes disengaged during a high wind event will become a projectile that can damage the roof membrane and other building components (windows, doors, walls, etc.), and possibly injure people. Therefore, metal edge systems should be tested per ES-1 whether they secure the membrane or not.

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 the BCAC website at: https://www.iccsafe.org/codes-tech-support/codes/code-development-process/building-code-action-committee-bcac/.

HICKMAN: This proposal clarifies that the edge metal systems need to be properly tested to the appropriate standard regardless if the roof membrane is either independently or dependently terminated. Metal edge systems prevent water infiltration, and in many cases to also secure the roof membrane. Loss of the edge system or components of the edge system during a high wind event could allow for water infiltration even if the roof membrane remains secure.

Furthermore, any component of the edge system that becomes disengaged during a high wind event will become a projectile that can damage the roof membrane and other building components (windows, doors, walls, etc.), and possibly injure people. Therefore, metal edge systems should be tested per ES-1 whether they secure the membrane or not.

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

KULIK: This proposal just clarifies that this test applies to edge metal regardless of installation method.

HICKMAN: The code change proposal will not increase or decrease the cost of construction. This proposal only clarifies that this test applies to edge metal regardless of installation method.
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.

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 the BCAC 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.
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.
S18-19

IBC®: 1504.7

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

2018 International Building Code

Revise as follows:

1504.7 Impact resistance. Roof coverings installed on low-slope roofs (roof slope < 2:12) in accordance with Section 1507 shall resist impact damage based on the results of tests conducted in accordance with ASTM D3746, ASTM D4272 or the “Resistance to Foot Traffic Test” in Section 5.5 of FM 4470.

Reason: The proposal removes the section reference to avoid correlation issues should the referenced standard section numbering be revised in the future. The correct reference is section 4.6 of FM 4470 which has been corrected from section 5.5 per the errata for IBC 2018.

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

The proposal is editorial.
S19-19
IBC: 1504.8, 1607.13.6 (New)

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

2018 International Building Code
Revise 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.

Exception: A roof that complies with all of the following:

1. A parapet is placed on all exterior sides of the roof.
2. The parapet is tall enough to retain the volume of roofing material, regardless of wind direction.
3. The roof and parapet are designed for the additional live load of the retained aggregate at the edge of the roof.

Add new text as follows:

1607.13.6 Surfacing and ballast materials. For a building located in a hurricane-prone region, 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, where aggregate is used as surfacing for roof coverings or aggregate, gravel or stone is used as ballast and a parapet is placed on all exterior sides of the roof to retain the volume of roofing material, the roof and parapet shall be designed for the additional live load of the retained aggregate, regardless of wind direction.

Reason: In the 2018 code change cycle, S20-16 proposed the replacement of Table 1504.8 with a table that would allow aggregate roofing systems to be used on roofs in various wind speed and wind exposure conditions if the building being designed had a parapet whose minimum height equaled or exceeded the parapet height noted in the revised table. The reason statement for the 2018 code change S20-16 implies that this proposal was based on “the K-W design method (Kind Wardlaw 1976), the wind tunnel studies underlying the KW design method (Kind 1977), or a quantitative analysis of observed good and bad roofing system performances in real wind events”.

NCSEA opposed S20-16. The proposal was revised by a public comment from the proponents, which was unsuccessful. However, members of the Structural Committee appeared to be in favor of using parapets to retain roofing aggregate.

Aggregate blow-off from roofs was reported in Houston, TX during Hurricane Alicia in 1982, in Miami-Dade County, FL during Hurricane Andrew, in New Orleans, LA during Hurricane Katrina, and in other cities during these and other events. After Hurricane Katrina, the NCSEA Code Advisory Committee witnessed the damage to the glazing systems of The New Orleans Shopping Center Office Building and The Amoco Building both of which were on Poydrus Street in New Orleans, LA. The glazing systems of these buildings were damaged by aggregate blown off buildings on the north side of Poydrus Street. We also witnessed the damage to the glazing system of the Hyatt Regency Hotel from the vantage point of the roof of the Amoco Building. The Amoco Building previously had an aggregate ballasted roof. Most of the aggregate had been blown off of the roof. Much of the aggregate that remained on the roof was ramped up against the parapet on the south side of the building. Once the aggregate ramp height equaled the parapet height, the remaining aggregate was swept up the ramp and off the roof. Directly south of the Amoco Building, windows of the Hyatt Regency Hotel had been broken (see Figure 1), and aggregate was retrieved from the bedrooms of the hotel.

Figure 1 - Glazing failures in Hyatt Regency Hotel, New Orleans, LA following Hurricane Katrina.
Wind speeds in New Orleans, LA during Hurricane Katrina were reported as being less than the design wind speeds from ASCE7.

In the 2006 Public Comment Hearing John Loscheider testified that the national roofing Contractors Association’s magazine reported aggregate roofing blow-off damage to other buildings in New Orleans after Hurricane Katrina.

The presence of aggregate ramps and aggregate blow-off has been reported previously. For example, aggregate ramps were observed against the six-foot tall parapets of the National Hurricane Center in Miami after Hurricane Andrew. We understand that aggregate blow-off from this roof was also reported.

This code change proposal would allow buildings, whose height exceeds the limitations of Table 1504.8, to be constructed using an aggregate surfaced or aggregate ballasted roof, if the building had a parapet that was of sufficient height that it could retain the volume of aggregate.

We note that there are other alternates to aggregate used as surfacing for roof coverings or for aggregate, gravel or stone used as ballast. They are probably more expensive, but we believe that they are almost certainly less expensive than the window replacement costs due to aggregate blow-off.

If the aggregate is transported to the edge of the roof, there may be the need for additional gravity load capacity. This requirement is dealt with by adding section 1607.13.6.

**Bibliography:**

**Cost Impact:** The code change proposal will increase the cost of construction
Increasing parapet height may increase the cost of construction if the parapet retention system is used, but it is not mandated, it is listed as an alternate. Another roofing alternative may be less expensive.
S20-19

IBC®: 1504.8

Proponent: Jay Crandell, P.E., ARES Consulting, representing self (jcrandell@aresconsulting.biz); Ellen Thorp, EPDM Roofing Association; Mike Fischer (mfischer@kellencompany.com)

2018 International Building Code

Revise 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.

**Exception:** Where the aggregate surfaced roof system and parapets shall be designed by a registered design professional to control aggregate blow-off.

**Reason:** There are proven and accepted design methods to control aggregate blow-off from roofs which are superior to those in Table 1504.8. These include the prescribed provisions in the code-referenced ANSI/SPRI RP-4 standard and also the design methodology used to develop those provisions (Kind and Wardlaw, 1976). Newer methodologies based on Kind and Wardlaw (1976) are explained and verified as being effective based on comparison to numerous sources of field data (Crandell and Smith, 2009; Crandell and Fischer, 2010; Morrison, 2011). Why is this important? The provisions of existing Table 1504.8 lack any requirement for use of parapets for building heights of up to 170-feet in height because the science and design approach behind the table is seriously flawed. Consequently, the requirements in Table 1504.8 are incomplete and potentially unsafe. For these reasons, alternative solutions by registered design professionals should be explicitly permitted. This proposal is also compatible with a separate proposal (by the same proponents) to fix the many problems with existing Table 1504.8 and Section 1504.8 as explained in the reason statement to that proposal.

**Bibliography:**

**Cost Impact:** The code change proposal will not increase or decrease the cost of construction. The proposed exception provides an alternative to Table 1504.8 and does not replace or change it.
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**

<table>
<thead>
<tr>
<th>NOMINAL DESIGN WIND SPEED, $V_{sec}$ (mph)</th>
<th>MAXIMUM MEAN ROOF HEIGHT (ft)_a, b, c, d</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Exposure category</strong></td>
<td><strong>B</strong></td>
</tr>
<tr>
<td>85</td>
<td>470</td>
</tr>
<tr>
<td>90</td>
<td>440</td>
</tr>
<tr>
<td>95</td>
<td>75</td>
</tr>
<tr>
<td>100</td>
<td>55</td>
</tr>
<tr>
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<td>48</td>
</tr>
<tr>
<td>110</td>
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</tr>
<tr>
<td>115</td>
<td>30</td>
</tr>
<tr>
<td>120</td>
<td>25</td>
</tr>
<tr>
<td>Greater than 120</td>
<td>15</td>
</tr>
</tbody>
</table>

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 $V_{sec}$ the height associated with the next higher value of $V_{sec}$ shall be used, or direct interpolation is permitted.
- c. NP = gravel and stone not permitted for any roof height.
- d. $V_{sec}$ shall be determined in accordance with Section 1609.3.1.

**TABLE 1504.8**

**MINIMUM REQUIRED PARAPET HEIGHT (INCHES) FOR AGGREGATE SURFACED ROOFS**

<table>
<thead>
<tr>
<th>AGGREGATE SIZE</th>
<th>MEAN ROOF HEIGHT (ft)</th>
<th>WIND EXPOSURE AND BASIC DESIGN WIND SPEED (MPH)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td><strong>Exposure B</strong></td>
</tr>
<tr>
<td></td>
<td></td>
<td>&lt;=95 100 105 110 115 120 130 140 150</td>
</tr>
<tr>
<td>ASTM D1863 (No.7 or No.67) or ASTM D7655 (No.4)</td>
<td>15</td>
<td>2 2 2 2 12 12 16 20 24</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>2 2 2 2 12 14 18 22 26</td>
</tr>
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<td></td>
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<td>2 2 2 13 15 17 21 25 30</td>
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<td></td>
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<td>12 12 14 16 18 21 25 30 35</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>14 16 19 21 24 27 32 37 42</td>
</tr>
</tbody>
</table>
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.


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.

Proposal # 5005
S22-19

IBC®: SECTION 1506 (New), 1506.1

Proponent: Mark Graham, National Roofing Contractors Association (NRCA), representing National Roofing Contractors Association (NRCA)
(mgraham@nrca.net)

2018 International Building Code

SECTION 1506
MATERIALS

Revise as follows:

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

Reason: This code change proposal is intended to clarify the intent of the code. The requirement for roof coverings “...be applied in accordance with... the manufacturer’s installation instructions.” is unnecessary and redundant in this section because this is already required in Section 1507-Requirements for Roof Coverings.

A requirement for the roofing covering to be applied according to the listing is added here for clarity. Section 1505-Fire Classification already requires roof assemblies and roof coverings to be listed and Section 1506.3 requires materials and product packaging to bear testing agency labels.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. The strigency of the code is not increased or decreased by this code change proposal.

Proposal # 4961
2018 International Building Code

Revise as follows:

1507.1.1 Underlayment. Underlayment for asphalt shingles, clay and concrete tile, metal roof panels, 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, D6757 and ASTM WK51913 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 ASTM D4869 Type IV, ASTM WK51913 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 using corrosion-resistant fasteners in a grid pattern of 12 inches (305 mm) between side laps and 6 inches (152 mm) at end laps. End laps shall be 4 inches (102 mm) and shall be offset by 6 feet (1829 mm). Metal caps shall have a thickness of not less than 0.010 inch (mm). Power-driven metal caps shall have a thickness of 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 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 1/2 inch (19.1 mm) into the roof sheathing.

4. Structural metal panels that do not require a substrate or underlayment.

<table>
<thead>
<tr>
<th>TABLE 1507.1.1(1) UNDERLAYMENT TYPES</th>
</tr>
</thead>
<tbody>
<tr>
<td>ROOF COVERING</td>
</tr>
<tr>
<td>Asphalt shingles</td>
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<tr>
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<tr>
<td></td>
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<tr>
<td>Clay and concrete tiles</td>
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<tr>
<td></td>
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<tr>
<td></td>
</tr>
<tr>
<td>Metal panels</td>
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<tr>
<td></td>
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<tr>
<td></td>
</tr>
<tr>
<td>Material Type</td>
</tr>
<tr>
<td>-------------------------------</td>
</tr>
<tr>
<td>Metal roof shingles</td>
</tr>
<tr>
<td>Mineral-surfaced roll</td>
</tr>
<tr>
<td>Slate shingles</td>
</tr>
<tr>
<td>Wood shingles</td>
</tr>
<tr>
<td>Wood shakes</td>
</tr>
<tr>
<td>Photovoltaic shingles</td>
</tr>
</tbody>
</table>

**Add new text as follows:**

**ASTM**

**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.

Proposal # 5322

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S23-19
S24-19

IBC®: 1507.1.1

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

2-1. 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.

2-2. As an alternative, two layers of underlayment complying with ASTM D226 Type II or ASTM D4869 Type IV 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). 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.

4-3. Structural metal panels that do not require a substrate or underlayment.

Reason: The requirements for ASTM D1970 underlayment are redundant as the standard is listed in Section 1507.1.1.

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

The proposal is editorial.

Proposal # 5678
S25-19

IBC®: 1507.3.1

Proponent: Shahen Akelyan (shahen.akelyan@lacity.org)

2018 International Building Code

Revise as follows:

1507.3.1 Deck requirements. Concrete and clay tile shall be installed only over solid sheathing or spaced structural sheathing boards.

Reason: Section 1507.3.1 is amended to require concrete and clay tiles to be installed only over solid structural sheathing boards. The change is necessary because there were numerous observations of tile roofs pulling away from wood framed buildings following the 1994 Northridge Earthquake. The SEAOSC/LA City Post Northridge Earthquake committee findings indicated significant problems with tile roofs was due to inadequate design and/or construction. Therefore, the amendment is needed to minimize such occurrences in the event of future significant earthquakes. This amendment will reduce the failure of concrete and clay tile roofs during a significant earthquake and is in accordance with the scope and objectives of the California Building Code.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
The revision limits use of spaced sheathing which does no increase any cost.
S26-19

IBC®: 1507.8.1

Proponent: David Roodvoets (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. The spaced sheathing shall be open to the building interior and shall not be backed with spray foam or other material.

Reason: Shingles installed over spaced sheathing 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.

Proposal # 4715
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 \( \frac{3}{4} \) inch (19.1 mm) into the sheathing. For sheathing less than \( \frac{1}{2} \) 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 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 shakes or pressure impregnated preservative treated shakes shall be stainless steel Type 316. Fasteners shall have a minimum penetration of \( \frac{3}{4} \) inch (19.1 mm) into the sheathing. For sheathing less than \( \frac{3}{8} \) 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 requirements in 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.

Proposal # 4708

S27-19
2018 International Building Code

Revised as follows:

1507.3.6 Fasteners. Tile fasteners shall be corrosion resistant and not less than 11-gage, 0.120 inch (3 mm), \(\frac{5}{16}\) inch (8.0 mm) head, and of sufficient length to penetrate the deck not less than \(\frac{3}{8}\) inch (19.1 mm) or through the thickness of the deck, whichever is less. Attaching wire for clay or concrete tile shall not be smaller than 0.083 inch (2.1 mm). Perimeter fastening areas include three tile courses but not less than 36 inches (914 mm) from either side of hips or ridges and edges of eaves and gable rakes.

Reason: ASTM F1667-18 requires that when gage is used as a diameter for nails, a decimal equivalent must also be shown. This requirement was put in place because of the multiple and conflicting wire gage tables that are used in the manufacturing of nails.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. This proposal will not change the cost of production. It only provides clarification required by ASTM F1667-18
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. Although most drying of the underlayment is to the outside; there is some drying that must occur into the building. Spray foam prevents the drying, allowing moisture to accumulate below the shake. Direct backing of the shake with insulating foam also raises the temperature 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 manufacturer'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.
**2018 International Building Code**

Revise as follows:

<table>
<thead>
<tr>
<th>MATERIAL STANDARD</th>
<th>STANDARD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acrylic coatings used in roofing</td>
<td>ASTM D6083</td>
</tr>
<tr>
<td>Aggregate surfacing</td>
<td>ASTM D1863; D7655</td>
</tr>
<tr>
<td>Asphalt adhesive used in roofing</td>
<td>ASTM D3747</td>
</tr>
<tr>
<td>Asphalt cements used in roofing</td>
<td>ASTM D2822; D3019; D4586</td>
</tr>
<tr>
<td>Asphalt-coated glass fiber base sheet</td>
<td>ASTM D4601</td>
</tr>
<tr>
<td>Asphalt coatings used in roofing</td>
<td>ASTM D1227; D2823; D2824; D4479</td>
</tr>
<tr>
<td>Asphalt glass felt</td>
<td>ASTM D2178</td>
</tr>
<tr>
<td>Asphalt primer used in roofing</td>
<td>ASTM D41</td>
</tr>
<tr>
<td>Asphalt-saturated and asphalt-coated organic felt base sheet</td>
<td>ASTM D2626</td>
</tr>
<tr>
<td>Asphalt-saturated organic felt (perforated)</td>
<td>ASTM D226</td>
</tr>
<tr>
<td>Asphalt used in roofing</td>
<td>ASTM D312</td>
</tr>
<tr>
<td>Coal-tar cements used in roofing</td>
<td>ASTM D4022; D5643</td>
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<tr>
<td>Coal-tar saturated organic felt</td>
<td>ASTM D227</td>
</tr>
<tr>
<td>Coal-tar pitch used in roofing</td>
<td>ASTM D450; Type I or II</td>
</tr>
<tr>
<td>Coal-tar primer used in roofing, dampproofing and waterproofing</td>
<td>ASTM D43</td>
</tr>
<tr>
<td>Glass mat, coal tar</td>
<td>ASTM D4990</td>
</tr>
<tr>
<td>Glass mat, venting type</td>
<td>ASTM D4897</td>
</tr>
<tr>
<td>Mineral-surfaced inorganic cap sheet</td>
<td>ASTM D3909</td>
</tr>
<tr>
<td>Thermoplastic fabrics used in roofing</td>
<td>ASTM D5665, D5726</td>
</tr>
</tbody>
</table>

**Reason:** This proposal adds an accepted ASTM standard for specification of aggregate for built-up roofs. It also coordinates with a separate proposal providing improved provisions for parapet height and aggregate size to control aggregate blow-off in extreme wind events.

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

The proposal lists an additional aggregate ASTM standard, which is already listed in the referenced standards, and therefore would not impact current construction costs.
2018 International Building Code

Revise as follows:

1507.12 Thermoset single-ply roofing. The installation of thermoset single-ply roofing shall comply with the provisions of this section.

1507.12.1 Slope. Thermoset single-ply membrane roofs shall have a design slope of not less than one-fourth unit vertical in 12 units horizontal (2-percent slope) for drainage.

1507.12.2 Material standards. Thermoset single-ply roof coverings shall comply with ASTM D4637 or ASTM D5019, the material standards in Table 1507.12.2.

Add new text as follows:

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>MATERIAL STANDARD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chlorosulfonated polyethylene (CSPE) or polyisobutylene (PIB)</td>
<td>ASTM D5019</td>
</tr>
<tr>
<td>Ethylene propylene diene monomer (EPDM)</td>
<td>ASTM D4637</td>
</tr>
<tr>
<td>Ketone Ethylene Ester (KEE)</td>
<td>ASTM D6754</td>
</tr>
<tr>
<td>Polyvinyl Chloride (PVC)</td>
<td>ASTM D4434</td>
</tr>
<tr>
<td>Thermoplastic polyolfin (TPO)</td>
<td>ASTM D6878</td>
</tr>
</tbody>
</table>

Revise as follows:

1507.12.3 Ballasted thermoset low-slope roofs. Ballasted thermoset low-slope roofs (roof slope < 2:12) shall be installed in accordance with this section and Section 1504.4. Stone used as ballast shall comply with ASTM D448 or ASTM D7655.

Delete without substitution:

1507.13 Thermoplastic single-ply roofing. The installation of thermoplastic single-ply roofing shall comply with the provisions of this section.

1507.13.1 Slope. Thermoplastic single-ply membrane roofs shall have a design slope of not less than one-fourth unit vertical in 12 units horizontal (2-percent slope).

1507.13.2 Material standards. Thermoplastic single-ply roof coverings shall comply with ASTM D4434, ASTM D6754 or ASTM D6878.

1507.13.3 Ballasted thermoplastic low-slope roofs. Ballasted thermoplastic low-slope roofs (roof slope < 2:12) shall be installed in accordance with this section and Section 1504.4. Stone used as ballast shall comply with ASTM D448 or ASTM D7655.

Reason: This code change proposal is intended to clarify and streamline the code's requirements applicable to single-ply membrane roof systems. The code currently addresses thermoset (i.e., EPDM, CSPE) single-ply membrane roofs in Section 1507.12 and thermoplastic (i.e., PVC, KEE, TPO) single-ply membrane roofs in Section 1507.13. Other than the references to specific ASTM material standards, the other requirements in Section 1507.12 and Section 1507.13 are identical.

This code change proposal combines the requirements for single-ply membrane roof systems into one subsection, Section 1507.12-Single-ply Roofs. Also, the ASTM material standards references are provided in a new table, Table 1507.12.2-Single-ply Roofing Material Standards; this type of material standards table is similar in format to Table 1507.10.2-Built-up Roofing Material Standards, et. al.

No changes to the technical requirements for single-ply membrane roof systems are included in this code change proposal.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. This code change proposal only reformats and rearranges the code's current requirements.
2018 International Building Code

1507.15 Liquid-applied roofing. The installation of liquid-applied roofing shall comply with the provisions of this section.

1507.15.1 Slope. Liquid-applied roofing shall have a design slope of not less than one-fourth unit vertical in 12 units horizontal (2-percent slope).

Revise as follows:

1507.15.2 Material standards. Liquid-applied roofing shall comply with ASTM C836, ASTM C957, ASTM D1227 or ASTM D3468, ASTM D6083, ASTM D6694 or ASTM D6947. D3468.

Reason: This code change proposal is intended to clarify the code's intent regarding the use of liquid-applied roof coverings. Currently, the material standards included in Section 1507.15.2 incorrectly include a combination of liquid-applied roof coverings and roof coating products. This proposal intends to remove the material standards for roof coating products from Section 1507.15-Liquid-applied Roofing to facilitate adding a new dedicated roof coating section in a separate code change proposal.

ASTM C836 (liquid-applied waterproofing membrane), ASTM C957 (liquid-applied waterproofing membrane with wearing surface) and ASTM D3468 (neoprene and CSPE used in roofing and waterproofing) are specific liquid-applied roof coverings. These three material standards are intended to remain in this section.

ASTM D1227 (asphaltic emulsion coating) and ASTM D6083 (acrylic roof coating) are specific roof coatings products, not liquid-applied roof coverings. These two standards are proposed to be removed from this section and be added to a new dedicated roofing coating section in a separate code change proposal.

Also, ASTM D6694 and ASTM D6947 are proposed to be removed from this section. ASTM D6694 (silicone for use in SPF roof systems) and ASTM D6947 (polyurethane for use in SPF roof systems) are specific roof coating products intended for use in SPF roof systems and are already included in Section 1507.14-Spray Polyurethane Foam Roofing.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. This code change proposal is a rearrangement of the code's current requirements regarding regarding liquid-applied roof covering and roof coating products.
2018 International Building Code

Revise as follows:

1507.17.6 Material standards. Photovoltaic shingles shall be listed and labeled in accordance with UL 1703.

1507.17.8 Wind resistance. Photovoltaic shingles shall be tested in accordance with procedures and acceptance criteria in ASTM D3161. Photovoltaic shingles shall comply with the classification requirements of Table 1504.1.1 for the appropriate maximum nominal design wind speed. Photovoltaic shingle packaging shall bear a label to indicate compliance with the procedures in ASTM D3161 and the required classification from Table 1504.1.1.

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

Delete without substitution:

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

Add new standard(s) as follows:

UL

7103-19: Outline of Investigation for Building-Integrated Photovoltaic Roof Coverings
2018 International Residential Code

Revise as follows:

R902.3 Building-integrated photovoltaic product. Building-integrated photovoltaic products installed as the roof covering shall be tested, listed and labeled for fire classification in accordance with Section R902.1. All BIPV products shall be installed where the edge of the roof is less than 3 feet (914 mm) from a lot line.

R905.16.6 Wind resistance. Photovoltaic shingles shall be tested in accordance with procedures and acceptance criteria in ASTM D3161. Photovoltaic shingles shall comply with the classification requirements of Table R905.2.4.1-R905.16.6 for the appropriate maximum basic wind speed. Photovoltaic shingle packaging shall bear a label to indicate compliance with the procedures in ASTM D3161 and the required classification from Table R905.2.4.1.

R905.16.4 Material standards. Photovoltaic shingles shall be listed and labeled in accordance with UL 7103.

Add new text as follows:

<table>
<thead>
<tr>
<th>MAXIMUM ULTIMATE DESIGN WIND SPEED, $V_{UL}$ FROM FIGURE R301.2(5)A (mph)</th>
<th>MAXIMUM BASIC WIND SPEED $V_{ASD}$ FROM TABLE R301.2.1.3 (mph)</th>
<th>UL 7103 SHINGLE CLASSIFICATION</th>
<th>UL 7103 SHINGLE CLASSIFICATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>110</td>
<td>85</td>
<td>D, G or H</td>
<td>A, D or F</td>
</tr>
<tr>
<td>116</td>
<td>90</td>
<td>D, G or H</td>
<td>A, D or F</td>
</tr>
<tr>
<td>129</td>
<td>100</td>
<td>G or H</td>
<td>A, D or F</td>
</tr>
<tr>
<td>142</td>
<td>110</td>
<td>G or H</td>
<td>F</td>
</tr>
<tr>
<td>155</td>
<td>120</td>
<td>G or H</td>
<td>F</td>
</tr>
<tr>
<td>168</td>
<td>130</td>
<td>H</td>
<td>F</td>
</tr>
<tr>
<td>181</td>
<td>140</td>
<td>H</td>
<td>F</td>
</tr>
<tr>
<td>194</td>
<td>150</td>
<td>H</td>
<td>F</td>
</tr>
</tbody>
</table>

Note: The standard calculations contained in UL 7103 assume Exposure Category B or C and a building height of 60 feet or less. Additional calculations are required for conditions outside of these assumptions.

Revise as follows:

R905.17.5 Material standards. BIPV roof panels shall be listed and labeled in accordance with UL 7103.

Delete without substitution:

R905.17.7 Wind resistance. BIPV roof panels shall be tested in accordance with UL 1897. BIPV roof panel packaging shall bear a label to indicate compliance with UL 1897.

Add new standard(s) as follows:

UL 7103-19: Outline of Investigation for Building-Integrated Photovoltaic Roof Coverings

Reason: BIPV products are designed to directly replace roof covering, therefore a BIPV system must be evaluated not only as a PV module but also as a roof covering with additional Code required to verify performance in the following areas: testing such as:

- Fire testing (UL 790 or ASTM E108)
- Impact testing
- Wind resistance (ASTM D3161 or UL 1897)
Having one standard, UL 7103, to address electrical, fire, wind resistance, impact resistance and durability of this new type of building material makes it far easier to determine compliance with all the minimum code requirements. The standard includes all the marking requirements for the ratings (fire classification, wind resistance, and electrical) and the minimum content for the installation instructions.

**Cost Impact:** The code change proposal will not increase or decrease the cost of construction. The requirements remain the same. This proposal is simply editorial by providing a different format in order to assist in determining code compliance.

**Staff Analysis:** A review of the standard proposed for inclusion in the code, UL7103-19, 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.
S34-19 Part I
PART I — IBC®: [BG] 1510.7.2, 1507.17.6, 1507.18.5, 3111.3.1, UL Chapter 35 (New)
PART II — IRC®: R324.3.1, R905.16.4, R905.17.5, UL Chapter 44 (New)
Proponent: Jonathan Roberts, UL LLC, representing UL LLC (jonathan.roberts@ul.com)

2018 International Building Code
Revise as follows:

[BG] 1510.7.2 Photovoltaic panels and modules. Rooftop-mounted photovoltaic panels and modules shall be listed and labeled in accordance with UL 1703 or with both UL 61730-1 and UL 61730-2, and shall be installed in accordance with the manufacturer’s instructions.

1507.17.6 Material standards. Photovoltaic shingles shall be listed and labeled in accordance with UL 1703 or with both UL 61730-1 and UL 61730-2.

1507.18.5 Material standards. BIPV roof panels shall be listed and labeled in accordance with UL 1703 or with both UL 61730-1 and UL 61730-2.

3111.3.1 Equipment. Photovoltaic panels and modules shall be listed and labeled in accordance with UL 1703 or with both UL 61730-1 and UL 61730-2. Inverters shall be listed and labeled in accordance with UL 1741. Systems connected to the utility grid shall use inverters listed for utility interaction.

Add new standard(s) as follows:

**UL**

**UL 61730-1-2017**: Photovoltaic (PV) Module Safety Qualification - Part 1: Requirements for Construction

**UL 61730-2-2017**: Photovoltaic (PV) Module Safety Qualification - Part 2: Requirements for Testing
S34-19 Part II
IRC®: R324.3.1, R905.16.4, R905.17.5, UL Chapter 44 (New)

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

2018 International Residential Code
Revise as follows:

R324.3.1 Equipment listings. Photovoltaic panels and modules shall be listed and labeled in accordance with UL 1703 or with both UL 61730-1 and UL 61730-2. Inverters shall be listed and labeled in accordance with UL 1741. Systems connected to the utility grid shall use inverters listed for utility interaction.

R905.16.4 Material standards. Photovoltaic shingles shall be listed and labeled in accordance with UL 1703 or with both UL 61730-1 and UL 61730-2.

R905.17.5 Material standards. BIPV roof panels shall be listed and labeled in accordance with UL 1703 or with both UL 61730-1 and UL 61730-2.

Add new standard(s) as follows:

UL LLC
333 Pfingsten Road
Northbrook IL 60062


Reason: UL 61730-1 and UL 61730-2 are new standards that will eventually replace UL 1703.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. There is no cost impact because this simply provides alternative standards.

Staff Analysis: A review of the standard proposed for inclusion in the code, UL 61730-1-2017 and 61730-2-2017, 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.
2018 International Building Code

Add new text as follows:

SECTION 1509
ROOF COATINGS

1509.1 General. The installation of a roof coating on a roof covering shall comply with the requirements of Section 1505 and this section.

1509.2 Material standards. Roof coating materials shall comply with the standards in Table 1509.2.

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>STANDARD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Acrylic coating</td>
<td>ASTM D6083</td>
</tr>
<tr>
<td>Asphaltic emulsion coating</td>
<td>ASTM D1227</td>
</tr>
</tbody>
</table>

Reason: This proposed code change is intended to provide specific requirements regarding the use of roof coating materials. The term "roof coating" is already defined in Chapter 2-Definitions and is used in Section 1511.3.1.4; however, the code currently provides little guidance or requirements relating to the use of roof coatings.

The new section proposed here provides a requirement that roof coatings be tested as a part of a fire-classified roof assembly/covering in accordance with Section 1505-Fire Classification and comply with applicable material standards.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. This code change proposal does not increase or decrease the stringency of the code; it reformats the code's existing requirements for roof coatings.

Proposal # 4966

S35-19
2018 International Building Code

Revise as follows:

[B] LIVE LOAD, ROOF. A load on a roof produced:

1. During maintenance by workers, equipment and materials; or
2. During the life of the structure by movable objects such as planters or other similar small decorative appurtenances that are not occupancy related; or
3. By the use and occupancy of the roof such as for roof gardens or assembly areas.

SECTION 1602 NOTATIONS

1602.1 Notations. The following notations are used in this chapter:

\[ D \] = Dead load.

\[ D_i \] = Weight of ice in accordance with Chapter 10 of ASCE 7.

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

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

\[ F_w \] = Flood load in accordance with Chapter 5 of ASCE 7.

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

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

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

\[ R \] = Rain load.

\[ S \] = Snow load.

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

\[ V_{pod} \] = 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.

Reason: This proposal both corrects an inconsistency within the IBC and coordinates the IBC with the referenced design load standard, Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7-16).

ASCE 7-16 considers occupancy related live loads to be Live Loads, symbol \( L \), regardless of whether they are applied to interior floors or on the roof. This distinction is important for two reasons. First, the load combinations used in structural design treat Live Loads, \( L \), differently from Roof Live Loads, \( L_r \). Second, the allowable reductions for Roof Live Loads, \( L_r \), are different than the allowable reductions for Live Loads, \( L \). In both instances, occupancy related loads are treated in the same manner, therefore it makes sense to place them under the same definition.

The IBC considers occupancy related live loads on a roof to be Roof Live Loads, not Live Loads, as indicated in the Roof Live Load definition in Chapter 2. In order to treat occupancy related roof live loads appropriately in load combinations, the IBC amends the definition of the symbols \( L \) and \( L_r \) in Section 1602 such that \( L_r \) is limited to loads of 20 psf or less. The changes to the definition of the symbols effectively modifies the definitions in Section 202. This practice is inconsistent and potentially confusing.
This proposal aligns the IBC provisions for Live Load and Roof Live Load with the ASCE 7-16 provisions.

**Cost Impact:** The code change proposal will not increase or decrease the cost of construction
This proposal does not create additional design requirements.
2018 International Building Code

Revise as follows:

1602.1 Notations. D = Dead load.

\[ D_i = \text{Weight of ice in accordance with Chapter 10 of ASCE 7.} \]

E = Combined effect of horizontal and vertical earthquake induced forces as defined in Chapter 12 Section 2.3.6 of ASCE 7.

\[ E_v = \text{Effect of horizontal seismic forces as determined in Chapter 12 of ASCE 7.} \]

\[ E_{sh} = \text{Effect of horizontal seismic forces including overstrength as determined in Chapter 12 of ASCE 7.} \]

\[ E_v = \text{Vertical seismic effect applied in the vertical downward direction as in determined in Chapter 12 of ASCE 7.} \]

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

\[ F_s = \text{Flood load in accordance with Chapter 5 of ASCE 7.} \]

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

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

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

R = Rain load.

S = Snow load.

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

\[ V_{asp}= \text{Allowable stress design wind speed, miles per hour (mph) (km/hr) where applicable.} \]

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.

\[ W_i = \text{Wind-on-ice in accordance with Chapter 10 of ASCE 7.} \]

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:

\[ 1.4(D + F) \text{ (Equation 16-1) } \]

\[ 1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) \text{ (Equation 16-2) } \]

\[ 1.2(D + F) + 1.6(L_r \text{ or } S \text{ or } R) + 1.6H + (f_1L \text{ or } 0.5W) \text{ (Equation 16-3) } \]

\[ 1.2(D + F) + 1.0W + f_1L + 1.6H + 0.5(L_r \text{ or } S \text{ or } R) \text{ (Equation 16-4) } \]

\[ 1.2(D + F) + 1.0E + f_1L + 1.0E + 1.6H + f_2S \text{ (Equation 16-5) } \]

\[ 0.9D + 1.0W + 1.6H \text{ (Equation 16-5.6) } \]

\[ 0.9(D + F) + 1.0E + 1.6H \text{ (Equation 16-7) } \]
where:

\[ f_1 = 1 \] for places of public assembly live loads in excess of 100 pounds per square foot (4.79 kN/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.

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:

\[
1.2(D + F) + 1.0E_2 + 1.0 E_{2m} + f_1 L + 1.6H + f_2 S \quad \text{(Equation 16-6)}
\]

\[
0.9(D + F) - 1.0E_2 + 1.0E_{2m} + 1.6H \quad \text{(Equation 16-7)}
\]

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

\[
1.2(D + F) + 1.0E_2 + 1.0 E_{2m} + f_1 L + 1.6H + f_2 S \quad \text{(Equation 16-8)}
\]

\[
0.9(D + F) - 1.0E_2 + 1.0E_{2m} + 1.6H \quad \text{(Equation 16-9)}
\]

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:

1605.2.2 Other loads. Where flood loads, \( F_n \), 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.

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-\( \theta \) 10)

\[ D + H + F + I \]

(Equation 16-\( \theta \) 11)
\[ D + H + F + (I_v \text{ or } S \text{ or } R) \]
(Equation 16-10 12)

\[ D + H + F + 0.75(I_v) + 0.75(I_v \text{ or } S \text{ or } R) \]
(Equation 16-13 13)

\[ D + H + F + (0.6W + 0.7E) \text{ (Equation 16-14 14)} \]

\[ D + H + F + 0.75(0.6W) + 0.75L + 0.75(I_v \text{ or } S \text{ or } R) \]
(Equation 16-15 15)

\[ D + H + F + 0.75(0.7E) + 0.75L + 0.75L + 0.75S \text{ (Equation 16-16 16)} \]

\[ 0.6D + 0.6W + H \]
(Equation 16-15 16)

\[ 0.6(D + F) + 0.7E + H \text{ (Equation 16-16 16)} \]

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

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:

\[ D + H + F + 0.7E_v + 0.7E_e \text{ (Equation 16-17 17)} \]

\[ D + H + F + 0.525E_v + 0.525E_e + 0.75(L) + 0.75(I_v \text{ or } S \text{ or } R) \text{ (Equation 16-18 18)} \]

\[ 0.6(D + F) - 0.7E_v - 0.7E_e + H \text{ (Equation 16-19 19)} \]

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

\[ D + H + F + 0.7E_v + 0.7E_{em} \text{ (Equation 16-20 20)} \]

\[ D + H + F + 0.525E_v + 0.525E_{em} + 0.75(L) + 0.75(I_v \text{ or } S \text{ or } R) \text{ (Equation 16-21 21)} \]

\[ 0.6(D + F) - 0.7E_v + 0.7E_{em} + H \text{ (Equation 16-22 22)} \]

Exceptions:

1. In Equations 16-18 and 16-21, flat roof snow loads of 30 psf (1.44 kN/m²) or less and roof live loads of 30 psf (1.44 kN/m²) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m²), 20 percent shall be combined with seismic loads.
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:

1605.3.1.2 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.

1605.3.1.3 Other loads. Where flood loads, $F_p$, 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.
IBC®: 1603.1, 1603.1.4

Proponent: Vincent Sagan, Thomas Associates, Inc., representing Metal Building Manufacturers Association (vsagan@mbma.com)

2018 International Building Code

Revise as follows:

1603.1 General. Construction documents shall show the size, section and relative locations of structural members with floor levels, column centers and offsets dimensioned. The design loads and other information pertinent to the structural design required by Sections 1603.1.1 through 1603.1.9 shall be indicated on the construction documents.

Exception: Construction documents for buildings constructed in accordance with the conventional light-frame construction provisions of Section 2308 shall indicate the following structural design information:

1. Floor and roof dead and live loads.
2. Ground snow load, $P_g$.
3. Basic design wind speed, $V$, miles per hour (mph) (km/hr) and allowable stress design wind speed, $V_{st,ad}$, as determined in accordance with Section 1609.3.1 and wind exposure.
4. Seismic design category and site class.
5. Flood design data, if located in flood hazard areas established in Section 1612.3.
6. Design load-bearing values of soils.
7. Rain load data.

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

1. Basic design wind speed, $V$, miles per hour and allowable stress design wind speed, $V_{st,ad}$, as determined in accordance with Section 1609.3.1.
2. Risk category.
3. Wind exposure. Applicable wind direction if more than one wind exposure is utilized.
4. Applicable internal pressure coefficient.
5. Design wind pressures to be used for exterior component and cladding materials not specifically designed by the registered design professional responsible for the design of the structure, psf (kN/m²).

Reason: Showing $V_{st,ad}$ on the construction documents is duplicative, unnecessary, and potentially allows for misapplication of a load combination reduction factor onto a wind pressure developed using an already reduced $V_{st,ad}$ wind velocity. IBC 2021 will be the fourth edition of IBC that shows basic design wind speed in ultimate terms and users should by now be knowledgeable of the changes in wind formatting that took place in ASCE 7-10 and IBC 2012.

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

The code change proposal removes requirements for providing information on the contract documents that is unnecessary. This would not affect the design, thus it would not affect the cost of construction.
2018 International Building Code

Revise as follows:

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

1. Basic design wind speed, $V$, miles per hour and allowable stress design wind speed, $V_{a,sd}$, as determined in accordance with Section 1609.3.1.
2. Risk category.
3. Wind exposure. Applicable wind direction if more than one wind exposure is utilized.
4. Applicable internal pressure coefficient.
5. Design wind pressures to be used for exterior component and cladding materials not specifically designed by the registered design professional responsible for the design of the structure, psf (kN/m$^2$).
6. Roof pressure coefficient (GC$_{Ro}$) zones locations and dimensions.

Reason: In educational sessions conducted by NRCA on IBC’s 2018 roofing-related requirements, participants appear to be notably unclear on ASCE 7-16’s new roof pressure coefficient zones. Adding a description of the roof pressure coefficient zones to Section 1603-Construction Document’s requirements for reporting wind design data (Section 1603.1.4) will add some clarity and should assist in proper roof assembly/covering application.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. The stringency of the code is not increased or decreased.
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.

Proposal #4046
## TABLE 1604.5

### RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

Portions of table not shown remain unchanged.

<table>
<thead>
<tr>
<th>RISK CATEGORY</th>
<th>NATURE OF OCCUPANCY</th>
</tr>
</thead>
<tbody>
<tr>
<td>III</td>
<td>Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to:</td>
</tr>
<tr>
<td></td>
<td>- Spaces in buildings and other structures whose primary occupancy is cumulatively over 10 percent of the building area containing public assembly occupancies with an occupant load greater than 300.</td>
</tr>
<tr>
<td></td>
<td>- Buildings and other structures containing Group E occupancies with an occupant load greater than 250.</td>
</tr>
<tr>
<td></td>
<td>- Buildings and other structures containing educational occupancies for students above the 12th grade with an occupant load greater than 500.</td>
</tr>
<tr>
<td></td>
<td>- Group I-2, Condition 1 occupancies with 50 or more care recipients.</td>
</tr>
<tr>
<td></td>
<td>- Group I-2, Condition 2 occupancies not having emergency surgery or emergency treatment facilities.</td>
</tr>
<tr>
<td></td>
<td>- Group I-3 occupancies.</td>
</tr>
<tr>
<td></td>
<td>- Any other occupancy with an occupant load greater than 5,000. a</td>
</tr>
<tr>
<td></td>
<td>- Power-generating stations, water treatment facilities for potable water, wastewater treatment facilities and other public utility facilities not included in Risk Category IV.</td>
</tr>
<tr>
<td></td>
<td>- Buildings and other structures not included in Risk Category IV containing quantities of toxic or explosive materials that:</td>
</tr>
<tr>
<td></td>
<td>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</td>
</tr>
<tr>
<td></td>
<td>Are sufficient to pose a threat to the public if released. b</td>
</tr>
</tbody>
</table>

### Reason:
This will clarify confusion between a use that is accessory vs. primary for purposes of triggering Risk Level III. The term “primary occupancy” is used and it is not in the definitions; therefore, there is confusion as to how this shall apply. Some reviewers assume “primary occupancy” means that if the public assembly use does not cumulatively total to greater than 50 percent for the entire building, then you do not trigger Risk Category III; whereas, other reviewers are requiring Risk Category III if the public assembly use is over 10 percent of the building area, thereby not an accessory use. This change will clarify that the more conservative requirement of Risk Category III shall be used where public assembly uses over 300 occupants exceed 10 percent (cumulatively) in a building.

### Bibliography:

### Cost Impact:
The code change proposal will increase the cost of construction. The code change proposal may increase the cost of construction depending on how it has been interpreted in a jurisdiction.
## 2018 International Building Code

### TABLE 1604.5
RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

Portions of table not shown remain unchanged.

<table>
<thead>
<tr>
<th>RISK CATEGORY</th>
<th>NATURE OF OCCUPANCY</th>
</tr>
</thead>
</table>
| 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 containing whose primary occupancy is a public assembly occupancy with an occupant load greater than 300.  
• Buildings and other structures containing Group E occupancies with an occupant load greater than 250.  
• Buildings and other structures containing educational occupancies for students above the 12th grade with an occupant load greater than 500.  
• Group I-2, Condition 1 occupancies with 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.

### 1604.5.1 Multiple occupancies.

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

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

**Reason:** For a public assembly occupancy with an occupant load greater than 300, the code currently requires Risk Category III only when this is the primary occupancy of a building, which is inconsistent with Section 1604.5.1 that requires the highest risk category to be used for multiple
occupancy buildings. For example, a 350 occupant occupant convention or event center in its own building would be assigned to Risk Category III, but if this same convention or event center is located within a large hotel, the building would be assigned to Risk Category II since the convention or event center isn't the primary occupancy. However, the hazard to life associated with this high occupant assembly occupancy doesn't change by putting it in a hotel building.

This proposal revises the Risk Category III to include any building that has a high occupant public assembly occupancy, regardless of whether this is the primary occupancy or not. This is consistent with the hazard associated with this type of occupancy and is consistent with Section 1604.5.1 that requires the highest risk category to be used for multiple occupancies. This is also consistent with other occupancy based thresholds for Risk Category III, since none of these other thresholds require the occupancy to be the primary occupancy. Furthermore, "primary occupancy" is not defined and is unenforceable - this proposal removes this unenforceable provision.

**Cost Impact:** The code change proposal will increase the cost of construction
Where a high occupant public assembly occupancy is not a primary occupancy, the cost of construction will increase since the building would now be classified as Risk Category III instead of Risk Category II.
**2018 International Building Code**

Revise as follows:

**TABLE 1604.5**

RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

Portions of table not shown remain unchanged.

<table>
<thead>
<tr>
<th>RISK CATEGORY</th>
<th>NATURE OF OCCUPANCY</th>
</tr>
</thead>
<tbody>
<tr>
<td>III</td>
<td>Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to:</td>
</tr>
<tr>
<td></td>
<td>• Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 500.</td>
</tr>
<tr>
<td></td>
<td>• Buildings and other structures containing Group E occupancies with an occupant load greater than 250.</td>
</tr>
<tr>
<td></td>
<td>• Buildings and other structures containing educational occupancies for students above the 12th grade with an occupant load greater than 500.</td>
</tr>
<tr>
<td></td>
<td>• Group I-2, Condition 1 occupancies with 50 or more care recipients.</td>
</tr>
<tr>
<td></td>
<td>• Group I-2, Condition 2 occupancies not having emergency surgery or emergency treatment facilities.</td>
</tr>
<tr>
<td></td>
<td>• Group I-3 occupancies.</td>
</tr>
<tr>
<td></td>
<td>• Any other occupancy with an occupant load greater than 5,000.(^a)</td>
</tr>
<tr>
<td></td>
<td>• Power-generating stations, water treatment facilities for potable water, wastewater treatment facilities and other public utility facilities not included in Risk Category IV.</td>
</tr>
<tr>
<td></td>
<td>• Buildings and other structures not included in Risk Category IV containing quantities of toxic or explosive materials that:</td>
</tr>
<tr>
<td></td>
<td>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</td>
</tr>
<tr>
<td></td>
<td>Are sufficient to pose a threat to the public if released.(^b)</td>
</tr>
</tbody>
</table>

**Reason:** There is not technical justification for the 300 occupant load threshold. Historically, 300 was used in the 1970 UBC and prior editions at a time when most assembly buildings did not have automatic sprinklers nor seismic design requirements. 300 defined a medium assembly occupancy separating Group B (at the time) into Divisions 1, 2, and 3. Over the last 50 years, the code has gradually evolved away from the medium size assembly concept and the 300 threshold.

More protective sprinkler, fire alarm, interior finish and structural design requirements have made their way into the code over the last 50 years. Today, a 300 assembly occupant load presents no more of a risk than 300 people inside a wholesale retail store, or 300 gathering inside a hotel both of which are Risk Category II under Table 1604.5.

We looked to the life safety egress provisions where 500 is the threshold for when a 3rd exit is required as a good point to upgrade the structural threshold. 500 is also consistent with the current Risk Category III threshold for post 12th grade educational occupancies.

**Bibliography:** 1970 Uniform Building Code - Chapter 6 Requirments for Group A Occupancies; Chapter 7 Requirements for Group B Occupancies

**Cost Impact:** The code change proposal will decrease the cost of construction

This proposal will decrease the cost of construction for structures with an occupant load of less than 500 by eliminating the requirement of seismic analysis and potential additional costs associated with improvements to existing structures.

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\(^a\) \(\text{ICC COMMITTEE ACTION HEARINGS} \cdot \text{April, 2019} \cdot S73\)

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Proposal # 4410
**2018 International Building Code**

Revise as follows:

**TABLE 1604.5**

RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

Portions of table not shown remain unchanged.

<table>
<thead>
<tr>
<th>RISK CATEGORY</th>
<th>NATURE OF OCCUPANCY</th>
</tr>
</thead>
<tbody>
<tr>
<td>III</td>
<td>Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to:</td>
</tr>
<tr>
<td></td>
<td>• Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300.</td>
</tr>
<tr>
<td></td>
<td>• Buildings and other structures containing 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.</td>
</tr>
<tr>
<td></td>
<td>• Buildings and other structures containing Group E occupancies with an occupant load greater than 250.</td>
</tr>
<tr>
<td></td>
<td>• Buildings and other structures containing educational occupancies for students above the 12th grade with an occupant load greater than 500.</td>
</tr>
<tr>
<td></td>
<td>• Group I-2, Condition 1 occupancies with 50 or more care recipients.</td>
</tr>
<tr>
<td></td>
<td>• Group I-2, Condition 2 occupancies not having emergency surgery or emergency treatment facilities.</td>
</tr>
<tr>
<td></td>
<td>• Group I-3 occupancies.</td>
</tr>
<tr>
<td></td>
<td>• Any other occupancy with an occupant load greater than 5,000.¹</td>
</tr>
<tr>
<td></td>
<td>• Power-generating stations, water treatment facilities for potable water, wastewater treatment facilities and other public utility facilities not included in Risk Category IV.</td>
</tr>
<tr>
<td></td>
<td>• Buildings and other structures not included in Risk Category IV containing quantities of toxic or explosive materials that:</td>
</tr>
<tr>
<td></td>
<td>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</td>
</tr>
<tr>
<td></td>
<td>Are sufficient to pose a threat to the public if released.²</td>
</tr>
</tbody>
</table>

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.

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.
IBC®: TABLE 1604.5

Proponent: 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

<table>
<thead>
<tr>
<th>RISK CATEGORY</th>
<th>NATURE OF OCCUPANCY</th>
</tr>
</thead>
</table>
| III           | Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to:  
              | • Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300.  
              | • Buildings and other structures containing Group E or Group I-4 occupancies, or combination therof, 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: The 2015 edition of the IBC has been changed to modify the educational and daycare uses for risk category III in Table 1604.5 to be occupancy based rather than “use” based (see S83-12 attached) as it was in the 2012 IBC. “Buildings containing elementary school, secondary school or day care facilities” has been changed to “Group E occupancies”. However, day care facilities are now classified in both Group E and Group I-4 occupancies. I-4 occupancies are not currently listed in Table 1604.5 which means that occupants attributed to I-4 will be classified under risk category II (Buildings and other structures except those listed in Risk Categories I, III and IV). This occurs even though the I-4 occupancy has a higher relative hazard compared to group E (see IEBC Table 1012.4 attached). The ICC Structural Committee that approved S83-12 may not have realized the loophole that was created when they supported this code change. This code change is needed to insure the safety of children who will be occupying these facilities.

Cost Impact: The code change proposal will increase the cost of construction

Adding Group I-4 occupancies to Risk Category III in IBC Table 1604.5 may result in an increase to the environmental structural loading demands.
(seismic, wind, and snow) to buildings with day care facilities. As stated above, the proposed change is intended to correct an oversight in the 2015 edition of the IBC when the ICC Structural Committee approved the S83-12 proposal. This proposal has no economic impact when compared to the previous code cycle (2012 edition of the IBC).
2018 International Building Code

TABLE 1604.5
RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

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<table>
<thead>
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<th>RISK CATEGORY</th>
<th>NATURE OF OCCUPANCY</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to:</td>
</tr>
<tr>
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<td>• Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300.</td>
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<td>• Buildings and other structures containing Group E occupancies with an occupant load greater than 250.</td>
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</tr>
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<td>III</td>
<td>• Group I-3 occupancies.</td>
</tr>
<tr>
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<td>• Any other occupancy with an occupant load greater than 5,000. a</td>
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</tr>
<tr>
<td></td>
<td>Are sufficient to pose a threat to the public if released. b</td>
</tr>
</tbody>
</table>

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. Where areas of the building are not simultaneously occupied and occupants are accounted for in other areas of the building, the occupant load shall be permitted to be reduced to 25% of the calculated occupant load for the area having the lesser calculated 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: The current definition of NET FLOOR AREA does not provide guidance on which areas should be excluded when determining the occupant load for parking garages. This proposal includes a modification to the definition specifying that drive aisles are not considered to be part of the net floor area. We've also seen many requests to account for non-simultaneously occupied areas so we've added to footnote "a" in the table to address this issue. For example, in an apartment building with a parking garage, it is not possible for one or more occupants to be in their vehicle and in the apartment at the same time. Another example is in an office building where employees are either in their work space or in the break room.

Table 1004.5 is designed to address the means of egress system and it is assumed that all portions of the building are occupied simultaneously. This is necessary so that all rooms and areas of the building will be provided with adequate egress design. That is not the case when it comes to more accurately determining the actual occupant load of the whole building to determine the Risk Category. Rather than considering all areas of the
building to be occupied simultaneously, this proposal allows a reduction to the occupant load for the non-simultaneous use areas where the occupants are otherwise accounted for in other areas of the building. The proponents believe that 25% is a more rational and appropriate ratio than 0% even though 0% may be closer to reality for non-simultaneously occupied areas.

If approved, this code change will reduce the number of alternative materials and methods of construction requests and improve the predictability for design engineers.

**Cost Impact:** The code change proposal will not increase or decrease the cost of construction
This code change will not affect the cost of construction. The purpose of this code change is to clarify how the threshold of the 5,000-person occupant load is determined.

Staff note: A question would be if this allowance conflicts with the exception in Section 1004.5.
S47-19

IBC: 1602.1, 1605.1, 1605.1.1, 1605.2, 1605.2.1, 1605.3, 1605.3.1, 1605.3.1.1, 1605.3.2, 1605.3.2.1, 1607.14

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

2018 International Building Code

SECTION 1605 LOAD COMBINATIONS

Delete and substitute as follows:

1605.1 General. Buildings and other structures and portions thereof shall be designed to resist all of the following:

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

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

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

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

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

Exceptions:

1. The modifications to Load Combinations of ASCE 7 Section 2.3, ASCE 7 Section 2.4, and Section 1605.2 specified in ASCE 7 Chapter 18 and 19 shall apply.
2. When the Allowable Stress Design Load Combinations of ASCE 7 Section 2.4 are used, flat roof snow loads of 30 psf (1.44 kN/m²) and roof live loads of 30 psf (1.44 kN/m²) or less need not be combined with seismic load. Where flat roof snow loads exceed 30 psf (1.44 kN/m²), 20 percent shall be combined with seismic loads.

Revise as follows:

1605.1.1 Stability. Regardless of which load combinations are used to design for strength, where overall structure stability (such as stability against overturning, sliding, or buoyancy) is being verified, use of the load combinations specified in ASCE 7 Section 1605.2 or 1605.3.2, ASCE 7 Section 2.4, and in Section 1605.2 shall be permitted. Where the load combinations specified in ASCE 7 Section 1605.2 or 1605.3.2 are used, strength reduction factors applicable to soil resistance shall be provided by a registered design professional. The stability of retaining walls shall be verified in accordance with Section 1807.2.3.

Delete without substitution:

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

\[
\begin{align*}
&1.0(D + F) + 1.0(L + H) = 0.5L, \text{ or } S \text{ or } R \\
&1.2(D + F) + 1.6(L + H) = 1.0L + S, \text{ or } R \\
&1.2(D + F) + 1.0W + 1.5L = 0.5L, \text{ or } S \text{ or } R
\end{align*}
\]
where:

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

\[ f_r = 0.7 \text{ for roof configurations (such as saw tooth) that do not shed snow off the structure, and } 0.2 \text{ for other roof configurations.} \]

**Exceptions:**

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

1605.2.2 Other loads. Where flood loads, \( F_r \), 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.

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:

\[ 1.2(D + F) + 1.6H + f_r S \]  
\[ 0.9D + 1.6H \]  
\[ 0.9(D + F) + 1.6H \]

**Exceptions:**

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

1605.3.1.1 Stress increases. Increases in allowable stresses specified in the appropriate material chapter or the referenced standards shall not be used with the load combinations of Section 1605.3.1, except that increases shall be permitted in accordance with Chapter 23.

1605.3.1.2 Other loads. Where flood loads, \( F_r \), 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.
of ASCE 7 shall be considered.

Revise as follows:

1605.2 Alternative basic allowable stress design load combinations. In lieu of the basic load combinations specified in Section 1605.3.1 Load Combinations in ASCE 7 Section 2.4, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following combinations. Where using these alternative basic allowable stress load combinations that include wind or seismic loads, allowable stresses are permitted to be increased or load combinations reduced where permitted by the material chapter of this code or the referenced standards. For load combinations that include the counteracting effects of dead and wind loads, only two-thirds of the minimum dead load likely to be in place during a design wind event shall be used. Where using allowable stresses that have been increased or load combinations that have been reduced as permitted by the material chapter of this code or the referenced standards, where wind loads are calculated in accordance with Chapters 26 through 31 of ASCE 7, the coefficient (ω) in the following equations shall be taken as 1.3. For other wind loads, (ω) shall be taken as 1. Where allowable stresses have not been increased or load combinations have not been reduced as permitted by the material chapter of this code or the referenced standards, (ω) shall be taken as 1. Where using these alternative load combinations to evaluate sliding, overturning and soil bearing at the soil-structure interface, the reduction of foundation overturning from Section 12.13.4 in ASCE 7 shall not be used. Where using these alternative basic load combinations for proportioning foundations for loadings, which include seismic loads, the vertical seismic load effect, \( E_v \), in Equation 12.4-4 of ASCE 7 is permitted to be taken equal to zero.

\[
D + L + (L_v, S, R) \\
\text{(Equation 16-17 16-1)}
\]

\[
D + L + 0.6 \omega W \\
\text{(Equation 16-16 16-2)}
\]

\[
D + L + 0.6 \omega W + S/2 \\
\text{(Equation 16-19 16-3)}
\]

\[
D + L + S + 0.6 \omega W/2 \\
\text{(Equation 16-20 16-4)}
\]

\[
D + L + S + E/1.4 \\
\text{(Equation 16-21 16-5)}
\]

\[
0.9D + E/1.4 \\
\text{(Equation 16-22 16-6)}
\]

Exceptions:

1. Crane hook loads need not be combined with roof live loads or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m²) or less and roof live loads of 30 psf (1.44 kN/m²) or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m²), 20 percent shall be combined with seismic loads.
3. Where required by ASCE 7 Chapters 12, 13, and 15, the Load Combinations including overstrength of ASCE 7 Sections 2.3.6 shall be used.

Delete without substitution:

1605.3.2.1 Other loads. Where \( F, H \), or \( T \) are to be considered in the design, each applicable load shall be added to the combinations specified in Section 1605.3.2. Where self-straining loads, \( T \), are considered in the design, their structural effects in combination with other loads shall be determined in accordance with Section 2.4.4 of ASCE 7.

Revise as follows:

1607.14 Crane loads. The crane live load shall be the rated capacity of the crane. Design loads for the runway beams, including connections and support brackets, of moving bridge cranes and monorail cranes shall include the maximum wheel loads of the crane and the vertical impact, lateral and longitudinal forces induced by the moving crane. Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow loads or one-half of the wind load.

1602.1 Notations. The following notations are used in this chapter:

\( D \) = Dead load.

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

\( E \) = Combined effect of horizontal and vertical earthquake induced forces as defined in Section 2.3.4 12.4 of ASCE 7.
F = Load due to fluids with well-defined pressures and maximum heights.

$F_a$ = Flood load in accordance with Chapter 5 of ASCE 7.

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

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

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

R = Rain load.

S = Snow load.

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

$V_{ard}$ = 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.

**Reason:** Since 2000, the IBC has contained three separate groups of load combinations including the following: (1) Strength Load Combinations (1605.2); (2) Basic Allowable Stress Load Combinations (1605.3.1); and (3) Alternative Allowable Stress Load Combinations (1605.3.2). Two of these, the Strength Load Combinations and Basic Allowable Stress Load Combinations are transcribed directly from an earlier edition of the ASCE 7 Standard. The third set of combinations are a legacy from the codes that predate the IBC.

This proposal is intended to remove minor discrepancies in requirements between the IBC and ASCE 7 Standard version of the Strength and Basic Allowable Stress Load Combinations by eliminating the duplication of this material from the IBC. Further, it is intended that removal of the duplicative Strength and Basic Allowable Stress Load Combinations from the IBC will reduce the likelihood of design errors that many engineers have been making when applying the Basic Allowable Stress Design Load Combinations.

The Alternative Allowable Stress Design Load Combinations permit the use of a 1/3 increase in allowable stresses when evaluating Load Combinations containing short-term transient loads including wind and seismic. The Basic Allowable Stress Combinations do not permit this, but instead apply a factor of 0.75 to the transient loads including live, snow, wind, and seismic, when more than one of these loading is considered simultaneously.

The ASCE 7 Load Combinations further permit increases in allowable stresses only when the material, such as wood, has increased available strength under short-term loading, as opposed to long-term loading. These further increases are not intended to be used for the design of masonry, concrete, or steel structures when using the Basic Allowable Stress Design Load Combinations because the strength of these materials does not have significant duration dependence. Unfortunately, and despite specific commentary within the IBC to discourage this, many engineers routinely apply the 1/3 increase to all allowable stresses when designing using the Basic Allowable Stress Design Load Combinations. This creates a potentially dangerous situation in which safety margins of structures designed in this manner are substantially reduced.

By removing the transcription of the ASCE 7 Load Combinations from the IBC, in addition to avoiding duplication of nearly identical material, we expect to reduce the likelihood that design engineers will misapply the 1/3 increase factor applicable to the Alternate Allowable Stress Design Load Combinations. With the approval of this proposal, the IBC will point to ASCE 7 for the Strength and Basic Allowable Stress Design Load Combinations where there is no mention of the 1/3 increase factor. The Alternate Allowable Stress Design Load Combinations will remain in the IBC with the permissible 1/3 increase.

It is important to note that this proposal does not result in any substantive technical change as all Load Combinations presently used by engineers will remain available to them including the exceptions for flat roof snow loads in combinations with seismic loads. The requirement that engineers reference ASCE 7 to determine the Strength and Basic Allowable Stress Design Load Combinations is not burdensome to engineers as they already must reference ASCE 7 to compute the values of the various loadings required by the load combinations for design.

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

This proposed change will not impact the cost of construction. This proposal is a reorganization of the pointers in the IBC to refer to the Load Combinations in the currently referenced loading standard ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7).
**2018 International Building Code**

Revise as follows:

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

\[
D + L + (L, S, R)
\]
(Equation 16-17)

\[
D + L + 0.6 \omega W
\]
\[
D + L + 0.6W
\]
(Equation 16-18)

\[
D + L + 0.6 \omega W + S/2
\]
\[
D + L + 0.6W + S/2
\]
(Equation 16-19)

\[
D + L + S + 0.6 \omega W/2
\]
\[
D + L + S + 0.6W/2
\]
(Equation 16-20)

\[
D + L + S + E/1.4
\]
(Equation 16-21)

\[
0.9D + E/1.4
\]
(Equation 16-22)

**Exceptions:**

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

**Reason:** The material chapters have been revised since the omega factor was introduced in the code to account for some of the material chapters allowing a one-third stress increase on the allowable stresses. This one-third stress increase has been eliminated from the material chapters. Thus, the omega factor is no longer necessary.

**Cost Impact:** The code change proposal will not increase or decrease the cost of construction. This code change proposal will have no effect on the cost of construction.
2018 International Building Code

Revise as follows:

1606.2 Design dead load. Weights of materials of construction. For purposes of design, the actual weights of materials of construction and fixed service equipment shall be used. In the absence of definite information, values used shall be subject to the approval of the building official.

Add new text as follows:

1606.3 Weight of fixed service equipment. In determining dead loads for purposes of design, the weight of fixed service equipment, including the maximum weight of the contents of fixed service equipment, shall be included. The components of fixed service equipment that are variable, such as liquid contents and movable trays, shall not be used to counteract forces causing overturning, sliding, and uplift conditions in accordance with Section 1.3.6 of ASCE 7.

Exceptions:

1. Where force effects are the result of the presence of the variable components, the components are permitted to be used to counter those load effects. In such cases, the structure shall be designed for force effects with the variable components present and with them absent.

2. For the calculation of seismic force effects, the components of fixed service equipment that are variable, such as liquid contents and movable trays, need not exceed those expected during normal operation.

Reason: This proposal coordinates how the weight of fixed service equipment is considered in the IBC with how it is considered in the referenced design load standard, Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7).

Dead load as defined in Section 202 has two parts, the weight of materials of construction and the weight of fixed service equipment. This proposal removes fixed service equipment from Section 1606.2 and creates a new section, Section 1606.3, to contain the provisions specific to fixed service equipment. This aligns the IBC with ASCE 7.

The weight of fixed service equipment includes both the empty weight of the equipment and the maximum weight of the contents. For example, the weight of liquids is to be included in the dead load of piping and tanks and the weight of conduit and wiring is to be included in the dead load of cable trays. The current text of the IBC does not address the dead load due to variable content weight.

In addition, the proposal clarifies that as the content weight is variable, it cannot be counted on to counteract overturning, sliding, and uplift conditions.

The exceptions address counteracting force effects and seismic force effects.

Exception 1 indicates that when the variable content weight is the source of the force causing overturning, sliding, or uplift, it can be used to counteract the force. For example, the liquid in a tank is the primary source of the seismic mass of the tank and therefore can be used to resist seismic uplift, however the liquid can not be used to resist overturning, sliding, and uplift from wind loads.

Exception 2 indicates that the maximum weight of the contents does not have to be used when calculating the seismic forces, rather the weight that exists during normal operation can be used. This is consistent with the methodology used for including variable components in the determination of the seismic weight.

Cost Impact: The code change proposal will not increase or decrease the cost of construction.
S50-19
IBC: 1606.3 (New)

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

2018 International Building Code
Add new text as follows:

1606.3 Photovoltaic panel systems. The weight of photovoltaic panel systems, their support system, and ballast shall be considered as dead load.

Reason: This proposal has two components. First it clarifies the dead load provisions applicable to photovoltaic panel systems. The new section makes it clear that the dead load of photovoltaic panels includes the weight of the support system and ballast. In addition, this proposal coordinates the dead load provisions of the IBC with the dead load provisions contained in the referenced design load standard, Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7).

Cost Impact: The code change proposal will not increase or decrease the cost of construction. This proposal clarifies that photovoltaic panels are dead loads; if a design engineer is accounting for these loads correctly already, it will not impact the cost of construction.

Proposal # 3836
2018 International Building Code

Add new text as follows:

1606.3 Vegetative and landscaped roofs. The weight of all landscaping and hardscaping materials shall be considered as dead load. The weight shall be computed considering both fully saturated soil and drainage layer materials and fully dry soil and drainage layer materials to determine the most severe load effects on the structure.

Revise as follows:

1607.13.3.1 Vegetative and landscaped roofs. The weight of all landscaping materials shall be considered as dead load and shall be computed on the basis of saturation of the soil as determined in accordance with Section 3.1.4 of ASCE 7. The uniform design live load in unoccupied landscaped areas on roofs shall be 20 psf (0.958 kN/m²). The uniform design live load for occupied landscaped areas on roofs shall be determined in accordance with Table 1607.1.

Reason: This proposal has two components. The first component is to move a dead load provision from the live load section of the IBC to the dead load section. The second component is to revise the IBC text regarding the dead load of vegetative roofs to coordinate with the text of the referenced design load standard, Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7).

Component 1: Dead load requirements should not be contained in the live load section of the IBC, Section 1607. This proposal moves text that addresses the dead load of vegetative and landscaped roofs from Section 1607.13.3.1 to a new section in Section 1606 Dead Loads. This change makes the IBC consistent with its own format as well as consistent with ASCE 7.

Component 2: The text pertaining to the dead load provisions for vegetative and landscaped roofs is revised to align with ASCE 7. This includes replacing the reference to ASCE 7 for soil saturation with the actual text from ASCE 7 and adding the term hardscaping. Including both terms, landscaping and hardscaping, is intended to make it clear that the weight of materials such as pavers, stones, and fences, commonly referred to as hardscaping, as well as the weight of soil and plants, commonly referred to as landscaping, is to be considered as dead load on roofs.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. This proposal relocates loads to the correct section and clarifies terminology. There is no technical or substantive change to the cost of construction.
IBC: SECTION 106, 106.1, 106.2, 106.3, 111.5 (New), SECTION 1607 (New), 1607.1 (New), 1607.7.5

Proponent: Ed Kulik, representing ICC Building Code Action Committee (bcac@icc safe.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/m²), 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 well 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.

Proposal # 4052
2018 International Building Code

Revise as follows:

**TABLE 1607.1**

<table>
<thead>
<tr>
<th>OCCUPANCY OR USE</th>
<th>UNIFORM (psf)</th>
<th>CONCENTRATED (pounds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4. Assembly areas</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fixed seats (fastened to floor)</td>
<td>60\textsuperscript{m}</td>
<td></td>
</tr>
<tr>
<td>Follow spot, projections and control rooms</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>Lobbies</td>
<td>100\textsuperscript{m}</td>
<td></td>
</tr>
<tr>
<td>Movable seats</td>
<td>100\textsuperscript{m}</td>
<td></td>
</tr>
<tr>
<td>Stage floors</td>
<td>150\textsuperscript{n}</td>
<td></td>
</tr>
<tr>
<td>Platforms (assembly)</td>
<td>100\textsuperscript{m}</td>
<td></td>
</tr>
<tr>
<td>Reviewing stands, grandstands and bleachers</td>
<td>100\textsuperscript{m}</td>
<td></td>
</tr>
<tr>
<td>Stadiums and arenas with fixed seats (fastened to the floor)</td>
<td>60\textsuperscript{m}</td>
<td></td>
</tr>
<tr>
<td>Other assembly areas</td>
<td>100\textsuperscript{m}</td>
<td></td>
</tr>
<tr>
<td>24. Recreational uses:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bowling alleys, poolrooms and similar uses</td>
<td>75\textsuperscript{m}</td>
<td></td>
</tr>
<tr>
<td>Dance halls and ballrooms</td>
<td>100\textsuperscript{m}</td>
<td></td>
</tr>
<tr>
<td>Gymnasiums</td>
<td>100\textsuperscript{m}</td>
<td></td>
</tr>
<tr>
<td>Ice skating rink</td>
<td>250\textsuperscript{n}</td>
<td></td>
</tr>
<tr>
<td>Reviewing stands, grandstands and bleachers</td>
<td>100\textsuperscript{m}</td>
<td></td>
</tr>
<tr>
<td>Roller skating rink</td>
<td>100\textsuperscript{m}</td>
<td></td>
</tr>
<tr>
<td>Stadiums and arenas with fixed seats (fastened to the floor)</td>
<td>60\textsuperscript{m}</td>
<td></td>
</tr>
</tbody>
</table>

c. Design in accordance with ICC 300.

m. Live load reduction is not permitted.

**Reason**: This proposal contains two changes which align Items 4 and 24 of Table 1607.1 in the IBC with the corresponding table in the referenced design load standard, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7). The changes do not change the magnitude of the live loads nor the footnote references.

Both of the items are moved from Item 24 to Item 4 as they more closely align with the functions contained in Item 4. Reviewing stands/bleachers are similar to Movable seats and Stadiums/arenas with fixed seats are similar to Fixed seats (fastened to floors).

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

This proposal contains editorial changes and clarifications.
TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L0, AND MINIMUM CONCENTRATED LIVE LOADS

Portions of table not shown remain unchanged.

<table>
<thead>
<tr>
<th>OCCUPANCY OR USE</th>
<th>UNIFORM (psf)</th>
<th>CONCENTRATED (pounds)</th>
</tr>
</thead>
</table>

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

Reason: This proposal one of several that are intended to coordinate the live load table in the IBC with the live load table in the referenced design load standard, Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7). The live load table in ASCE 7 no longer has footnotes. The footnotes were removed to make the table more user friendly. The information in the footnotes was moved to new or existing sections in the live load chapter in ASCE 7.

This proposal addresses Footnote G. Footnote G is deleted from Table 1607.1, rather than moved, because it is not needed. This footnote is unnecessary for the following reasons.

1) Footnote G deals with snow loads which are not addressed in Table 1607.1. Snow loads are addressed in IBC Section 1608, and by reference, ASCE 7-16 Chapter 7. ASCE 7-16 Chapter 7 addresses unbalanced snow loads, drifting snow on lower roofs, including snow drift on lower roofs of adjacent structures, drift loads due to roof projections and parapets, and snow loads on existing roofs.

2) Section 1607 Live Loads, contains a section on roof loads, Section 1607.13, which states that in addition to live loads, roofs shall be designed to support dead, wind, snow, and earthquake loads.

3) Section 1605 Load Combinations contains the requirements for combining loads, including combining live, roof live, and snow loads.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. The removal of footnote G will not increase or decrease the cost of construction.
2018 International Building Code

Revised as follows:

**TABLE 1607.1**

**MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L0, AND MINIMUM CONCENTRATED LIVE LOADS**

Portions of table not shown remain unchanged.

<table>
<thead>
<tr>
<th>OCCUPANCY OR USE</th>
<th>UNIFORM (psf)</th>
<th>CONCENTRATED (pounds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6. Catwalks for maintenance access</td>
<td>40</td>
<td>300</td>
</tr>
<tr>
<td>35. Yards and terraces, pedestrian</td>
<td>100²</td>
<td>—</td>
</tr>
</tbody>
</table>

**Reason:**
This proposal is editorial. The proposal contains two changes which align Items 6 and 35 of Table 1607.1 in the IBC with the corresponding table in the referenced design load standard, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7).

The added text, for maintenance access, clarifies the use of the catwalks covered in the table. Catwalks that are part of a public space and accessible to the public are not intended to be included in this item.

The change to Item 35 simply corrects grammar.

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

This is an editorial change and not intended to impact cost of construction.

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Proposal #4136

S55-19
2018 International Building Code

Revise as follows:

### TABLE 1607.1

<table>
<thead>
<tr>
<th>OCCUPANCY OR USE</th>
<th>UNIFORM (psf)</th>
<th>CONCENTRATED (pounds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>14. Fixed ladders</td>
<td>See Section 1607.16</td>
<td></td>
</tr>
</tbody>
</table>

 Portions of table not shown remain unchanged.

Add new text as follows:

**1607.16 Fixed ladders.** Fixed ladders with rungs shall be designed to resist a single concentrated load of 300 lb (1.33 kN) in accordance with Section 4.5.4 of ASCE 7. Where rails of fixed ladders extend above a floor or platform at the top of the ladder, each side rail extension shall be designed to resist a single concentrated load of 100 lb (0.445 kN) in accordance with Section 4.5.4 of ASCE 7. Ships ladders shall be designed to resist the stair loads given in Table 1607.1.

Revise as follows:

**1011.15 Ships ladders.** Ships ladders are permitted to be used in Group I-3 as a component of a means of egress to and from control rooms or elevated facility observation stations not more than 250 square feet (23 m²) with not more than three occupants and for access to unoccupied roofs. The minimum clear width at and below the handrails shall be 20 inches (508 mm). Ships ladders shall be designed for the live loads indicated in Section 1607.16.

**1011.16 Ladders.** Permanent ladders shall not serve as a part of the means of egress from occupied spaces within a building. Permanent ladders shall be constructed in accordance with Section 306.5 of the International Mechanical Code and designed for the live loads indicated in Section 1607.16. Permanent ladders shall be permitted to provide access to the following areas:

1. Spaces frequented only by personnel for maintenance, repair or monitoring of equipment.
2. Nonoccupiable spaces accessed only by catwalks, crawl spaces, freight elevators or very narrow passageways.
3. Raised areas used primarily for purposes of security, life safety or fire safety including, but not limited to, observation galleries, prison guard towers, fire towers or lifeguard stands.
4. Elevated levels in Group U not open to the general public.
5. Nonoccupied roofs that are not required to have stairway access in accordance with Section 1011.12.1.
6. Where permitted to access equipment and appliances in accordance with Section 306.5 of the International Mechanical Code.

**Reason:** This proposal coordinates requirements for fixed ladders in the IBC with the referenced design load standard, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7).* Currently the IBC does not specify live loads to be used in the design of ladders. This proposal adds the ladder live loads contained in ASCE 7-16 to the IBC. The format of the proposed text in new Section 1607.16 mirrors the format of the existing text contained in Sections 1607.8.1 and 1607.8.1.1 where the value of the specified live load is given with a reference to the appropriate section in ASCE 7. This format provides the code user the live load value but leaves the accompanying design information with the referenced standard. This format aids in keeping the two documents coordinated while still providing the fundamental requirement, the load value, within the IBC.

The pointers to Chapter 16 that have been added to Sections 1011.15 and 1011.16 are patterned after the existing structural pointers in the handrail (Section 1014) and guards (Section 1015) IBC sections.

This proposal adds one new item to Table 1607.1. The new item is placed as Item 14, with subsequent items being simply renumbered.

**Cost Impact:** The code change proposal will increase the cost of construction. The cost of fixed ladders may increase due to the addition of the design requirements.
**Proponent:** 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:**

#### TABLE 1607.1

**MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L₀, AND MINIMUM CONCENTRATED LIVE LOADS**

| OCCUPANCY OR USE | UNIFORM (psf) | CONCENTRATED (pounds) | ALSO SEE
|------------------|---------------|-----------------------|-----------
| 14. Garages (passenger vehicles only) | 40° | Note a |
| Passenger vehicles only | 40° | See Section 1607.7 |
| Trucks and buses | | See Section 1607.7-1607.8 |
| 15. Handrails, guards and grab bars | See Section 1607.8 |
| 19. Libraries | |
| Corridors above first floor | 80 | 1,000 |
| Reading rooms | 60 | 1,000 |
| Stack rooms | 150; n | 1,000 | Section 1607.17 |
| 25. Residential | |
| One- and two-family dwellings | |
| Uninhabitable attics without storage | 10 |
| Uninhabitable attics with storage | 20 |
| Habitable attics and sleeping areas | 30 |
| Canopies, including marquees | 20 |
| All other areas | 40 |
| Hotels and multifamily dwellings | |
| Private rooms and corridors serving them | 40 |
| Public rooms and corridors serving them | 100 |
| 26. Roofs | |
| All roof surfaces subject to maintenance workers | 300 |
| Awnings and canopies: | |
| Fabric construction supported by a skeleton structure | 5° |
| All other construction, except one-and two-family dwellings | 20 |
| Ordinary flat, pitched, and curved roofs (that are not occupiable) | 20 |
| Primary roof members exposed to a work floor | |
| Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages | 2,000 |
| All other primary roof members | 300 |
| Occupiable roofs: | |
| Roof gardens | 100 |
Assembly areas

<table>
<thead>
<tr>
<th>100&quot;</th>
<th></th>
</tr>
</thead>
</table>

All other similar areas

<table>
<thead>
<tr>
<th>Note 1</th>
<th>Note 1</th>
</tr>
</thead>
</table>

28. Scuttles, skylight ribs and accessible ceilings

<table>
<thead>
<tr>
<th>200</th>
<th></th>
</tr>
</thead>
</table>

29. Sidewalks, vehicular driveways and yards, subject to trucking

<table>
<thead>
<tr>
<th>250&quot;</th>
<th>8,000&quot;</th>
</tr>
</thead>
</table>

Section 1607.18

30. Stairs and exits

<table>
<thead>
<tr>
<th>40</th>
<th>300&quot;</th>
</tr>
</thead>
</table>

Section 1607.19

One- and two-family dwellings

<table>
<thead>
<tr>
<th>100</th>
<th>300&quot;</th>
</tr>
</thead>
</table>

Section 1607.19

33. Vehicle barriers

| See Section 1607.9 - 1607.10 |

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm²,

1 square foot = 0.0929 m²,

1 pound per square foot = 0.0479 kN/m², 1 pound = 0.004448 kN,

1 pound per cubic foot = 16 kg/m³.

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

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

1. The nominal book stack unit height shall not exceed 90 inches.
2. The nominal shelf depth shall not exceed 12 inches for each face.
3. Parallel rows of double-faced book stacks shall be separated by aisles not less than 36 inches wide.

c. Design in accordance with ICC 300.

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

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

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

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

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

i. Uninhabitable attics without storage are those where the maximum clear height between the joists and rafters is less than 42 inches, or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses. This live load need not be assumed to act concurrently with any other live load requirements.

j. Uninhabitable attics with storage are those where the maximum clear height between the joists and rafters is 42 inches or greater, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches in height by 24 inches in width, or greater, within the plane of the trusses.

The live load need only be applied to those portions of the joists or truss bottom chords where both of the following conditions are met:

1. The attic area is accessible from an opening not less than 20 inches in width by 30 inches in length that is located where the clear height in the attic is not less than 30 inches.
2. The slopes of the joists or truss bottom chords are not greater than two units vertical in 12 units horizontal.
The remaining portions of the joists or truss bottom chords shall be designed for a uniformly distributed concurrent live load of not less than 10 pounds per square foot.

i. Attic spaces served by stairways other than the pull-down type shall be designed to support the minimum live load specified for habitable attics and sleeping rooms.

l. Areas of occupiable roofs, other than roof gardens and assembly areas, shall be designed for appropriate loads as approved by the building official. Unoccupied landscaped areas of roofs shall be designed in accordance with Section 1607.13.3.

m. Live load reduction is not permitted.

n. Live load reduction is only permitted in accordance with Section 1607.11.1.2 or Item 1 of Section 1607.11.2.

o. Live load reduction is only permitted in accordance with Section 1607.11.1.3 or Item 2 of Section 1607.11.2.

Add new text as follows:

1607.7 Passenger vehicle garages. Floors in garages or portions of a building used for the storage of motor vehicles shall be designed for the uniformly distributed live loads indicated in Table 1607.1 or the following concentrated load:

1. For garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 pounds (13.35 kN) acting on an area of 4.5 inches by 4.5 inches (114 mm by 114 mm).

2. For mechanical parking structures without slab or deck that are used for storing passenger vehicles only, 2,250 pounds (10 kN) per wheel.

Revise as follows:

1607.8 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.

Add new text as follows:

1607.17 Library stack rooms. The live loading indicated in Table 1607.1 for library stack rooms applies to stack room floors that support nonmobile, double-faced library book stacks, subject to the following limitations:

1. The nominal book stack unit height shall not exceed 90 inches (2,290 mm).
2. The nominal shelf depth shall not exceed 12 inches (305 mm) for each face.
3. Parallel rows of double-faced book stacks shall be separated by aisles not less than 36 inches (914 mm) wide.

1607.18 Sidewalks, vehicular driveways, and yards subject to trucking. The live loading indicated in Table 1607.1 for sidewalks, vehicular driveways, and yards subject to trucking shall comply with the requirements of this section.

1607.18.1 Uniform loads. In addition to the loads indicated in Table 1607.1, other uniform loads in accordance with an approved method which contains provisions for truck loading, shall be considered where appropriate.

1607.18.2 Concentrated loads. The concentrated wheel load indicated in Table 1607.1 shall be applied on an area of 4.5 inches by 4.5 inches (114 mm by 114 mm).

1607.19 Stair treads. The concentrated load indicated in Table 1607.1 for stair treads shall be applied on an area of 2 inches by 2 inches (51 mm by 51 mm). This load need not be assumed to act concurrently with the uniform load.

1607.20 Residential Attics. The live loads indicated in Table 1607.1 for attics in residential occupancies shall comply with the requirements of this section.

1607.20.1 Uninhabitable attics without storage. In residential occupancies, uninhabitable attic areas without storage are those where the maximum clear height between the joists and rafters is less than 42 inches (1067 mm), or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches (1067 mm) in height by 24 inches (610 mm) in width, or greater, within the plane of the trusses. The live load in Table 1607.1 need not be assumed to act concurrently with any other live load requirement.

1607.20.2 Uninhabitable attics with storage. In residential occupancies, uninhabitable attic areas with storage are those where the maximum clear height between the joist and rafter is 42 inches (1067 mm) or greater, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 inches (1067 mm) in height by 24 inches (610 mm) in width, or greater, within the plane of the trusses. The live load in Table 1607.1 need only be applied to those portions of the joists or truss bottom chords where both of the following conditions are met:

1. The attic area is accessed from an opening not less than 20 inches (508 mm) in width by 30 inches (762 mm) in length that is located where the clear height in the attic is not less than 30 inches (762 mm).
2. The slope of the joists or truss bottom chords is not greater than two units vertical in 12 units horizontal.
The remaining portions of the joists or truss bottom chords shall be designed for a uniformly distributed concurrent live load of not less than 10 pounds per square foot (0.48 kN/m²).

1607.20.3 Attics served by stairs. Attic spaces served by stairways other than the pull-down type shall be designed to support the minimum live load specified for habitable attics and sleeping rooms.

Reason: This proposal is one of several that are intended to coordinate the live load table in the IBC with the live load table in the referenced design load standard, Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7). The live load table in ASCE 7 no longer has footnotes. The footnotes were removed to make the table more user friendly. The information in the footnotes was moved to new or existing sections in the live load chapter of ASCE 7. This proposal is editorial in nature as it does not change the technical requirements, it only reorganizes them. This proposal moves the content of eight footnotes to their own sections within the live load section of the IBC. This proposal also adds a new column to the live load table where the reference to an accompanying section is provided. This change was done to the live load table in ASCE 7-16 and has resulted in a more user friendly table. Footnotes, which are in smaller font than the rest of the table, are not well-suited to contain large amounts of text or to provide technical content. In addition the footnote superscript letter within the table is even smaller and easy to miss. The proposed new column has regular size font and readily alerts the user to the additional information they need to review.

Footnote Changes in this Proposal

Footnote A moved to new Section 1607.7
Footnote B moved to new Section 1607.17
Footnote D moved to new Section 1607.18
Footnote E moved to new Section 1607.18
Footnote F moved to new Section 1607.19
Footnote I moved to new Section 1607.20.1
Footnote J moved to new Section 1607.20.2
Footnote K moved to new Section 1607.20.3

Separate proposals address the other footnotes to Table 1607.1.

Additional notes regarding Footnote A: Footnote A is moved to a new section, Section 1607.7 and the subsequent sections are renumbered. This places the Footnote A text immediately in front of the existing section on heavy live loads, therefore keeping the two sections on garages close to one another (1607.7 for passenger vehicle garages and 1607.8 for heavy vehicle garages). Also note, an editorial change for readability was made to the text, "portions of buildings" was changed to "portions of a building".

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This proposed changes are editorial.
Proponent: 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:

<table>
<thead>
<tr>
<th>OCCUPANCY OR USE</th>
<th>UNIFORM (psf)</th>
<th>CONCENTRATED (pounds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24. Recreational uses:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bowling alleys, poolrooms and similar uses</td>
<td>75 (^m)</td>
<td></td>
</tr>
<tr>
<td>Dance halls and ballrooms</td>
<td>100 (^m)</td>
<td></td>
</tr>
<tr>
<td>Gymnasiums</td>
<td>100 (^m)</td>
<td></td>
</tr>
<tr>
<td>Ice skating rink</td>
<td>250 (^m)</td>
<td></td>
</tr>
<tr>
<td>Reviewing stands, bleachers, folding and telescopic seating and grandstands and bleachers</td>
<td>100 (^m) (see Section 1607.16)</td>
<td></td>
</tr>
<tr>
<td>Roller skating rink</td>
<td>100 (^m)</td>
<td></td>
</tr>
<tr>
<td>Stadiums and arenas with fixed seats (fastened to floor)</td>
<td>60 (^m) (see Section 1607.16)</td>
<td></td>
</tr>
</tbody>
</table>

c. Design in accordance with ICC 300.

Add new text as follows:

1607.16 Seating for assembly uses. Bleachers, folding and telescopic seating and grandstands shall be designed for the loads specified in ICC 300. Stadiums and arenas with fixed seats shall be designed for the horizontal sway loads in Section 1607.16.1.

1607.16.1 Horizontal sway loads. The design of stadiums and arenas with fixed seats shall include horizontal swaying forces applied to each row of seats as follows:

1. 24 lb per linear foot (0.35 kN/m) of seat applied in a direction parallel to each row of seats, and
2. 10 lb per linear foot (0.15 kN/m) of seat applied in a direction perpendicular to each row of seats.

The parallel and perpendicular horizontal swaying forces are not required to be applied simultaneously.

Reason: This proposal is one of several that are intended to coordinate the live load table in the IBC with the live load table in the referenced design load standard, Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7). The live load table in ASCE 7 no longer has footnotes. The footnotes were removed to make the table more user friendly. The information in the footnotes was moved to new or existing sections in the live load chapter of ASCE 7.

This proposal addresses Footnote C by doing the following:

A) Moving Footnote C to a new section within the live load section of the IBC.

B) Removing the reference to ICC 300 for stadiums with fixed seats, but maintaining the requirement to design fixed seats for horizontal sway forces.

C) Revising the terms used for ‘bleachers’ in Table 1607.1 to match the terms used in ICC 300.

A. Footnote C is moved to a new section at the end of the live load section. Footnotes, which are in smaller font than the rest of the table, are not well-suited to contain large amounts of text or to provide technical content. In addition the footnote superscript letter within the table is even smaller and easy to miss. Moving the technical content to its own section makes the table more user friendly.

B. The scope of ICC 300 does not include fixed seats, therefore it is necessary to revise Footnote C. The proposed revision creates two parts. The first part, sentence one in 1607.16, maintains the reference to ICC 300 for ‘bleachers’. The second part, sentence two in 1607.16, requires stadiums
and arenas with fixed seats to be designed for horizontal sway forces (which are the same as the ICC 300 sway forces). These forces are considered fundamental to the design of stadiums and arenas and should be required by the IBC.

C. The terminology used in Table 1607.1 is revised from “Reviewing stands, grandstands and bleachers” to “Bleachers, folding and telescopic seating, and grandstands” in order match the terms used in ICC 300. This is considered to be an editorial change as all of these terms refer to systems that are free-standing, i.e. they are not fixed to the building. However, when referring to a standard, it is preferable for the terms used in the IBC to be consistent with the terms used in the standard. The term “reviewing stand” is not used in ICC 300. If unchanged, the IBC could be interpreted as adding to, or over-riding, the scope of ICC 300, which is not the intent here.

It is noted that the existing IBC text, “Reviewing stands, grandstands and bleachers”, matches the current ASCE 7 text. However the changing the text to “Bleachers, folding and telescopic seating, and grandstands” is necessary for the reasons stated above. The Dead & Live Load Committee of ASCE 7 will be reviewing this terminology and monitoring the outcome of this proposal with the intent of keeping the IBC and ASCE 7 coordinated.

This proposal stands on its own merit, but also coordinates with another ASCE 7 sponsored proposal that adds a new column to Table 1607.1 with the title “See Also Section”. If that proposal is approved, ICC staff has indicated that the reference to new Section 1607.16 will be placed in that new column editorially.

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

This proposal contains editorial changes and clarifications. If the design engineer is accounting for horizontal sway loads correctly already, it will not impact the cost of construction.
### 2018 International Building Code

Revise as follows:

**TABLE 1607.1**

**MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, \( L_0 \), AND MINIMUM CONCENTRATED LIVE LOADS**

Portions of table not shown remain unchanged.

<table>
<thead>
<tr>
<th>OCCUPANCY OR USE</th>
<th>UNIFORM (psf)</th>
<th>CONCENTRATED (pounds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>26. Roofs</td>
<td></td>
<td></td>
</tr>
<tr>
<td>All roof surfaces subject to maintenance workers</td>
<td>300</td>
<td></td>
</tr>
<tr>
<td>Awnings and canopies:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fabric construction supported by a skeleton structure</td>
<td>500</td>
<td></td>
</tr>
<tr>
<td>All other construction, except one- and two-family dwellings</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>Ordinary flat, pitched, and curved roofs (that are not occupiable)</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>Roof areas used for occupants</td>
<td>Same as occupancy served</td>
<td></td>
</tr>
<tr>
<td>Roof areas used for assembly purposes</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Vegetative and landscaped roofs:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof areas not intended for occupancy</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>Roof areas used for assembly purposes</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Roof areas used for other occupancies</td>
<td>Same as occupancy served</td>
<td></td>
</tr>
<tr>
<td>Awnings and canopies:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fabric construction supported by a skeleton structure</td>
<td>500</td>
<td></td>
</tr>
<tr>
<td>All other construction, except one- and two-family dwellings</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>Primary roof members exposed to a work floor:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages</td>
<td>2,000</td>
<td></td>
</tr>
<tr>
<td>All other primary roof members</td>
<td>300</td>
<td></td>
</tr>
<tr>
<td>All roof surfaces subject to maintenance workers</td>
<td>300</td>
<td></td>
</tr>
<tr>
<td>Occupiable roofs:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof gardens</td>
<td>400</td>
<td></td>
</tr>
<tr>
<td>Assembly areas</td>
<td>400</td>
<td></td>
</tr>
<tr>
<td>All other similar areas</td>
<td>Note 1</td>
<td></td>
</tr>
</tbody>
</table>

---

1. Areas of occupiable roofs, other than roof gardens and assembly areas, shall be designed for appropriate loads as approved by the building official. Unoccupied landscaped areas of roofs shall be designed in accordance with Section 1607.13.3.

m. Live load reduction is not permitted.

**Delete without substitution:**

1607.13.3.1 Vegetative and landscaped roofs. The weight of all landscaping materials shall be considered as dead load and shall be computed on the basis of saturation of the soil as determined in accordance with Section 3.1.4 of ASCE 7. The uniform design live load in unoccupied landscaped
areas on roofs shall be 20 psf (0.958 kN/m²). The uniform design live load for occupied landscaped areas on roofs shall be determined in accordance with Table 1607.1.

**Reason:** This proposal coordinates the Roof live load item in Table 1607.1 of the IBC with the Roof live load item in Table 4.3-1 in the referenced design load standard, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7). Both the content and the layout of the Roof item is revised for coordination, including the associated footnote, Footnote L.

The content change consists of the following:

The text contained in Footnote L is replaced with the text "Same as occupancy served", which is placed in the table itself. This change removes the vague language in the footnote, "appropriate loads", and replaces it with more specific language that references the occupancy served by the occupiable roof area. This language is used in other areas of the table and requires the load to be commensurate with occupancy served. Note, no other roof loads are changed, the base roof live load is still 20 psf, and the roof live load for assembly areas is still 100 psf (both in landscaped and non-landscaped areas).

Section 1607.13.3.1 is no longer needed as the table now clearly lists the 20 psf load for unoccupied landscaped areas on roofs, a 100 psf load for assembly areas on roofs, and "Same as occupancy served" for non-assembly occupied areas on roofs. Note, the first sentence in Section 1607.13.3.1 which relates to the dead load of landscaped roofs is moved by another proposal and doesn't belong in the live load section of Chapter 16 anyway.

The layout change consists of placing the base roof live load (20 psf) first, then the other uniform roof loads, and finally the concentrated roof loads. This layout is more logical and follows the layout in the referenced design load standard.

**Cost Impact:** The code change proposal will not increase or decrease the cost of construction. The proposal does not substantially change the roof live loads.
S60-19

IBC®: 1603.1.1, TABLE 1607.1

Proponent: 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>
<thead>
<tr>
<th>OCCUPANCY OR USE</th>
<th>UNIFORM (psf)</th>
<th>CONCENTRATED (pounds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>31. Storage warehouses (shall be designed for heavier loads if required for anticipated storage)</td>
<td></td>
<td>See Section 1603.1.1</td>
</tr>
<tr>
<td>Heavy</td>
<td>250&quot;</td>
<td></td>
</tr>
<tr>
<td>Light</td>
<td>125&quot;</td>
<td></td>
</tr>
</tbody>
</table>

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.

Proposal # 5596
**S61-19**

**IBC®: TABLE 1607.1**

**Proponent:** 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:

TABLE 1607.1

_{MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L0, AND MINIMUM CONCENTRATED LIVE LOADS^2_}

Portions of table not shown remain unchanged.

<table>
<thead>
<tr>
<th>OCCUPANCY OR USE</th>
<th>UNIFORM (psf)</th>
<th>CONCENTRATED (pounds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>31. Storage areas above ceilings</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>32. Storage warehouses (shall be designed for heavier loads if required for anticipated storage)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Heavy</td>
<td>250^o</td>
<td>250^o</td>
</tr>
<tr>
<td>Light</td>
<td>125^o</td>
<td>125^o</td>
</tr>
</tbody>
</table>

**Reason:** This proposal adds a live load to align Table 1607.1 in the IBC with the corresponding table in the referenced design load standard, Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7). Table 1607.1 currently contains live load requirements for residential attic storage in Item 25, however the table does not address storage for other uses. This proposal adds a storage live load for non-residential uses.

**Cost Impact:** The code change proposal will not increase or decrease the cost of construction. This load exists in ASCE 7 so it is not a new load, and should not impact the cost of construction.
**S62-19**

IBC®: TABLE 1607.1

**Proponent:** 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:

**TABLE 1607.1**

MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L₀, AND MINIMUM CONCENTRATED LIVE LOADS

Portions of table not shown remain unchanged.

<table>
<thead>
<tr>
<th>OCCUPANCY OR USE</th>
<th>UNIFORM (psf)</th>
<th>CONCENTRATED (pounds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5. Balconies and decks †</td>
<td>1.5 times the live load for the area served, not required to exceed 100</td>
<td>—</td>
</tr>
</tbody>
</table>

† See Section 1604.8.3 for decks attached to exterior walls.

**Reason:** This proposal is one of several that are intended to coordinate the live load table in the IBC with the live load table in the referenced design load standard, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7). The live load table in ASCE 7 no longer has footnotes. The footnotes were removed to make the table more user friendly. The information in the footnotes was moved to new or existing sections in ASCE 7.

This proposal addresses Footnote H. Footnote H is deleted from Table 1607.1, rather than moved, because it is not needed. This footnote does not address balcony or deck live loads, but rather is simply a pointer to Section 1604.8.3 which contains requirements for anchoring decks to the primary structure. Section 1604.8.3 does not contain any additional live loads. Footnotes to tables are intended to provide information that clarifies the content/requirements of the table and should not be used to provide additional technical content or as pointers.

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

The removal of footnote H will not increase or decrease the cost of construction.

Proposal # 4314
Proponent: 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:

1607.2 Loads not specified. For occupancies or uses not designated in Table 1607.1, Section 1607, the live load shall be determined in accordance with a method approved by the building official.

Reason: This code change is editorial. Changing the reference from the live load table to the entire live load section is more appropriate as not all of the live loads are specified in the table.

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

Editorial change.
2018 International Building Code

1607.3 Uniform live loads. The live loads used in the design of buildings and other structures shall be the maximum loads expected by the intended use or occupancy but shall not be less than the minimum uniformly distributed live loads given in Table 1607.1.

Add new text as follows:

1607.3.1 Live load posting. Where the live loads for which each floor or portion thereof of a commercial building is designed to exceed 100 psf (4.79 kN/m²), such design live loads shall be conspicuously posted by the owner in that part of each story in which they apply using durable signs acceptable to the authority having jurisdiction. It shall be unlawful to remove or deface such notices.

Reason: Buildings constantly undergo change throughout their existence. As changes occur, the original building plans and associated calculations are misplaced or simply lost. The intent of the proposal is to provide future ready reference for building owners, contractors, designers and code enforcement officials.

Cost Impact: The code change proposal will not increase or decrease the cost of construction.
There will be no change to the construction requirements of the building.
2018 International Building Code

Revise as follows:

1607.8 Loads on handrails, guards, grab bars and seats, accessible benches. Handrails and guards shall be designed and constructed for the structural loading conditions set forth in Section 1607.8.1. Grab bars, shower seats and accessible benches shall be designed and constructed for the structural loading conditions set forth in Section 1607.8.2.

1607.8.2 Grab bars, shower seats and dressing room bench seats, accessible benches. Grab bars, and shower seats and dressing room bench seats shall be designed to resist a single concentrated load of 250 pounds (1.11 kN) applied in any direction at any point on the grab bar or seat so as to produce the maximum load effects. Benches in sauna and steam rooms, dressing, fitting and locker rooms, holding cells and housing cells, required to be accessible in ICC A117.1 shall be designed to resist a single concentrated load of 250 pounds (1.11 kN) applied in any direction at any point on the seat of the bench so as to produce the maximum load effects.

Reason: There is inconsistency between the language in two sections. Plus, it must be clear which benches that this requirement is for. The 250 lbs. is required at the dressing room bench because it is a transfer location. The 250 lbs. is in ADA and ICC A117.1 Section 903.6. Transfer benches are required in saunas (612.2), dressing rooms and locker rooms (803.4), jail cells (806.2.2). The original language probably put in ‘dressing rooms’ so people would not try to apply this to any bench anywhere.

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

This is correlation with the requirements in the ICC A117.1 for benches.
IBC®: 1607.8.1.1

Proponent: 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:

1607.8.1.1 Concentrated load. Handrails and guards shall be designed to resist a concentrated load of 200 pounds (0.89 kN) in accordance with Section 4.5.1.1 of 4.5.1 of ASCE 7.

Reason: This proposal corrects a reference to the referenced design load standard, Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7). Section 4.5.1 of ASCE 7 contains the requirements for concentrated loads on handrails and guards, not Section 4.5.1.1.

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

This is an editorial correction to the section number. There is no impact on construction costs.
2018 International Building Code

Revise as follows:

1607.8.1.2 Intermediate rails. Guard component loads. Intermediate rails (all those balusters, panel fillers, and guard infill components, including all rails except the handrail), balusters and panel fillers shall and the top rail, shall be designed to resist a concentrated load of 50 pounds (0.22 kN) in accordance with Section 4.5.1.1 of ASCE 7.

Reason: This proposal coordinates the IBC with the referenced design load standard, Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7). The 50 lb load is intended to apply to the parts of the guard below the top rail. This proposal removes the term intermediate rails and replaces it with guard infill components, as is used in ASCE 7. This proposal also corrects the reference to the section in ASCE 7.

Note, this change is editorial.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This is an editorial change and will not impact the cost of construction.
2018 International Building Code

Revise as follows:

1607.8.2 Grab bars, shower seats and dressing room accessible bench seats. Grab bars, shower seats and dressing room accessible bench seats shall be designed to resist a single concentrated load of 250 pounds (1.11 kN) applied in any direction at any point on the grab bar or seat so as to produce the maximum load effects.

Reason: This change is intended to clarify which seats are required by the IBC to resist the specified concentrated load. This section is not intended to apply to furniture, however the current wording which uses the term dressing room bench seat, is overly broad and can be interpreted to apply to a typical furniture bench placed in a dressing room/bedroom. The intent is to apply to accessible benches. These benches are required in specific locations by ADA and/or ICC A117.1, such as saunas and dressing rooms/locker rooms. The term dressing room is removed and replaced with accessible to clarify the intended type of bench.

Note, accessible is purposely not placed in front of grab bars or shower seats. These items are typically built-in features and need to resist the specified load even if not required for accessibility reasons.

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

Clarifies code intent.
2018 International Building Code

Revise as follows:

1607.10.4 Fall arrest and lifeline anchorages. In addition to any other applicable live loads, fall arrest and lifeline anchorages and structural elements that support these anchorages shall be designed for a live load of not less than 3,100 pounds (13.8 kN) for each attached lifeline, in **any** direction that a fall arrest load can be applied.

**Reason:** This is an editorial change. The word “any” is more grammatically appropriate than “every”. The intent is for a single load to be applied to each anchorage in the worst possible orientation(s) -- one at a time -- not for an infinite number of loads to be applied to a single anchorage at the same time.

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

This is an editorial change. No impact on cost of construction is anticipated or intended.
S70-19

IBC®: 1607.10.4

**Proponent:** Gwenyth Searer, representing myself (gsearer@wje.com)

**2018 International Building Code**

Revise as follows:

1607.10.4 Fall arrest, and lifeline and rope descent system anchorages. In addition to any other applicable live loads, fall arrest and lifeline, and rope descent system anchorages and structural elements that support these anchorages shall be designed for a live load of not less than 3,100 pounds (13.8 kN) for each attached lifeline line, in every direction that a fall arrest the load can be applied.

Anchorages of horizontal lifelines and the structural elements that support these anchorages shall be designed for the maximum tension that develops in the horizontal cable from these live loads.

**Reason:** In January 2017, after more than a decade of public comment, OSHA adopted new regulations in Section 1910.27 (https://www.osha.gov/pls/oshaweb/owadisp.show_document?p_id=9719&p_table=standards) that specifically require all anchorages of rope descent systems (such as boatswain's chairs) to be able to support 5,000 pounds in any direction for each attached worker. Since OSHA has added specific language addressing rope descent systems, and since the systems and loads are basically identical to those for other fall arrest lines, it makes sense to update the current requirements in this section to include rope descent systems. If this change is not made, then ASCE 7 will provide loads for all fall arrest lines and lifelines except safety lines for rope descent systems, which would not make sense. OSHA modifies these regulations very rarely; this is the first change to these regulations since 1971, and further modifications are not expected anytime soon. The language to design horizontal lifeline anchorages and their supports for the loads that develop in the horizontal cable is required to ensure that the anchorage design correctly considers the increases in forces associated with the geometry of the horizontal lifeline.

Examples of boatswain's chairs and how they are used to wash windows and perform routine maintenance on building facades can be found here: https://en.wikipedia.org/wiki/Bosun%27s_chair

**Bibliography:** https://www.osha.gov/pls/oshaweb/owadisp.show_document?p_id=9719&p_table=STANDARDS

https://en.wikipedia.org/wiki/Bosun%27s_chair

**Cost Impact:** The code change proposal will increase the cost of construction

Prior to OSHA’s recent changes, it was technically possible (though unwise) to design half of the anchorages used for rope access to resist just the weight of the attached worker and to design the remainder for fall arrest loads of 3,100 pounds x 1.6 load factor. This wasn't done in practice very often, since workers often cannot tell which anchorages were intended for primary support lines and which anchorages were intended for fall arrest purposes. While there is probably some increase in cost associated with the new OSHA requirements, that increase was mandated by OSHA itself as opposed to this particular code change, which is just intended to convert OSHA loads into loads that are compatible with material design standards and can be properly understood and used by structural and mechanical engineers.

Similarly, the requirement to consider the geometry of the horizontal lifeline may result in higher costs for projects where designers had previously (and incorrectly) ignored the increased loads resulting from the geometry of the horizontal lifeline.

Proposal # 1027
IBC: 1607.13.2, 1607.13.3, 1607.13.3.1

Proponent: 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:

1607.13 Roof loads. The structural supports of roofs and marquees shall be designed to resist wind and, where applicable, snow and earthquake loads, in addition to the dead load of construction and the appropriate live loads as prescribed in this section, or as set forth in Table 1607.1. The live loads acting on a sloping surface shall be assumed to act vertically on the horizontal projection of that surface.

1607.13.1 Distribution of roof loads. Where uniform roof live loads are reduced to less than 20 psf (0.96 kN/m²) in accordance with Section 1607.13.2.1 and are applied to the design of structural members arranged so as to create continuity, the reduced roof live load shall be applied to adjacent spans or to alternate spans, whichever produces the most unfavorable load effect. See Section 1607.13.2 for reductions in minimum roof live loads and Section 7.5 of ASCE 7 for partial snow loading.

1607.13.2 General. Reduction in uniform roof live loads. The minimum uniformly distributed live loads of roofs and marquees, L₀, in Table 1607.1 are permitted to be reduced in accordance with Section 1607.13.2.1.

1607.13.2.1 Ordinary roofs, awnings and canopies. Ordinary flat, pitched and curved roofs, and awnings and canopies other than of fabric construction supported by a skeleton structure, are permitted to be designed for a reduced uniformly distributed roof live load, Lₗ, as specified in the following equations or other controlling combinations of loads as specified in Section 1605, whichever produces the greater load effect.

In structures such as greenhouses, where special scaffolding is used as a work surface for workers and materials during maintenance and repair operations, a lower roof load than specified in the following equations shall not be used unless approved by the building official. Such structures shall be designed for a minimum roof live load of 12 psf (0.58 kN/m²).

\[
L_\ell = L_0 R_1 R_2
\]

where: 12 ≤ Lₗ ≤ 20

For SI:
\[
L_\ell = L_0 R_1 R_2
\]

where: 0.58 ≤ Lₗ ≤ 0.96

L₀ = Unreduced roof live load per square foot (m²) of horizontal projection supported by the member (see Table 1607.1).

Lₗ = Reduced roof live load per square foot (m²) of horizontal projection supported by the member.

The reduction factors R₁ and R₂ shall be determined as follows:

\[
R_1 = 1 \text{ for } A_t \leq 200 \text{ square feet (18.58 m}^2\text{)}
\]

\[
R_1 = 1.2 - 0.001A_t \text{ for 200 square feet}
\]

\[
< A_t < 600 \text{ square feet}
\]

For SI:
\[
R_1 = 1.2 - 0.01A_t \text{ for 18.58 square meters}
\]

\[
< A_t < 55.74 \text{ square meters}
\]

\[
R_1 = 0.6 \text{ for } A_t \geq 600 \text{ square feet (55.74 m}^2\text{)}
\]

where:

\[
A_t = \text{Tributary area (span length multiplied by effective width) in square feet (m}^2\text{) supported by the member, and}
\]

\[
R_2 = 1 \text{ for } F \leq 4
\]

\[
R_2 = 1.2 - 0.05F \text{ for } 4 < F < 12
\]

\[
R_2 = 0.6 \text{ for } F \geq 12
\]

where:

\[
F = \text{For a sloped roof, the number of inches of rise per foot (for SI: } F = 0.12 \times \text{slope, with slope expressed as a percentage), or for an arch or dome, the rise-to-span ratio multiplied by 32.}
\]
**4607.13.3 Occupiable roofs.** Areas of roofs that are occupiable, such as vegetative roofs, roof gardens or for assembly or other similar purposes, and marquees are permitted to have their uniformly distributed live loads reduced in accordance with Section 1607.11.

**4607.13.3.1 Vegetative and landscaped roofs.** The weight of all landscaping materials shall be considered as dead load and shall be computed on the basis of saturation of the soil as determined in accordance with Section 3.1.4 of ASCE 7. The uniform design live load in unoccupied landscaped areas on roofs shall be 20 psf (0.958 kN/m²). The uniform design live load for occupied landscaped areas on roofs shall be determined in accordance with Table 4607.1-1607.1.

**Reason:** This proposal is editorial. It is intended to make the outline format of Section 1607.13, which contains requirements related to roof loads, more clear. This proposal also more closely aligns the format of the roof load provisions in the IBC with the format of the referenced design load standard, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7).

The change to the title of Section 1607.13.2 better reflects the content of this section. The title, General, is appropriate for a section when it is the first numbered item after a primary section title, see Sections 1604, 1605, and 1606. It is not well suited as a title when the section is further down the outline list.

Section 1607.13.3 is renumbered to place it as a subsection under the roof live load reduction section as this section only deals with live load reduction.

Section 1607.13.3.1 is renumbered to make it its own subsection under the roof load section. This section is currently placed as a subsection to occupiable roofs, however it contains provisions not related to occupiable roofs (for unoccupied areas on roofs). This section is better suited as a separate subsection under roof loads.

**Cost Impact:** The code change proposal will not increase or decrease the cost of construction. This proposal is editorial to reorganize and clarify the provisions. There is no impact on construction costs.

Proposal # 4121
IBC®: 1607.13.5.2.1, 1607.13.5.3, 1607.13.5.4


JoeCainPE@gmail.com

2018 International Building Code

Revise as follows:

1607.13.5.2.1 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).

1607.13.5.3 Photovoltaic panels or modules installed as an independent structure. Ground-mounted photovoltaic (PV) panel systems. Solar photovoltaic panels or modules that are independent structures and do not have accessible/occupied space underneath are not required to accommodate a roof photovoltaic live load, provided 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 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 redundancy 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:

1607.13.2 is for rooftop-mounted PV systems

1607.13.3 is for overhead structures with open-grid framing (renumbered from 1607.13.5.2.1)

1607.13.4 is for ground-mounted PV systems (renumbered from 1607.13.5.3)

1607.13.5 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.

Proposal # 5679
2018 International Building Code

Revise as follows:

1607.14.2 Vertical impact force. The maximum wheel loads of the crane shall be increased by the following percentages to determine account for the induced effects of vertical impact or vibration force:

<table>
<thead>
<tr>
<th>Type of Crane</th>
<th>Percentage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monorail cranes (powered)</td>
<td>25 percent</td>
</tr>
<tr>
<td>Cab-operated or remotely operated bridge cranes (powered)</td>
<td>25 percent</td>
</tr>
<tr>
<td>Pendant-operated bridge cranes (powered)</td>
<td>10 percent</td>
</tr>
<tr>
<td>Bridge cranes or monorail cranes with hand-geared bridge, trolley and hoist</td>
<td>0 percent</td>
</tr>
</tbody>
</table>

Reason: This proposal coordinates the text of the crane vertical impact force requirements in the IBC with the referenced design load standard, Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7). This proposal is editorial. It is intended to make the intent of Section 1607.14.2 more clear.

The existing text states that increasing the wheel loads by the given percentage determines the vertical impact or vibration force. However, increasing the wheel loads determines the total load, not just the increase. The proposed text, which is taken from ASCE 7, more clearly indicates the purpose and application of this section.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. This change is editorial and not intended to impact the cost of construction.

Proposal # 4122
2018 International Building Code

Revise as follows:

SECTION 1608
SNOW LOADS

1608.2 Ground snow loads. The ground snow loads to be used in determining the design snow loads for roofs shall be determined in accordance with ASCE 7 or Figure 1608.2 for the contiguous United States and Table 1608.2 for Alaska. Site-specific case studies shall be made in areas designated “CS” in Figure 1608.2. Ground snow loads for sites at elevations above the limits indicated in Figure 1608.2 and for all sites within the CS areas shall be approved. Ground snow load determination for such sites shall be based on an extreme value statistical analysis of data available in the vicinity of the site using a value with a 2-percent annual probability of being exceeded (50-year mean recurrence interval). Snow loads are zero for Hawaii, except in mountainous regions as approved by the building official.

Delete and substitute as follows:
FIGURE 1608.2
GROUND SNOW LOADS, \(p_{gs}\), FOR THE UNITED STATES (psf)

In areas, site-specific Case Studies are required to establish ground snow loads. Extreme local variations of ground snow loads in these areas preclude mapping at this scale.

Numbers in parentheses represent the upper elevation limits in feet for the ground snow load values presented below. Site-specific case studies are required to establish ground snow loads at elevations not covered.

To convert lb/sq ft to kNm\(^2\), multiply by 0.0479.

To convert feet to meters, multiply by 0.3048.
NOTE: See ASCE 7 Tables 7.2-2 for Colorado; see Table 7.2-3 for Idaho; see Table 7.2-4 for Montana; see Table 7.2-5 for Washington; see Table 7.2-6 for New Mexico; see Table 7.2-7 for Oregon; see Table 7.2-8 for New Hampshire.

FIGURE 1608.2
GROUND SNOW LOADS, $p_g$, FOR THE UNITED STATES (psf)
Areas, site-specific Case Studies are required to establish ground snow loads. Extreme local variations in ground snow loads in these areas preclude mapping at this scale.

Numbers in parentheses represent the upper elevation limits in feet for the ground snow load values presented below. Site-specific case studies are required to establish ground snow loads at elevations not covered.

To convert lb/sq ft to kN/m², multiply by 0.0479.

To convert feet to meters, multiply by 0.3048.

FIGURE 1608.2
GROUND SNOW LOADS, $p_{g1}$, FOR THE UNITED STATES (psf)
Reason: This proposed change to Section 1608 Snow Loads will harmonize the provision in the IBC with the 2016 edition of the referenced loading standard ASCE 7 Minimum Design Loads and Associated Criteria For Buildings and Other Structures (ASCE 7-16), which is currently the adopted reference standard. This proposal replaces the current Figure 1608.2 with shown map from ASCE 7-16. The 2016 edition of ASCE 7 has included the basic ground snow map that is unchanged from previous editions, with the exception of seven new state ground snow load tables (Colorado, Idaho, Montana, Washington, New Mexico, Oregon and New Hampshire). The new state tables list the ground snow load and elevation for a number of cities or towns in each state. The tables are based on state ground snow reports by regional experts and state structural engineering associations with specialized knowledge in local climatic conditions. The reports were vetted by the ASCE 7 Snow Loads Subcommittee as having been developed followed appropriate and consistent procedures. The revised map indicates which states have supplemental data within the ASCE 7-16 standard. The new note added in the figure reads: "NOTES: For state tables, see Chapter 7 of ASCE 7; See Table 7.2-2 for Colorado; See Table 7.2-3 for Idaho; See Table 7.2-4 for Montana; See Table 7.2-5 for Washington; See Table 7.2-6 for New Mexico; See Table 7.2-7 for Oregon; See Table 7.2-8 for New Hampshire."

These maps shown are low-resolution per file size limits in cdAcess; high-resolution images are provided to ICC for printing purposes.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
The new ground snow loads included in ASCE 7-16 is the same snow design data currently required by the states. ASCE 7-16 has added it to the standard to become consistent with vetted state requirements. Therefore this data will govern at the state level already and will therefore not impact construction costs.

Proposal # 4162
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. Wind shall be assumed to come from any horizontal direction and wind pressures shall be assumed to act normal to the surface considered.

Exceptions:

1. Subject to the limitations of Section 1609.1.1.1, the provisions of ICC 600 shall be permitted for applicable Group R-2 and R-3 buildings.
2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AWC WFCM.
3. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AISI S230.
5. Designs using TIA-222 for antenna-supporting structures and antennas, provided 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, \( V_{std} \), when the provisions of the standards referenced in Exceptions 4, 5 and 7 are used.

Add new standard(s) as follows:

**AASHTO**

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.


**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.
IBC: SECTION 1610, 1610.1, 1610.2 (New)

**Proponent:** 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 1610**

SOIL LATERAL LOADS AND HYDROSTATIC PRESSURE

1610.1 **General. Lateral pressures.** Foundation walls and retaining walls shall be designed to resist lateral soil loads. Soil loads specified in Table 1610.1 shall be used as the minimum design lateral soil loads unless determined otherwise by a geotechnical investigation in accordance with Section 1803. Foundation walls and other walls in which horizontal movement is restricted at the top shall be designed for at-rest pressure. Retaining walls free to move and rotate at the top shall be permitted to be designed for active pressure. Design lateral pressure from surcharge loads shall be added to the lateral earth pressure load. Design lateral pressure shall be increased if soils at the site are expansive. Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 1805.4.2 and 1805.4.3.

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

Add new text as follows:

1610.2 **Uplift loads on floor and foundations.** Basement floors, slabs on ground, foundations, and similar approximately horizontal elements below grade shall be designed to resist uplift loads where applicable. The upward pressure of water shall be taken as the full hydrostatic pressure applied over the entire area. The hydrostatic load shall be measured from the underside of the construction. The design for upward loads caused by expansive soils shall comply with Section 1808.6.

**Reason:** This proposal coordinates the IBC with the referenced design load standard, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7)* by adding requirements from ASCE 7 to the IBC.

Currently Chapter 16 of the IBC does not address uplift loads, either from hydrostatic pressure or from expansive soils. The addition of these provisions makes it clear that when applicable, the uplift forces shall be included in the design. The text proposed here is taken from Section 3.2.2 of ASCE 7-16 with revisions to account for the fact that the IBC already addresses expansive soil loads in Section 1808.6.

The hydrostatic pressure provision includes the requirement to determine the load based on measuring to the underside of the construction per ASCE 7-16 Section 3.2.2. While this is a relatively basic provision of fluid mechanics, pressure = Specific Weight * the height of the fluid, including this text is intended to prevent the use of floor elevations, top of construction, which are commonly shown on construction drawings.

**Cost Impact:** The code change proposal will increase the cost of construction

The cost of construction will only increase for those designs which did not previously consider uplift forces.

Proposal # 3941
2018 International Building Code

Revise as follows:

1610.1 General. Foundation walls and retaining walls shall be designed to resist lateral soil loads from adjacent soil. Soil loads specified in Table 1610.1 shall be used as the minimum design lateral soil loads unless determined otherwise by a geotechnical investigation in accordance with Section 1803. Foundation walls and other walls in which horizontal movement is restricted at the top shall be designed for at-rest pressure. Retaining walls free to move and rotate at the top shall be permitted to be designed for active pressure. Design lateral pressure from surcharge loads shall be added to the lateral earth pressure soil load. Design lateral pressure shall be increased if expansive soils are present at the site are expansive. Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 1805.4.2 and 1805.4.3.

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

Reason: This proposal contains editorial changes to the soil lateral load section in the IBC to coordinate the text of the IBC with the referenced design load standard, Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7). The proposed changes are editorial and do not change the technical requirements in the IBC. This proposal replaces the word earth with the word soil to be consistent with the term used throughout the section. It also changes the term design lateral pressure to lateral pressure where appropriate, such as when the pressure is being referred to in a general sense.

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

This proposal contains editorial changes and clarifications.
S78-19

IBC: 1610.1.1 (New)

Proponent: Terry Kozlowski, representing Southern Nevada Chapter (kozlowskit@cityofnorthlasvegas.com); Valarie Evans, 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

1610.1 General. Foundation walls and retaining walls shall be designed to resist lateral soil loads. Soil loads specified in Table 1610.1 shall be used as the minimum design lateral soil loads unless determined otherwise by a geotechnical investigation in accordance with Section 1803. Foundation walls and other walls in which horizontal movement is restricted at the top shall be designed for at-rest pressure. Retaining walls free to move and rotate at the top shall be permitted to be designed for active pressure. Design lateral pressure from surcharge loads shall be added to the lateral earth pressure load. Design lateral pressure shall be increased if soils at the site are expansive. Foundation walls shall be designed to support the weight of the full hydrostatic pressure of undrained backfill unless a drainage system is installed in accordance with Sections 1805.4.2 and 1805.4.3. Foundation walls extending not more than 8 feet (2438 mm) below grade and laterally supported at the top by flexible diaphragms shall be permitted to be designed for active pressure.

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

Add new text as follows:

1610.1.1 Seismic load due to lateral earth pressure. All basement, foundation, and retaining walls shall be designed to resist the seismic load due to the lateral earth pressure based on the following equations:

For yielding walls: $F_{eq} = \frac{3}{8} (k_u) (\text{backfill soil unit weight}) (H)^2$ (Equation 16-35)

For nonyielding walls: $F_{eq} = (k_u) (\text{backfill soil unit weight}) (H)^2$ (Equation 16-36)

Where $k_u$ (peak ground acceleration) = $S_{ps} / 2.5$

$H$ = the height of the backfill behind the wall

$F_{eq}$ = the minimum seismic inducing force

These equations represent the dynamic (seismic) lateral thrust. The point of application of the resultant dynamic thrust is taken at a height of 0.6H above the base of the wall. This is represented as an inverted trapezoidal pressure distribution. These equations apply to level backfill and walls that retain no more than 15 feet.

Reason: An owner/builder or contractor should not make the determination of soil seismic load. Rather than utilizing soil classification, the seismic load due to lateral earth pressure is required to clarify the retaining wall earthquake loading necessary to determine the factor of safety required in section 1807.2.3. Section 1807.2.2 refers to Section 1610 for the design lateral soil loads. This proposal is necessary to determine the soil seismic load when a geotechnical report is not required given Table 1610.1 is not adequate and doesn’t address soil seismic load.

There are several theories with respect to seismic accelerations on a retaining wall and the Seed-Whitman method is frequently used. The geotechnical engineer should be allowed to specify the appropriate pressure to be used in the design of the retaining wall based on the site investigation rather than the assumption of at-rest pressure.

Cost Impact: The code change proposal will decrease the cost of construction

By using the equations provided, a geotechnical report may not be required.

Proposal # 4387
S79-19

IBC: 1611.1, 1611.2

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

2018 International Building Code

SECTION 1611
RAIN LOADS

Revise as follows:

1611.1 Design rain loads. Each portion of a roof shall be designed to sustain the load of rainwater that will accumulate on it if 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, as per the requirements of Chapter 8 of ASCE 7. The design rainfall shall be based on the 100-year hourly rainfall rate indicated in Figure 1611.1-15-minute duration event, or on other rainfall rates determined from approved local weather data. Alternatively, a design rainfall of twice the 100-year hourly rainfall rate indicated in Figure 1611.1 shall be permitted.

\[ R = 5.2(d_h + d_s) \]

(Equation 16-35)
For SI: \[ R = 0.0098(d_s + d_h) \]

where:

- \( d_h \) = Additional depth of water on the undeflected roof above the inlet of secondary drainage system at its design flow (in other words, the hydraulic head), in inches (mm).
- \( d_s \) = Depth of water on the undeflected roof up to the inlet of secondary drainage system when the primary drainage system is blocked (in other words, the static head), in inches (mm).

\( R \) = Rain load on the undeflected roof, in psf (kN/m²). Where the phrase “undeflected roof” is used, deflections from loads (including dead loads) shall not be considered when determining the amount of rain on the roof.

1611.2 Ponding instability. Susceptible bays of roofs shall be evaluated for ponding instability in accordance with Section 6.4 Chapter 7 and Chapter 8 of ASCE 7.

Reason: This proposed changes to Section 1611 will harmonize the provision in the IBC with the currently referenced loading standard ASCE 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE 7).

The proposed changes to secondary (overflow) system design harmonized the roof load design for the structure with the expectations for the design of the roof drainage system. This proposal coordinates the IBC with ASCE 7, which was updated to be consistent with the International Plumbing Code (IPC) provisions. The changes provide a basis for the design mean reoccurrence interval and duration for determining the Hydraulic Head (dh). Currently the IBC requires the calculation of dh; however, the code does not state the design storm (mean reoccurrence interval and duration) for determining the design rain load (depth of water on the undeflected roof) and it has led to some confusion. Typical design values for plumbing systems have been between 15 minute and 60 minutes; the 1995 IPC first used the 100-year/60-minute duration for the design of the primary drainage system and twice the flow rate from the 100-year/60-minute duration storm for the secondary drainage system.

Note that the use of twice the flow rate of the 60-minute duration is close to the design intensity for the 15-minute duration storm. The IPC also used a 15-minute duration rainfall event for the design of roof drainage systems. Therefore, by adding this as an alternative the data within Figure 1611.1 is permitted to be used.

The basis for the use of a 60-minute duration storm is unclear - the critical duration for most roof geometries is closer to 15 minutes. Graber (2009) provides guidance for determining the critical duration and the paper advises against the use of the 60-minute storm for the design of the primary and secondary drainage systems in hopes of handling the critical short-duration rainfall event.

NOTE that ASCE 7 does not provide rainfall data or maps for determing the rainfall rate. The best source currently is the National Oceanic and Atmospheric Administration (NOAA) National Weather Service Precipitation Frequency Data Server - Hydrometeorological Design Studies Center (http://hdsc.nws.noaa.gov/hdsc/pfds/index.html) for precipitation intensity (inches per hour) based on the 100-year mean reoccurrence interval.

**Cost Impact:** The code change proposal will increase the cost of construction
The proposed changes may impact the design of roofs where the secondary (overflow) system was previously based on an unconservative hydraulic head from a lower rainfall intensity. The changes harmonizes the roof load design for the structure with the expectations for the design of the roof drainage system. This proposal coordinates the IBC with the referenced loading standard ASCE 7, which was updated in the 2016 edition to be consistent with the International Plumbing Code provisions for secondary drainage systems.
**Proponent:** Gregory Wilson, representing Federal Emergency Management Agency (gregory.wilson2@fema.dhs.gov); Rebecca Quinn, RCQuinn Consulting, on behalf of Federal Emergency Management Agency, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

### 2018 International Building Code

**Revise as follows:**

**1612.4 Flood hazard documentation.** The following documentation shall be prepared and sealed by a registered design professional and submitted to the building official:

1. For construction in flood hazard areas other than coastal high hazard areas or coastal A zones:
   1.1. The elevation of the lowest floor, including the basement, as required by the lowest floor elevation inspection in Section 110.3.3 and for the final inspection in Section 110.3.11.1.
   1.2. For fully enclosed areas below the design flood elevation where provisions to allow for the automatic entry and exit of floodwaters do not meet the minimum requirements in Section 2.7.2.1 of ASCE 24, construction documents shall include a statement that the design will provide for equalization of hydrostatic flood forces in accordance with Section 2.7.2.2 of ASCE 24.
   1.3. For dry floodproofed nonresidential buildings, construction documents shall include a statement that the dry floodproofing is designed in accordance with ASCE 24 and shall include the flood emergency plan specified in Chapter 6 of ASCE 24.

2. For construction in coastal high hazard areas and coastal A zones:
   2.1. The elevation of the bottom of the lowest horizontal structural member as required by the lowest floor elevation inspection in Section 110.3.3 and for the final inspection in Section 110.3.11.1.
   2.2. Construction documents shall include a statement that the building is designed in accordance with ASCE 24, including that the pile or column foundation and building or structure to be attached thereto is designed to be anchored to resist flotation, collapse and lateral movement due to the effects of wind and flood loads acting simultaneously on all building components, and other load requirements of Chapter 16.
   2.3. For breakaway walls designed to have a resistance of more than 20 psf (0.96 kN/m²) determined using allowable stress design, construction documents shall include a statement that the breakaway wall is designed in accordance with ASCE 24.

**Reason:** This proposal emphasizes the requirement for a flood emergency plan consistent with ASCE 24 and makes clear that such a plan, when indicated, is to be submitted with other flood hazard documentation. ASCE 24 requires the submittal and approval of a flood emergency plan where dry floodproofing measures requiring human intervention are used. ASCE 24 requires flood emergency plans to specify the storage location of the shields, the method of installation, conditions activating installation, maintenance of shields and attachment devices, periodic practice of installing shields, testing sump pumps and other drainage measures, and inspecting necessary material and equipment to activate or implement floodproofing. The design professional developing dry floodproofing measures that require human intervention should take into consideration the effort needed to effectively deploy such measures. Preparation of a flood emergency plan ensures that the methods specified by the design professional can be installed and implemented within the given warning time. If a design requires more warning time than reasonably available before the onside of flooding, then the designer should interpret that to mean the contemplated dry floodproofing measures must be redesigned, or that dry floodproofing may not be appropriate for the building. Additionally, maintenance, testing, and inspection are critical to ensuring system performance. The possible inability of owners or occupants to implement dry floodproofing due to lack of preparation or maintenance is regarded as an unacceptable risk.

After Hurricanes Harvey and Irma, FEMA Mitigation Assessment Teams (MATs) observed dry floodproofing measures that failed for a variety of reasons directly related to inadequate deployment or improper maintenance, validating the ASCE 24 requirement. Challenges included systems that required sizeable crews with heavy and specialized equipment to mobilize over a period of several days in advance of the storm to properly install the system. Lack of maintenance of gaskets around doors and flood shields contributed to water intrusion. Lack of inspection and owner/manager awareness of components integral to dry floodproofing meant inadvertent alterations (in one case, a large opening had been cut into a concrete wall to install new utility lines and was not restored to watertight condition). The MATs observed failures and difficulties related to storage (e.g., storage outside where ultraviolet radiation and temperature extremes degrade rubber seals, gaskets, and component identification labels; unsecured storage locations vulnerable to theft and vandalism).

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

No additional cost. Flood emergency plans are already required by ASCE 24 when designs for dry floodproofing are prepared.
Revise as follows:

1612.4 Flood hazard documentation. The following documentation shall be prepared and sealed by a registered design professional and submitted to the building official:

1. For construction in flood hazard areas other than coastal high hazard areas or coastal A zones:
   - The elevation of the lowest floor, including the basement, as required by the lowest floor elevation inspection in Section 110.3.3 and for the final inspection in Section 110.3.11.1.
   - For fully enclosed areas below the design flood elevation where provisions to allow for the automatic entry and exit of floodwaters do not meet the minimum requirements in Section 2.7.2.1 of ASCE 24, construction documents shall include a statement that the design will provide for equalization of hydrostatic flood forces in accordance with Section 2.7.2.2 of ASCE 24.
   - For dry floodproofed nonresidential buildings, construction documents shall include a statement that the dry floodproofing is designed in accordance with ASCE 24.

2. For construction in coastal high hazard areas and coastal A zones:
   - The elevation of the bottom of the lowest horizontal structural member as required by the lowest floor elevation inspection in Section 110.3.3 and for the final inspection in Section 110.3.11.1.
   - Construction documents shall include a statement that the building is designed in accordance with ASCE 24, including that the pile or column foundation and building or structure to be attached thereto is designed to be anchored to resist flotation, collapse and lateral movement due to the effects of wind and flood loads acting simultaneously on all building components, and other load requirements of Chapter 16.
   - For breakaway walls designed to have a resistance of more than 20 psf (0.96 kN/m²) determined using allowable stress design, construction documents shall include a statement that the breakaway wall is designed in accordance with ASCE 24.
   - For breakaway walls where provisions to allow for the automatic entry and exit of floodwaters do not meet the minimum requirements in Section 2.7.2.1 of ASCE 24, construction documents shall include a statement that the design will provide for equalization of hydrostatic flood forces in accordance with Section 2.7.2.2 of ASCE 24.

Reason: For construction in flood hazard areas, the 2018 IBC refers to the 2014 edition of ASCE 24, Flood Resistant Design and Construction. ASCE 24 requires openings in breakaway walls in all flood hazard areas (as does the IRC, in Section R322.2.2 and R322.3.6). Flood openings may be non-engineered (providing 1 square inch of net open area for each square foot of enclosure area) or engineered. Certification of engineered openings is a requirement of the NFIP (and IRC Section R322.2.2). Currently, Section 1612.4 only requires certification of engineered openings in flood hazard areas other than coastal high hazard areas or coastal A Zones (Item 1 of Section 1612.4).
This proposal specifies that construction documents include certification of engineered openings when used in breakaway walls in coastal high hazard areas and coastal A Zones.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
No additional cost because certification of engineered openings has always been required by the NFIP.
S82-19

IBC: 1702.1, 1703.1.2, 1703.1.3

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

2018 International Building Code

Revise as follows:

1702.1 General. New building materials, equipment, appliances, systems or methods of construction not provided for in this code, and any material of questioned suitability proposed for use in the construction of a building or structure, shall be subjected to the tests prescribed in this chapter and in the approved rules Section 104.11 to determine character, quality and limitations of use.

1703.1.2 Equipment. An approved agency shall have adequate equipment to perform required tests. The equipment shall be periodically calibrated with a frequency as appropriate to the equipment type and associated industry standard as defined by the building official.

1703.1.3 Personnel. An approved agency shall employ experienced personnel educated in conducting, supervising and evaluating tests and special inspections, or furnishing inspection services or both.

Reason: The phrase “approved rules” is not defined in Chapter 2 of the IBC; however, the word “approved” is defined. This proposal would utilize the existing code language of Chapter 1 to better define the necessary test for new building materials, equipment, appliances, systems, or methods no currently provided for in the code. The word “periodically” as used in 1703.1.2 may lead to confusion as that word is most commonly associated with periodic special inspections as used in Chapter 17. Since the intent of 1703.1.2 is to broadly cover various types of equipment, all of which may require calibration at different frequency intervals, the above language is proposed to clarify the code requirements.

Approved agency is defined in Chapter 2 of the IBC, and the definition makes clear that this agency may conduct tests or furnish inspection services. The current version of the code text in 1703.1.3 implies that an approved agency must employ personnel that both perform tests and furnish inspection services, although testing agencies come in various forms, with some performing tests, some furnishing inspection services, and some doing both. With this clarification, the discrepancy between the Chapter 2 definition and the Chapter 17 code language can be resolved.

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

This is a clarification to the 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 included in the IBC to assist the jurisdictions in having the language properly incorporated into their respective codes. For example, the Georgia Building Code now includes 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.
S84-19

IBC: 1704.2, 1704.3.1

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

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 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. The contractor is permitted to employ the approved agencies where the contractor is also the owner.

1704.3.1 Content of statement of special inspections. The statement of special inspections shall identify the following:

1. The materials, systems, components and work required to have special inspections or tests by the building official or by the registered design professional responsible for each portion of the work.
2. The type and extent of each special inspection.
3. The type and extent of each test.
4. Additional requirements for special inspections or tests for seismic or wind resistance as specified in Sections 1705.11, 1705.12 and 1705.13.
5. For each type of special inspection, identification as to whether it will be continuous special inspection, periodic special inspection or performed in accordance with the notation used in the referenced standard where the inspections are defined.
6. The approved agency responsible for performing each test and special inspection.

Reason: This proposal requires that agencies responsible for each test and special inspection be identified within the Statement of Special Inspections documentation. Identifying the agencies on the Statement of Special Inspections ensures that the registered design professional(s) who prepares the Statement, the permit applicant who submits the Statement, the special inspectors and contractors who refer to the Statement for extent of inspections, the owner who identifies the agencies to be employed, and the building official to whom the Statement is submitted for approval; are all aware of who the inspection agencies will be.

Identifying which agency is responsible for each test or special inspection also makes it easier for the contractor to identify what agency needs to be contacted when work is ready for inspection.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. This is only a clarification.
Add new definition as follows:

**QUALITY ASSURANCE COORDINATOR.** An agency or individual responsible for coordinating tests, special inspections, approval records, reports and submittals related to the requirements of chapter 17.

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. The contractor is permitted to employ the approved agencies where the contractor is also the owner.

Add new text as follows:

**1704.2.1 Quality Assurance Coordinator.** The owner or the owner’s authorized agent shall employ a Quality Assurance Coordinator to perform the following administrative tasks:

1. Verify that each of the tests and special inspections identified in the statement of special inspections have been completed and that reports identify status of compliance with the construction documents.
2. Verify qualifications of special inspectors.
3. Collect approval records per Sections 1703.2 and 1703.3 and reports of tests and special inspections. Submit those records and reports to the building official, the registered design professional responsible for the design, the contractor, and the owner.
4. Maintain a log of discrepancies discovered, including dates that the discrepancy was corrected or otherwise resolved.
5. Report uncorrected discrepancies to the building official and the registered design professional responsible for the design prior to the completion of the applicable phase of work.
6. Coordinate status of approved fabricators with the building official and notify the contractor, the registered design professional responsible for the design, testing agencies, and special inspectors of applicable exemptions from tests and special inspections.
7. Collect contractor’s statements of responsibility per Section 1704.4 and submit to the building official.
8. Collect submittals identified in Section 1704.5 and submit to the building official for approval.
9. Collect reports of structural observations and submit those records to the building official.
10. Notify the contractor, the registered design professional responsible for the design, testing agencies, and special inspectors of any submittals that are not approved by the building official.
11. Administer supplemental statements of special inspections prepared by registered design professionals responsible for the design of deferred submittal items per the tasks listed above.

**1704.2.2 Quality Assurance Coordinator qualification.** The Quality Assurance Coordinator shall be independent from the contractor unless the contractor is also the owner. The Quality Assurance Coordinator is not required to be a registered design professional. The building official is permitted to perform the tasks listed in 1704.2.1 and waive the requirement for a Quality Assurance Coordinator.

Revise as follows:

**1704.2.7.1 Fabricator approval.** Special inspections during fabrication are not required where the work is done on the premises of a fabricator approved to perform such work without special inspection. Approval shall be based on review of the fabricator’s written fabrication procedures and quality control manuals that provide a basis for control of materials and workmanship, with periodic auditing of fabrication and quality
control practices by an approved agency or the building official. At completion of fabrication, the approved fabricator shall submit a certificate of compliance to the owner or the owner’s authorized agent Quality Assurance Coordinator for submittal to the building official as specified in Section 1704.5 stating that the work was performed in accordance with the approved construction documents.

1704.3.1 Content of statement of special inspections. The statement of special inspections shall identify the following:

1. The materials, systems, components and work required to have special inspections or tests by the building official or by the registered design professional responsible for each portion of the work.
2. The type and extent of each special inspection.
3. The type and extent of each test.
4. Additional requirements for special inspections or tests for seismic or wind resistance as specified in Sections 1705.11, 1705.12 and 1705.13.
5. For each type of special inspection, identification as to whether it will be continuous special inspection, periodic special inspection or performed in accordance with the notation used in the referenced standard where the inspections are defined.
6. The Quality Assurance Coordinator.

1704.4 Contractor responsibility. Each contractor responsible for the construction of a main wind- or seismic force-resisting system, designated seismic system or a wind- or seismic force-resisting component listed in the statement of special inspections shall submit a written statement of responsibility to the Quality Assurance Coordinator for submittal to the building official and the owner or the owner’s authorized agent prior to the commencement of work on the system or component. The contractor’s statement of responsibility shall contain acknowledgement of awareness of the special requirements contained in the statement of special inspections.

1704.5 Submittals to the building official. In addition to the submittal of reports of special inspections and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner’s authorized agent Quality Assurance Coordinator to the building official for each of the following:

1. Certificates of compliance for the fabrication of structural, load-bearing or lateral load-resisting members or assemblies on the premises of an approved fabricator in accordance with Section 1704.2.5.1.
2. Certificates of compliance for the seismic qualification of nonstructural components, supports and attachments in accordance with Section 1705.13.2.
3. Certificates of compliance for designated seismic systems in accordance with Section 1705.13.3.
4. Reports of preconstruction tests for shotcrete in accordance with Section 1908.5.
5. Certificates of compliance for open web steel joists and joist girders in accordance with Section 2207.5.
6. Reports of material properties verifying compliance with the requirements of AWS D1.4 for weldability as specified in Section 26.6.4 of ACI 318 for reinforcing bars in concrete complying with a standard other than ASTM A706 that are to be welded.
7. Reports of mill tests in accordance with Section 20.2.2.5 of ACI 318 for reinforcing bars complying with ASTM A615 and used to resist earthquake-induced flexural or axial forces in the special moment frames, special structural walls or coupling beams connecting special structural walls of seismic force-resisting systems in structures assigned to Seismic Design Category B, C, D, E or F.

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. 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 Quality Assurance Coordinator for submittal 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 Quality Assurance Coordinator for submittal 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: This proposal adds a requirement that there be one central point of contact for construction quality assurance related items, similar to the role of the “Registered Design Professional in Responsible Charge” for design related items. Chapter 17 currently directs submittals to either the building official or the owner. It does not limit the number of statements of special inspection, special inspection agencies, or structural observers. Records and reports can come from numerous sources and it is unlikely that the building official and/or owner will have regular contact with all of those sources. It is logistically difficult for the building official to keep track of all the submittals required by Chapter 17. Defining a role for one agency or individual to track the completion of the specified testing and special inspections, distribute records, and communicate important issues to other project roles ensures that the complete program of tests and inspections is implemented as Chapter 17 intends.

The defined role is purely administrative. It can be performed by any number of entities and doesn’t have to be filled by someone who isn’t already working on the job. The important thing is that someone is clearly responsible for looking at the whole program, making sure it is completed, and acting as a central point of contact for the approvals, testing, and special inspection scope of Chapter 17.
Cost Impact: The code change proposal will not increase or decrease the cost of construction. An increase in cost of construction should not occur because all of these activities are being performed. This proposal only identifies that person should the Building Official desire.
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.
S87-19

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

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 conducting 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 proposal, 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.
S88-19

IBC®: 1704.3.1

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

2018 International Building Code

Revise as follows:

1704.3.1 Content of statement of special inspections. The statement of special inspections shall identify the following:

1. The materials, systems, components and work required to have special inspections or tests by the building official or by the registered design professional responsible for each portion of the work.
2. The type and extent of each special inspection.
3. The type and extent of each test.
4. Additional requirements for special inspections or tests for seismic or wind resistance as specified in Sections 1705.11, 1705.12 and 1705.13.
5. For each type of special inspection, identification as to whether it will be continuous special inspection, periodic special inspection or performed in accordance with the notation used in the referenced standard where the inspections are defined.
6. Deferred submittals in accordance with section 107.3.4.1 that require a supplemental statement of special inspections to be prepared, including identification of the registered design professional to be responsible for the supplemental statement.

Reason: There is often confusion regarding responsibility for preparing a statement of special inspections when the design for a particular scope of design work is deferred to construction. It can lead to portions of the structure not receiving special inspection and arguments about "signing off" on a project because the scope doesn't get identified. This proposal requires that the statement of special inspections that is submitted as a condition for permit approval clearly identifies scope of work which requires a supplemental specification of tests and special inspections, and identifies who is responsible for providing it so that the Building Official (or other party) can follow-up, where necessary.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. No change in work or cost of work is anticipated. This clarifies responsibilities already in the code.
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 responsible for structural design, or their designated agent, shall perform structural observations. Structural observation does not include or waive the responsibility for the inspections in Section 110 or the special inspections in Section 1705 or other sections of this code. Prior to the commencement of observations, the structural observer shall submit to the building official a written statement identifying the frequency and extent of structural observations.

At the conclusion of the work included in the permit, the structural observer shall submit to the building official a written statement that the site visits have been made and identify any reported deficiencies that, to the best of the structural observer’s knowledge, have not been resolved.

Reason: The structural observer reviews as-built construction for general conformance to the project design intent. It is best that the registered design professional responsible for the structural design perform structural observations because that professional, usually referred to as the structural Engineer of Record (EOR), is most familiar with the design intent. In the event that the EOR is not able to perform the observations, then the EOR should designate a qualified individual/agency with experience designing projects of similar type and complexity to perform that work in their place.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
For the vast majority of projects, there will be no increase in the cost of construction as structural engineers already perform this task.
Proponent: Gregory Robinson, representing National Council of Structural Engineers Associations (NCSEA) (grobinson@byd.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 Chapter 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.

Proposal # 4371
S91-19
IBC: 1704.6, 1704.7 (New)

Proponent: 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. Structural observation does not include or waive the responsibility for the inspections in Section 110 or the special inspections in Section 1705 or other sections of this code.

Prior to the commencement of observations, the structural observer shall submit to the building official a written statement identifying the frequency and extent of structural observations.

At the conclusion of the work included in the permit, the structural observer shall submit to the building official a written statement that the site visits have been made and identify any reported deficiencies that, to the best of the structural observer’s knowledge, have not been resolved.

Add new text as follows:

1704.7 Statement of structural observations Where structural observations are required by Section 1706, the structural observer shall prepare a statement of structural observations for submittal to the building official as a condition for permit issuance. The statement of structural observations shall include the following:

1. Contact information for the structural observer
2. Qualification data for the structural observer if the structural observer is not the registered design professional responsible for the structural design
3. The extent of structural observations
4. The frequency of structural observations

Reason: The proposal requires that when applicable, a plan for structural observations is completed at the same time as the plan for special inspections and submitted simultaneously as a comprehensive program and a condition for a permit. It requires that the observer be identified, and that if the observer is not the RDP responsible for the structural design, that qualification data be submitted to ensure that the observer is familiar with the design of similar projects.

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

No tasks are added or eliminated, the overall program of construction quality assurance (tests, inspections, and observations) is just more clearly defined at time of permitting.
S92-19
IBC: 1704.6.1, 1704.6.2, 1704.6.3

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

2018 International Building Code
Revise as follows:

1704.6.1 Structural observations for structures. Structural observations shall be provided for those structures where one or more of the following conditions exist:

1. The structure is classified as Risk Category III or IV.
2. The structure is a high-rise building.
3. The structure is assigned to Seismic Design Category E, and is greater than two stories above the grade plane.
4. Such observation is required by the registered design professional responsible for the structural design.
5. Such observation is specifically required by the building official.

Delete without substitution:

1704.6.2 Structural observations for seismic resistance. Structural observations shall be provided for those structures assigned to Seismic Design Category D, E or F where one or more of the following conditions exist:

1. The structure is classified as Risk Category III or IV.
2. The structure is assigned to Seismic Design Category E, is classified as Risk Category I or II, and is greater than two stories above the grade plane.

1704.6.3 Structural observations for wind resistance. Structural observations shall be provided for those structures sited where V is 130 mph (58 m/sec) or greater and the structure is classified as Risk Category III or IV.

Reason: Construction site observations by a structural engineer to verify that as-built construction generally conforms to the structural design intent are currently not required for facilities such as schools, colleges, stadia, arenas, health care facilities, power stations, structures that store hazardous materials, and water treatment facilities in Seismic Design Categories A, B, or C, or with design wind speeds less than 130 mph. In the conterminous United States, structural observations for these types of structures are only typically required in the green shaded areas of the sketch attached below (not adjusted for site-specific site class).

Risk Category III Occupancy is defined in Table 1604.5 and provides several examples of buildings or other structures that represent a substantial hazard to human life in the event of failure. Given the relative risk and hazard, it is appropriate to require that a structural engineer conduct site visits to verify general conformance to the design intent for these types of structures.

Structural Observations are general, visual overviews of structural systems and load paths. Observations for seismic or wind resisting systems and load paths are performed by the same RDP that observes gravity systems and load paths. Lateral force resisting systems and load paths are integrated with gravity force resisting systems, components, and cladding so it makes no sense to differentiate “wind or seismic resistance observations” from “Structural Observations.” The removal of sections 1704.6.2 and 1704.6.3 is a result of lumping the additional scope for SDC E...
buildings into 1704.6.1, as those portions then become redundant.

**Cost Impact:** The code change proposal will not increase or decrease the cost of construction
There will be no increase in construction cost from this proposal. In areas where it is common practice not to pay a registered design professional to make construction site visits, there may be an increase in the cost of associated construction administration services for Risk Category III buildings that aren't in SDC D, E, or F, or a region where $V_{ult} = 130$ mph. The site visit portion of engineering design fees are generally on the order of 0.1% of construction cost. The assurance greatly outweighs the potential cost.
S93-19

IBC: 110.7 (New), [BS] 202, 1705.1.1

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

2018 International Building Code

Add new text as follows:

110.7 Pre-fabricated components or assemblies. Construction designed to be fabricated off site shall be permitted by the building official, to be inspected, covered and concealed in accordance with Section 1705.1.1, item 4 and other applicable provisions of Chapter 17.

Revise as follows:

[BS] FABRICATED ITEM. Structural, load-bearing or lateral load-resisting members or assemblies consisting of materials assembled prior to installation in a building or structure, or subjected to operations such as heat treatment, thermal cutting, cold working or reforming after manufacture and prior to installation in a building or structure including appurtenant and enclosing materials. Materials produced in accordance with standards referenced by this code, such as rolled structural steel shapes, steel reinforcing bars, masonry units and wood structural panels, or in accordance with a referenced standard that provides requirements for quality control done under the supervision of a third-party quality control agency, are not “fabricated items.”

1705.1 General. Special inspections and tests of elements and nonstructural components of buildings and structures shall meet the applicable requirements of this section.

1705.1.1 Special cases. Special inspections and tests shall be required for proposed work that is, in the opinion of the building official, unusual in its nature, such as, but not limited to, the following examples:

1. Construction materials and systems that are alternatives to materials and systems prescribed by this code.
2. Unusual design applications of materials described in this code.
3. Materials and systems required to be installed in accordance with additional manufacturer’s instructions that prescribe requirements not contained in this code or in standards referenced by this code.
4. Components and assemblies of building elements fabricated off site where materials or finishes cover, conceal or interfere with required inspections including structural, plumbing, mechanical, electrical, and fire resistance inspections.

Reason: An evolution is occurring where building elements are pre-fabricated in factories and assembled on site in order to improve quality and minimize construction time and site labor. Typically in the past, prefabricated construction takes the form of commercial coaches or modular construction. Many states already have provisions and approval processes to address modular construction. In addition to modular construction of buildings, individual building elements such as walls, roofs and floors are being prefabricated in off-site factories and incorporated into the construction of the completed building on site. When prefabricated elements contains no finish or other enclosing materials they can be incorporated into the normal rough framing inspection described in IBC Section 110.3.4. That section reads:

A] 110.3.4 Frame inspection. Framing inspections shall be made after the roof deck or sheathing, all framing, fire blocking and bracing are in place and pipes, chimneys and vents to be concealed are complete and the rough electrical, plumbing, heating wires, pipes and ducts are approved.

This on-site framing inspection provision can be applied to both modular construction and pre-fabricated panelized building elements if they are left open such that electrical, plumbing, heating and framing is visible. The proposed provisions would allow components and elements to be inspected, finished, and covered in the factory through the use of special inspections.

Since Chapter 17 already creates the framework whereby an approved agency is qualified and managed in the permit process, this proposal utilizes that successful framework to allow building elements and components to be constructed and finished in the factory.

Cost Impact: The code change proposal will decrease the cost of construction
Because this proposal provides another inspection pathway as an option to the typical practice of leaving all areas open for inspection in the field this change does not increase the cost of construction.

Proposal # 5531
2018 International Building Code

Revise as follows:

1705.2.4 Cold-formed steel trusses spanning 60 feet or greater. Where a cold-formed steel truss clear span is 60 feet (18 288 mm) or greater, the special inspector shall periodically inspect to verify that the temporary installation restraint/bracing and the permanent individual truss member restraint/bracing are installed in accordance with the approved truss submittal package.

1705.5.1 High-load diaphragms. High-load diaphragms designed in accordance with Section 2306.2 shall be installed with special inspections as indicated in Section 1704.2. The special inspector shall periodically inspect the wood structural panel sheathing to ascertain whether it is of the grade and thickness shown on the approved construction documents. Additionally, the special inspector must periodically verify the nominal size of framing members at adjoining panel edges, the nail or staple diameter and length, the number of fastener lines and that the spacing between fasteners in each line and at edge margins agrees with the approved construction documents.

1705.5.2 Metal-plate-connected wood trusses. Special inspections of wood trusses with overall heights of 60 inches (1524 mm) or greater shall be performed to periodically inspect and verify that the installation of the permanent individual truss member restraint/bracing has been installed in accordance with the approved truss submittal package. For wood trusses with a clear span of 60 feet (18 288 mm) or greater, the special inspector shall periodically inspect to verify during construction that the temporary installation restraint/bracing is installed in accordance with the approved truss submittal package.

Reason: Special Inspections in Chapter 17 are defined on either a periodic or continuous frequency basis. The above three sections define the required special inspections for cold-formed steel, high-load diaphragms, and wood trusses; however, they do not define a frequency of special inspections. This proposal would correct that missing information to provide greater clarity.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. This is just a clarification.
2018 International Building Code

Revise as follows:

**1705.3 Concrete construction.** Special inspections and tests of concrete construction shall be performed in accordance with this section and Table 1705.3 ACI 311.7.

**Exceptions:**

1. Special inspections and tests shall not be required for:
   - 1.1 Isolated spread concrete footings of buildings three stories or less above grade plane that are fully supported on earth or rock.
   - 1.2 Continuous concrete footings supporting walls of buildings three stories or less above grade plane that are fully supported on earth or rock where:
     - 1.2.1 The footings support walls of light-frame construction.
     - 1.2.2 The footings are designed in accordance with Table 1809.7.
     - 1.2.3 The structural design of the footing is based on a specified compressive strength, $f'_c$, not more than 2,500 pounds per square inch (psi) (17.2 MPa), regardless of the compressive strength specified in the approved construction documents or used in the footing construction.
   - 1.3 Nonstructural concrete slabs supported directly on the ground, including prestressed slabs on grade, where the effective prestress in the concrete is less than 150 psi (1.03 MPa).
   - 1.4 Concrete foundation walls constructed in accordance with Table 1807.1.6.2.
   - 1.5 Concrete patios, driveways and sidewalks, on grade.

2. Special inspection for welding reinforcing bars shall be in accordance with section 1705.3.1.

3. Continuous special inspection is required for placement of reinforcing steel for special moment frames, boundary elements of special structural walls, and coupling beams.

**1705.3.1 Welding of reinforcing bars.** Special inspection of welding of reinforcing bars shall be as follows:

1. Special inspections of welding and qualifications of special inspectors for reinforcing bars shall be in accordance with the requirements of AWS D1.4 for special inspection and of AWS D1.4 for special inspector qualification.
2. Perform continuous special inspection for welding of reinforcing steel for special moment frames, boundary elements of special structural walls, and coupling beams.
3. Perform periodic inspection for all other welds.
4. Verify weldability of reinforcing bars other than ASTM A706

**1705.3.2 Material tests.** In the absence of sufficient data or documentation providing evidence of conformance to quality standards for materials in Chapters 19 and 20 of ACI 318, the building official shall require testing of materials in accordance with the appropriate standards and criteria for the material in Chapters 19 and 20 of ACI 318.

Add new text as follows:

**1705.3.3 Post-installed anchors installation.** Specific requirements for special inspection of post-installed anchors shall be included in the research report for the anchor issued by an approved source in accordance with Section 5.1 in ACI 311.7, 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.

Delete without substitution:

**TABLE 1705.3**

**REQUIRED SPECIAL INSPECTIONS AND TESTS OF CONCRETE CONSTRUCTION**

<table>
<thead>
<tr>
<th>TYPE</th>
<th>CONTINUOUS SPECIAL INSPECTION</th>
<th>PERIODIC SPECIAL INSPECTION</th>
<th>REFERENCED STANDARD®</th>
<th>IBCREFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Inspect reinforcement, including prestressing tendons, and verify placement.</td>
<td>—</td>
<td>X</td>
<td>ACI 318: Ch. 20, 25.2, 25.3, 26.6.1, 26.6.3</td>
<td>1908.4</td>
</tr>
</tbody>
</table>
2. Reinforcing bar welding:
   a. Verify weldability of reinforcing bars other than ASTM A706;
   b. Inspect single-pass fillet welds, maximum / in.
   c. Inspect all other welds.

3. Inspect anchors cast in concrete.

4. Inspect anchors post-installed in hardened concrete members:
   a. Adhesive anchors installed in horizontally or upwardly inclined orientations to resist sustained tension loads.
   b. Mechanical anchors and adhesive anchors not defined in 4.a.

5. Verify use of required design mix.

6. Prior to concrete placement, fabricate specimens for strength tests, perform slump and air content tests, and determine the temperature of the concrete.

7. Inspect concrete and shotcrete placement for proper application techniques.

8. Verify maintenance of specified curing temperature and techniques.


10. Inspect erection of precast concrete members.

11. Verify in-situ concrete strength, prior to stressing of tendons in post-tensioned concrete and prior to removal of shores and forms from beams and structural slabs.

12. Inspect formwork for shape, location and dimensions of the concrete member being formed.

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.

Add new standard(s) as follows:

**ACI**

311.7-18: Specification for Inspection of Concrete Construction

**Reason:** The list of criteria in the current Code is not as comprehensive as the list required by ACI 318. ACI 311.7 is written to the inspector and complies with the requirements of ACI 318. This code change simplifies this code, references ACI 311.7 on special inspection and removes conflicts with the requirements of ACI 318.

1) ACI 311.7 is added as a reference with modification to language to align 2018 IBC, ACI 318-19, and ACI 311.7 language.

2) Table 1705.3 is deleted to avoid conflicts between Table 1705.3 and ACI 318.
3) As not to lose pertinent information provided in footnote b to the Table 1705.3, new section 1705.3.3 Anchor installation is added and more appropriately references the applicable section of ACI 311.7.

The criteria in the IBC is not as accurate, complete, and extensive as the criteria in ACI 311.7. ACI 311.7 is aligned with ACI 318 more than Table 1705.3. Further Table 1705.3 does not include all the special inspection requirements of ACI 318. The omissions of criteria in ACI 318 suggest that the additional special inspections required by ACI 318 are not necessary. The result is that the lack of the special inspections as identified in ACI 318 could pose life safety issues. Coordinating and maintaining duplicate lists is always challenging and tends to lead to omissions and errors. The solution, as recommended by this code change proposal, is to comply with the requirements of ACI 311.7. If for some reason it is important for the building code officials to have a partial list of the inspection criteria, such as that in the 2018 edition of the IBC, then this abridged list would be more appropriate as commentary to the IBC.

Differences between IBC Table 1705.3 and ACI 311.7 are:

- Item 1 – exception 2 is added to comply with ACI 318-19 for special moment frame, boundary elements of special structural walls, and coupling beams

- Item 2 – necessary language is retained in Section 1705.3.1 for reinforcing steel and to modify provisions of the IBC and ACI 311.7 to comply with ACI 318-19 for special moment frames, boundary elements of special structural walls and coupling beams.

- Item 3 – no difference

- Item 4 – ACI 311.7 includes a reference to ACI 355.4 Qualification of Post-Installed Adhesive Anchors in Concrete, a standard that prescribes the qualifications for adhesive anchors. This standard was developed by ACI to fill a void that exists due to the absence of an ASTM Standard on adhesive anchor qualifications. Without this reference there are no requirements for qualifying adhesive anchors. ACI 311.7 also requires compliance with both Sections 17.1.2 and 17.8.2 for mechanical anchors whereas the IBC only requires compliance with 17.8.2. ACI 318 Section 17.1.2 prescribes the minimum age of the concrete for anchoring adhesive anchors to concrete. This is crucial criteria necessary to achieve the performance of the adhesive anchors.

- Item 5 - IBC requires compliance with 26.4.3 of ACI 318, but this section does not provide compliance criteria. Chapter 19 of the IBC requires concrete comply with ACI 318 so this specific reference is not required in the table; and IBC requires compliance with 26.4.4 of ACI 318, however ACI 311.7 provides more specific direction to the use. The compliance requirement appropriate for special inspections are specifically included in ACI 318 Section 26.4.4.1, as cited in ACI 311.7.

- Item 6 - Where the IBC only cites compliance with ASTM C31 and C172, ACI 311.6 Specification for Ready Mixed Concrete Testing Services, referenced in ACI 311.7, also provides for compliance with:

  C39 Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens,

  C138 Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete,

  C143 Standard Test Method for Slump of Hydraulic-Cement Concrete,

  C173 Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method,

  C231 Standard Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method,

  C511 Standard Specification for Mixing Rooms, Moist Cabinets, Moist Rooms, and Water Storage Tanks Used in the Testing of Hydraulic Cements and Concretes, and

  C1064 Standard Test Method for Temperature of Freshly Mixed Hydraulic-Cement Concrete

- Item 7 – no difference, ACI 311.7 reference more precise.

- Item 8 – no difference.

- Item 9 – ACI 311.7 more precisely identifies the ACI 318 Section for compliance requirements. ACI 311.7, consistent with ACI 318 also includes compliance with ACI 318 Section 26.13.2:

  (a) Placement of concrete.

  (b) Tensioning of prestressing steel and grouting of bonded tendons.
(c) Installation of adhesive anchors in horizontal or upwardly inclined orientations to resist sustained tension loads in accordance with 17.8.2.4 and where required as a condition of the anchor assessment in accordance with ACI 355.4.

(d) Reinforcement for special moment frames.

- Item 10 – no difference.

- Item 11 – § ACI 311.7 more precisely identifies the ACI 318 Section for compliance requirements. ACI 311.7, consistent with ACI 318 also includes compliance with ACI 318 Section 26.13.3.3(e): “Verification of in-place concrete strength before stressing post-tensioned reinforcement and before removal of shores and formwork from beams and structural slabs.”

- Item 12 – no difference

In addition, ACI 311.7 is written specifically for special inspectors and provides the necessary direction to aid special inspectors determining compliance. ACI 311.7 also includes references to specifications necessary to properly conduct special inspections for specific elements, ACI 355.4 for post-installed anchors and ACI 311.6 for testing of ready-mixed concrete.

This code change avoids confusion for compliance with the intent of both the IBC and ACI 318. It also addresses items omitted from the IBC but required in ACI 318. Proper special inspection should be in accordance with ACI 311.7 and not only the truncated list in the IBC. Without this code change items crucial for life safety could be omitted from special inspection as the IBC criteria supersede the criteria of referenced standards. The omissions in the IBC suggest to the user that the additional criteria of ACI documents are not required.

ACI, a technical professional society, recommends approval of this code change proposal as submitted to avoid confusion and conflicts between the IBC and ACI 318 and to help assure that all items identified as warranting special inspection in ACI 318 are addressed as compliance criteria where special inspection of concrete is required in the IBC.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. There is no increase in the initial cost of construction. Design and construction professionals adhering to the requirements of ACI 318, would be complying with these special inspections requirements as proposed herein and required by Chapter 19 of the IBC. Code change avoids confusion for compliance with both the IBC and ACI documents.

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.
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.

<table>
<thead>
<tr>
<th>TYPE</th>
<th>CONTINUOUS SPECIAL INSPECTION</th>
<th>PERIODIC SPECIAL INSPECTION</th>
<th>REFERENCED STANDARD&lt;sup&gt;a&lt;/sup&gt;</th>
<th>IBC REFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Inspect reinforcement, including prestressing tendons, and verify placement.</td>
<td>—</td>
<td>X</td>
<td>ACI 318: Ch. 20, 25.2, 25.3, 26.6.1-26.6.3</td>
<td>1908.4</td>
</tr>
<tr>
<td>2. Reinforcing bar welding:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. Verify weldability of reinforcing bars other than ASTM A706;</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>b. Inspect single-pass fillet welds, maximum $\frac{1}{16}$&quot; welding of reinforcement for special moment frames, boundary elements of special structural walls, and coupling beams;</td>
<td></td>
<td></td>
<td>AWS D1.4 ACI 318: 26.6.4 13.3</td>
<td></td>
</tr>
<tr>
<td>c. Inspect welded reinforcement splices; and</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>d. Inspect all other welds.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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.
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.
2018 International Building Code

Revise as follows:

**TABLE 1705.3**

<table>
<thead>
<tr>
<th>TYPE</th>
<th>CONTINUOUS SPECIAL INSPECTION</th>
<th>PERIODIC SPECIAL INSPECTION</th>
<th>REFERENCED STANDARD</th>
<th>IBC REFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>10. Inspect erection of precast concrete members.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>11. For precast concrete diaphragm connections or reinforcement at joints classified as moderate or high deformability elements (MDE or HDE) in structures assigned to Seismic Design Category C, D, E, or F, inspect such connections and reinforcement in the field for:</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a. Installation of the embedded parts</td>
<td>X</td>
<td></td>
<td>ACI 318: 26.13.1.3</td>
<td></td>
</tr>
<tr>
<td>b. Completion of the continuity of reinforcement across joints.</td>
<td></td>
<td></td>
<td>ACI 550.5</td>
<td></td>
</tr>
<tr>
<td>c. Completion of connections in the field.</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>13. Verify in-situ concrete strength, prior to stressing of tendons in post-tensioned concrete and prior to removal of shores and forms from beams and structural slabs.</td>
<td></td>
<td>X</td>
<td>ACI 318: 26.11.2</td>
<td></td>
</tr>
<tr>
<td>14. Inspect formwork for shape, location and dimensions of the concrete member being formed.</td>
<td></td>
<td>X</td>
<td>ACI 318: 26.11.1.2(b)</td>
<td></td>
</tr>
</tbody>
</table>

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.

Add new standard(s) as follows:

**ACI**

American Concrete Institute
38800 Country Club Drive
Farmington Hills MI 48331

**550.5-18: Code Requirements for the Design of Precast Concrete Diaphragms for Earthquake Motions**

**Reason:** ACI 318-19 has new provisions for the design of precast concrete diaphragms in Section 18.12.11. Such diaphragms are required to comply with the requirements of ACI 550.5. ACI 550.5 has special inspection requirements for “precast concrete diaphragm connections or reinforcement at joints classified as high deformability elements (HDE), installation of the embedded parts and completion of the continuity of reinforcement across joints, and completion of connections in the field” … in structures assigned to SDC C, D, E and F. ACI 318-19 Section 26.13.1.3 has special inspection requirements for “concrete placement and reinforcement for … precast concrete diaphragms assigned to SDC C, D, E, or F using moderate or high-deformability connections.” The proposed Item 11 added to Table 1705.3 is a conservative synthesis of the two requirements. ACI 318-19 Section 26.13.1.3 also requires that “Installation tolerances of precast concrete diaphragm connections shall be inspected for compliance with ACI 550.5.” The proposed Item 12 added to Table 1705.3 mirrors this requirement.
**Cost Impact:** The code change proposal will increase the cost of construction. The proposed inspection requirements will slightly increase the cost of construction but are needed for safety.

**Staff Analysis:** A review of the standard proposed for inclusion in the code, ACI 550.5-18, 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.
2018 International Building Code

Revised 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.

Proposal # 4393
IBC®: 1705.4.1

Proponent: Phillip Samblanet, The Masonry Society, representing The Masonry Society (psamblanet@masonrysociety.org); Jason Thompson, National Concrete Masonry Association, representing National Concrete Masonry Association (jthompson@ncma.org)

2018 International Building Code
Revise as follows:

1705.4.1 Empirically designed masonry, glass unit masonry and masonry veneer in Risk Category IV. Special inspections and tests for 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 IV shall be performed in accordance with TMS 402, Level B Quality Assurance.

Reason: This is an editorial clean-up item. TMS 402, Section A.1.2.4 (empirical design of masonry) specifically prohibits the use of empirical design in structures assigned to Risk Category IV. IBC Section 2019 addresses empirically designed adobe and imposed the limits of TMS 402, Section A.1.2 on adobe systems. As such, via TMS 402 Section A.1.2.4 adobe systems are prohibited in Risk Category IV structures. Therefore, including empirically designed masonry in Section 1705.4.1 is not needed, because it cannot be used for Risk Category IV.

Bibliography: None

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This is an editorial change to coordinate the requirements between IBC Chapter 17 and IBC Chapter 21.
S100-19
IBC: 1705.5.3 (New), TABLE 1705.5.3 (New)

Proponent: 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**

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

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 prefabricated 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.
S101-19

IBC: 1705.5, 1705.5.1 (New), TABLE 1705.5.1 (New), 1705.5.2 (New), 1705.5.3.1, 1705.5.3.2, 2304.10.1 (New)

Proponent: Stephen Skalko, representing self (svskalko@svskalko-pe.com); Jason Thompson, representing National Concrete Masonry Association (jthompson@ncma.org); Jason Krohn, Precast/Prestressed Concrete Institute, representing Precast/Prestressed Concrete Institute (jkrohn@pci.org); Amy Trygestad, representing Concrete Reinforcing Steel Institute (atrygestad@crsi.org); Jonathan Humble, American Iron and Steel Institute, representing American Iron and Steel Institute (jhumble@steel.org); Scott Campbell, representing National Ready Mixed Concrete Association (scampbell@nrmca.org); Larry Williams, Steel Framing Industry Association, representing Steel Framing Industry Association (lwilliams@steelframingassociation.org)

2018 International Building Code

SECTION 1705
REQUIRED SPECIAL INSPECTIONS AND TESTS

Revise as follows:

1705.5 Wood construction. Special inspections of prefabricated wood structural elements and assemblies shall be in accordance with Section 1704.2.5. Sections 1705.5.1 and 1705.5.2 or 1705.5.3. Special inspections of site-built assemblies shall be in accordance with this section.

Add new text as follows:

1705.5.1 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 1705.5.1

TABLE 1705.5.1
Required Special Inspection of Mass Timber Construction

<table>
<thead>
<tr>
<th>Type</th>
<th>Continuous Special Inspection</th>
<th>Periodic Special Inspection</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Inspection of anchorage and connections of mass timber construction to timber deep foundation systems</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>2. Inspect erection of mass timber construction</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>3. Inspection of connections where installation methods are required to meet design loads</td>
<td></td>
<td></td>
</tr>
<tr>
<td>a) Threaded fasteners</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Verify use of proper installation equipment</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>2. Verify use of pre-drilled holes where required</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>3. Inspect screws, including diameter, length, head type, spacing, installation angle, and depth</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>b) Adhesive anchors, installed in horizontal or upwardly inclined orientation to resist sustained tension loads</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>c) Adhesive anchors not defined in 3.b.</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>d) Bolted connections</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>e) Concealed connections</td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>4. Inspection of connections where installation methods are required to meet the fire resistance design in 2304.10.1</td>
<td>X</td>
<td></td>
</tr>
</tbody>
</table>

1705.5.2 Prefabricated wood construction Special inspections of prefabricated wood structural elements and assemblies shall be in accordance with Section 1704.2.5.

1705.5.3 Sitebuilt wood construction Special inspections of site-built assemblies shall be in accordance with this section.

Revise as follows:

1705.5.4.1 1705.5.5.1 High-load diaphragms. High-load diaphragms designed in accordance with Section 2306.2 shall be installed with special inspections as indicated in Section 1704.2. The special inspector shall inspect the wood structural panel sheathing to ascertain whether its thickness is of the grade and thickness shown on the approved construction documents. Additionally, the special inspector must verify the nominal size of framing members at adjoining panel edges, the nail or staple diameter and length, the number of fastener lines and that the spacing between fasteners in...
each line and at edge margins agrees with the approved construction documents.

**1705.5.2 1705.5.3.2 Metal-plate-connected wood trusses.** Special inspections of wood trusses with overall heights of 60 inches (1524 mm) or greater shall be performed to verify that the installation of the permanent individual truss member restraint/bracing has been installed in accordance with the approved truss submittal package. For wood trusses with a clear span of 60 feet (18 288 mm) or greater, the special inspector shall verify during construction that the temporary installation restraint/bracing is installed in accordance with the approved truss submittal package.

### SECTION 2304
GENERAL CONSTRUCTION REQUIREMENTS

**2304.10 Connectors and fasteners.** Connectors and fasteners shall comply with the applicable provisions of Sections 2304.10.1 through 2304.10.7.

Add new text as follows:

**2304.10.1 Connection fire resistance.** Fire resistance ratings for connections in Type IV-A, IV-B, or IV-C construction shall be determined by one of the following:

1. Testing in accordance with Section 703.2 where the connection is part of the fire resistance test.
2. Engineering analysis that demonstrates that the temperature rise at any portion of the connection is limited to an average temperature rise of 250°F (139°C), and a maximum temperature rise of 325°F (181°C), for a time corresponding to the required fire resistance rating of the structural element being connected. For the purposes of this analysis, the connection includes connectors, fasteners, and portions of wood members included in the structural design of the connection.

**Reason:** This proposal adds special inspection provisions to Section 1705 for mass timber. Mass timber is a new construction type, and contractors and inspectors have no experience working with this system. Due to the importance of connections in the performance of mass timber systems, and the inexperience of all involved parties, a level of inspection beyond that required of other construction methods is required, until such a time as both contractors and inspectors have gained the necessary practical knowledge of the systems and their construction. Thus, the special inspections proposed are greater than what is required for similar systems such as pre-cast concrete and structural steel in certain critical areas. This is consistent with the intentions of Section 1705.5.1 where special inspections are intended for unusual design applications of materials included in the code, or where adherence to manufacturer's instructions not specified in the code is required.

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. Tall wood buildings utilizing pre-fabricated elements need 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, adhesive, or concealed connections are considered continuous. The strength of many connection designs is predicated on pre-drilling, specific screw lengths and installation angles. Most of these cannot be verified by the jurisdictional inspector, or after installation, so continuous special inspections are required. Similarly, use of the correct materials and installation procedures cannot be verified for concealed connections, and proper installation is required for structural performance, requiring continuous special inspection. Adhesive anchorage installed in horizontal or upwardly inclined positions resisting tension loads shall be continuously inspected, 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.

- Bolted connections require specific diameters, and for lag bolts, specific lengths. Correct bolt usage and installation are required for structural performance, and hence periodic special inspection is required.

(4) Inspection of fire-resistant design connections. During fire testing connections have proven to be critical to the behavior of mass timber assemblies. Since the fire rated connection designs in mass timber often require specific applications of cover material, embedded depth of connectors, etc., to perform adequately, continuous special inspection is indicated to preserve the integrity of the system. This is especially important where engineered design analysis may be the method used for determining the fire resistance of the mass timber connections.

The CLT Handbook available for designers raises some of these concerns [CLT Cross-Laminated Timber Handbook US Edition, 2013]. In Chapter 8, Fire, Section 5 Connections the handbook states:

*Due to the high thermal conductivity of steel, metallic fasteners and plates directly exposed to fire may heat up and conduct heat into the wood members. The wood components may then experience charring on the exposed surface and around the fastener. As a result, the capacity of the metallic connection is reduced to the strength reduction of the steel fasteners at elevated temperatures and...*
the charring of the wood members. Therefore, where a fire resistance rating is required by the IBC, connections and fasteners are required to be protected from fire exposure by wood, gypsum board or other protection approved for the required rating.

While the protection cited may increase the fire endurance of the metallic portions of the connections, the connection elements will still be subjected to elevated temperatures during a fire event.

A technical research report on connections for tall wood buildings prepared for the National Research Council of Canada reported that the fire resistance for concealed connections may be on the order of 1 to 1-1/2 hours [Canadian Commission on Building and Fire Codes, Standing Committee on Fire Protection, Review of Fire Resistant Design of Connections, January 2017, page 8]. The report conclusion suggests that some extra overlay of wood may be necessary for the 2-hour and 3-hour fire resistance of mass timber provisions proposed by the ICC TWB Committee. This is not to suggest that 2-hour fire or 3-hour resistances of connections cannot be achieved, but that connections must be given extra attention and standard methods for the industry may not be sufficient. This extra attention is what is intended through the requirement for continuous special inspection begin proposed.

Besides adding the special inspection table this proposed change also modifies Section 1705.5 to clarify that prefabricated wood structural members and sitebuilt wood members assembled in the field are not exempt from the requirements for special inspection. The original wording in the code to refer the user to Section 1704.2.5 for prefabricated wood assemblies and to Sections 1705.5.1 & 1705.5.2 for sitebuilt wood assemblies is still maintained but these sections have been renumbered based on new 1705.5.1.

Finally, this proposal 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 consistent with the language proposed by the Tall Wood Ad-Hoc Committee. It is included in this proposal because it is referenced in Table 1705.5.1 and should not be considered as a separate code proposal.

**Cost Impact:** The code change proposal will increase the cost of construction
The 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 insure the code is met.
2018 International Building Code

Revise as follows:

1705.6 Soils. Special inspections and tests of existing site soil conditions, fill placement and load-bearing requirements shall be performed in accordance with this section and Table 1705.6. The approved geotechnical report and the construction documents prepared by the registered design professionals shall be used to determine compliance. During fill placement, the special inspector shall verify that proper materials and procedures are used in accordance with the provisions of the approved geotechnical report.

Exception: Where Section 1803 does not require reporting of materials and procedures for fill placement, the special inspector shall verify that the in-place dry density of the compacted fill is not less than 90 percent of the maximum dry density at optimum moisture content determined in accordance with ASTM D1557.

### TABLE 1705.6
REQUIRED SPECIAL INSPECTIONS AND TESTS OF SOILS

<table>
<thead>
<tr>
<th>TYPE</th>
<th>CONTINUOUS SPECIAL INSPECTION</th>
<th>PERIODIC SPECIAL INSPECTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Verify materials below shallow foundations are adequate to achieve the design bearing capacity.</td>
<td>—</td>
</tr>
<tr>
<td>2.</td>
<td>Verify excavations are extended to proper depth and have reached proper material.</td>
<td>—</td>
</tr>
<tr>
<td>3.</td>
<td>Perform classification and testing of compacted fill materials.</td>
<td>—</td>
</tr>
<tr>
<td>4.</td>
<td>During fill placement, verify use of proper materials and procedures in accordance with the provisions of the approved geotechnical report. Verify densities and lift thicknesses during placement and compaction of compacted fill.</td>
<td>X</td>
</tr>
<tr>
<td>5.</td>
<td>Prior to placement of compacted fill, inspect subgrade and verify that site has been prepared properly.</td>
<td>—</td>
</tr>
</tbody>
</table>

### TABLE 1705.7
REQUIRED SPECIAL INSPECTIONS AND TESTS OF DRIVEN DEEP FOUNDATION ELEMENTS

<table>
<thead>
<tr>
<th>TYPE</th>
<th>CONTINUOUS SPECIAL INSPECTION</th>
<th>PERIODIC SPECIAL INSPECTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Verify element materials, sizes and lengths comply with the requirements.</td>
<td>X</td>
</tr>
<tr>
<td>2.</td>
<td>Determine capacities of test elements and conduct additional load tests, as required.</td>
<td>X</td>
</tr>
<tr>
<td>3.</td>
<td>Inspect driving operations and maintain complete and accurate records for each element.</td>
<td>X</td>
</tr>
<tr>
<td>4.</td>
<td>Verify placement locations and plumbness, confirm type and size of hammer, record number of blows per foot of penetration, determine required penetrations to achieve design capacity, record tip and butt elevations and document any damage to foundation element.</td>
<td>X</td>
</tr>
<tr>
<td>5.</td>
<td>For steel elements, perform additional special inspections in accordance with Section 1705.2.</td>
<td>In accordance with 1705.2</td>
</tr>
<tr>
<td>6.</td>
<td>For concrete elements and concrete-filled elements, perform tests and additional special inspections in accordance with Section 1705.3.</td>
<td>In accordance with 1705.3</td>
</tr>
<tr>
<td>7.</td>
<td>For specialty elements, perform additional inspections as defined in the statement of special inspections.</td>
<td>In accordance with Statement of Special Inspections</td>
</tr>
</tbody>
</table>

Revise as follows:

### TABLE 1705.8
REQUIRED SPECIAL INSPECTIONS AND TESTS OF CAST-IN-PLACE DEEP FOUNDATION ELEMENTS
<table>
<thead>
<tr>
<th>TYPE</th>
<th>CONTINUOUS SPECIAL INSPECTION</th>
<th>PERIODIC SPECIAL INSPECTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Inspect drilling operations and maintain complete and accurate records for each element.</td>
<td>X</td>
<td>—</td>
</tr>
<tr>
<td>2. Verify placement locations and plumbness, confirm element diameters, bell diameters (if applicable), lengths, embedment into bedrock (if applicable) and adequate end-bearing strata capacity. Record concrete or grout volumes.</td>
<td>X</td>
<td>—</td>
</tr>
<tr>
<td>3. For concrete elements, perform tests and additional special inspections in accordance with Section 1705.3.</td>
<td>In accordance with 1705.3</td>
<td></td>
</tr>
</tbody>
</table>

**Reason:** The last sentence of Section 1705.6 overlaps with item 4 of Table 1705.6. For clarity, relocating this provision to Table 1705.6 is proposed so that all required inspections and tests can be contained in one table location.

Items 5 to 7 of Table 1705.7 contain only hyphens under the headings of Continuous or Periodic Special Inspection, which may mislead users to feel that no special inspections are required for these items. By making the proposed change herein, it becomes clear that for these three items the user must refer to other code Sections or project team members to define the required special inspections. Similar proposal for Item 3 of Table 1705.8.

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

This proposal is for clarification.

Proposal # 4210

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S102-19
S103-19

IBC: 1705.10 (New), ASTM Chapter 35 (New)

Proponent: Dale Biggers, P.E. GeoCoalition, representing GeoCoalition (dbiggers@bohbros.com); Daniel Stevenson, P.E., representing GeoCoalition (dstevenson@berkelapg.com); Lori Simpson, Langan, representing GeoCoalition (lsimpson@langan.com)

2018 International Building Code

Add new text as follows:

1705.10 Structural Integrity of Deep Foundation Elements. When directed by the registered design professional in responsible charge or by the building official, an engineering assessment for structural integrity shall be conducted a deep foundation element. The engineering assessment shall include tests for defects performed in accordance with ASTM D4945, ASTM D5882, ASTM D6760, or ASTM D7949 or other approved method.


D6760-16: Standard Test Method for Integrity Testing of Concrete Deep Foundations by Ultrasonic Crosshole Testing

D7949-14: Standard Test Methods for Thermal Integrity Profiling of Concrete Deep Foundations

Reason: Significant defects affect the structural strength of the deep foundation elements, and therefore need to be detected and corrected prior to further construction to prevent foundation failures.

- When the integrity of a deep foundation element is in doubt (e.g. due to the installation records, due to difficult soil conditions, or to approve the construction procedures) the deep foundation element should be tested during installation to assure no material defects.

- Sections 1705.7, 1705.8 and 1705.9 already address visual inspections. The tests in this new proposed section provide a means to assess portions of deep foundation elements that cannot be visually inspected.

- Most foundation failures are caused by inadequate geotechnical capacity. This proposed section addresses the other possible failure mode – namely lack of structural integrity.

- Use of “other approved method” allows for possible methods not yet known or standardized which the building official has confidence in.


ASTM D6760 is “Standard Test Method for Integrity Testing of Concrete Deep Foundations by Ultrasonic Crosshole Testing”

ASTM D7949 is “Standard Test Methods for Thermal Integrity Profiling of Concrete Deep Foundations”

(all methods above are non-destructive tests used to assess the integrity of deep foundations)

This proposal is presented for your consideration by the GeoCoalition.

The GeoCoalition is a consortium of of eight trade and professional associations and our active group includes 37 geotechnical engineers, structural engineers, and specialty contractors from across the country.

To access the GeoCoalition roster,
Cost Impact: The code change proposal will not increase or decrease the cost of construction.

The proposal may increase or decrease the cost of construction, depending on whether the tests disprove or prove the ability of the questionable foundation element to withstand the required loads. Assuring the foundation element has no structural defects will avoid subsequent substantial remediation costs for foundations that would have failed due to undetected defects, and thus produce overall savings to the project.

Staff Analysis: A review of the standard proposed for inclusion in the code, ASTM D5882-16, D6760-16 and D7949-14, 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.
**2018 International Building Code**

Revise as follows:

**1705.11 Special inspections for wind resistance.** Special inspections for wind resistance specified in Sections 1705.11.1 through 1705.11.3, unless exempted by the exceptions to Section 1704.2, are required for buildings and structures constructed in the following areas:

1. In wind Exposure Category B, where $V$ as determined in accordance with Section 1609.3.1 is 120 is 150 miles per hour (52.8 - 67 m/sec) or greater.
2. In wind Exposure Category C or D, where $V$ as determined in accordance with Section 1609.3.1 is 110 is 140 mph (49.6 - 62.6 m/sec) or greater.

**Reason:** This is an editorial change to reference the wind speed triggers to the Chapter 16 mapped basic design wind speed, $V$ for consistency with other section of this chapter. Currently the user would be required to convert the mapped basic design wind speed to Vasd. Thus the change in wind speed indicated is a conversion from the previous Vasd values to $V$.

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

This is an editorial change to reference the wind speed triggers to the Chapter 16 mapped basic design wind speed, $V$ for consistency with other section of this chapter. Currently the user would be required to convert the mapped basic design wind speed to Vasd. Thus the change in wind speed indicated is a conversion from the previous Vasd values to $V$. 

Proposal # 4756
2018 International Building Code

Revise as follows:

1705.11.1 Structural wood. **Continuous special inspection** is required during field gluing operations of elements of the main windforce-resisting system. **Periodic special inspection** is required for nailing, bolting, anchoring and other fastening of elements of the main windforce-resisting system, including wood shear walls, wood diaphragms, drag struts, braces and hold-downs.

**Exception:** **Special inspections** are not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other elements of the main windforce-resisting system, where the lateral resistance is provided by structural sheathing and the specified fastener spacing at panel edges is more than 4 inches (102 mm) on center.

1705.11.2 Cold-formed steel light-frame construction. **Periodic special inspection** is required for welding operations of elements of the main windforce-resisting system. **Periodic special inspection** is required for screw attachment, bolting, anchoring and other fastening of elements of the main windforce-resisting system, including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

**Exception:** **Special inspections** are not required for cold-formed steel light-frame shear walls and diaphragms, including screwing, bolting, anchoring and other fastening to components of the windforce resisting system, where either of the following applies:

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

1705.12.2 Structural wood. For the seismic force-resisting systems of structures assigned to Seismic Design Category C, D, E or F:

1. **Continuous special inspection** shall be required during field gluing operations of elements of the seismic force-resisting system.
2. **Periodic special inspection** shall be required for nailing, bolting, anchoring and other fastening of elements of the seismic force-resisting system, including wood shear walls, wood diaphragms, drag struts, braces, shear panels and hold-downs.

**Exception:** **Special inspections** are not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other elements of the seismic force-resisting system, where the lateral resistance is provided by structural sheathing and the fastener spacing of the sheathing at panel edges is more than 4 inches (102 mm) on center.

1705.12.3 Cold-formed steel light-frame construction. For the seismic force-resisting systems of structures assigned to Seismic Design Category C, D, E or F, periodic special inspection shall be required for both:

1. Welding operations of elements of the seismic force-resisting system.
2. Screw attachment, bolting, anchoring and other fastening of elements of the seismic force-resisting system, including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

**Exception:** **Special inspections** are not required for cold-formed steel light-frame shear walls and diaphragms, including screw installation, bolting, anchoring and other fastening to components of the seismic force-resisting system, where either of the following applies:

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

**Reason:** The primary purpose of this proposal is to clarify the intent of the exceptions from special inspection of wood diaphragms and shear walls in high-seismic and high wind areas. The original exception was intended to apply to buildings of light-frame construction where wood studs or joists are sheathed with a variety of structural sheathing materials (e.g. oriented-strand board, plywood, or gypsum board) to form the diaphragm, and where the capacity of shear walls, panels, and diaphragms for resisting wind and seismic loads is defined in the American Wood Council’s Special Design Provisions for Wind and Seismic (AWC SDPWS). The exceptions should apply to shear walls, shear panels and diaphragms constructed with traditional 2x Dimensional lumber or equivalent products (e.g. I-Joists or LVL’s) and structural sheathing, or nail-laminated or dowel laminated diaphragms with sheathing, but not to lateral force-resisting systems relying solely on mass timber products for lateral resistance.

In evaluating special inspection requirements for mass timber buildings, the ICC Ad-Hoc Committee on Tall Wood Buildings did not feel the exception should apply unless a mass timber building relied on a separate layer of wood structural panel sheathing or other sheathing to provide lateral load resistance. However, since this issue is not specific to tall mass timber buildings, the TWB determined that proposing changes to the exception was out of its scope, and referred the issue to the BCAC for review and modification as needed.
Similar exceptions to those for special inspection of wood diaphragms and wood shear walls on wood buildings are provided for wood structural panel or steel sheet diaphragms on cold-formed steel buildings. The same clarifications that the fastener spacing is the specified fastener spacing based on the structural engineer’s design and tabulated diaphragm and shear wall capacities in the material design standards and that the fastening in question is that at panel edges (or sheet edges for diaphragms and shear walls sheathed with steel sheet) are made for the corresponding wind and seismic special inspections for cold-formed steel buildings.

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 the BCAC website at: https://www.iccsafe.org/codes-tech-support/codes/code-development-process/building-code-action-committee-bcac/.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
The code change does not change the application of the two exceptions to diaphragms and shear walls using sheathing materials currently permitted by the code via the reference to the AWC SDPWS. Thus, there is no cost increase for light-frame buildings that currently qualify for the exception. Mass timber buildings not already permitted under existing limits on Type IV construction must go through an alternate means and methods process to gain approval. The work of the AHC-TWB to gain code recognition for tall wood buildings will reduce the cost of construction for such buildings as they will not require special approval procedures. The corresponding clarifications for cold-formed steel buildings do not change the intended application of those exceptions.

Proposal # 4126
2018 International Building Code

Revise as follows:

1705.11.1 Structural wood. Continuous special inspection is required during field gluing operations of elements of the main windforce-resisting system. Periodic special inspection is required for nailing, bolting, anchoring and other fastening of elements of the main windforce-resisting system, including wood shear walls, wood diaphragms, drag struts, braces and hold-downs.

Exception: Special inspections are not required for light-frame wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other elements of the main windforce-resisting system, where the specified fastener spacing at panel edges is more than 4 inches (102 mm) on center.

1705.12.2 Structural wood. For the seismic force-resisting systems of structures assigned to Seismic Design Category C, D, E or F:

1. Continuous special inspection shall be required during field gluing operations of elements of the seismic force-resisting system.
2. Periodic special inspection shall be required for nailing, bolting, anchoring and other fastening of elements of the seismic force-resisting system, including wood shear walls, wood diaphragms, drag struts, braces, shear panels and hold-downs.

Exception: Special inspections are not required for light-frame wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other elements of the seismic force-resisting system, where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center.

Reason: The term "light-frame" is added to the exceptions in Sections 1705.11.1 and 1705.12.2 to make clear the special inspection exemption only applies to light wood frame assemblies and not assemblies of mass timber such as CLT panels that may be serving as shear walls, shear panels or diaphragms.

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

The code change is a clarification of the code and should not have an impact on construction costs.
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

<table>
<thead>
<tr>
<th>Type</th>
<th>Continuous Inspection</th>
<th>Periodic Inspection</th>
<th>Referenced Standard</th>
<th>IBC Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Verify materials used comply with one or more of the material test reports in accordance with the approved construction documents</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fabricated storage rack elements</td>
<td>X</td>
<td></td>
<td>MH16.1</td>
<td>1704.2.5</td>
</tr>
<tr>
<td>Installation of storage rack anchorage</td>
<td>X</td>
<td></td>
<td>Section 7.3.2</td>
<td></td>
</tr>
<tr>
<td>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</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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.


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.

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.
**S108-19**

**IBC®: 1709.5, AAMA Chapter 35 (New)**

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

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**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 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 labeled assembly. Where engineering analysis is used, they 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

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**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.
S109-19

IBC: 1709.5.2, 1709.5.2.1 (New)

Proponent: T. Eric Stafford, representing Insurance Institute for Business and Home Safety (testafford@charter.net)

2018 International Building Code

Revise as follows:

1709.5.2 Exterior windows and door assemblies not provided for in Section 1709.5.1. Exterior window and door assemblies shall be tested in accordance with ASTM E330. Structural performance of garage doors and rolling doors shall be determined in accordance with either ASTM E330 or ANSI/DASMA 108, and shall meet the acceptance criteria of ANSI/DASMA 108. Exterior window and door assemblies containing glass shall comply with Section 2403. The design pressure for testing shall be calculated in accordance with Chapter 16. Each assembly shall be tested for 10 seconds at a load equal to 1.5 times the design pressure.

Add new text as follows:

1709.5.2.1 Garage doors and rolling doors. Garage doors and rolling doors shall be tested in accordance with either ASTM E 330 or ANSI/DASMA 108, and shall meet the acceptance criteria of ANSI/DASMA 108. Garage doors and rolling doors shall have a permanent label identifying the door manufacturer, the door model/series number, the positive and negative design wind pressure rating, the installation instruction drawing reference number, and the applicable test standard.

Reason: This proposal is one of several that are addressing labeling of critical components of the building envelope. The primary purpose of this code change is to require that garage doors have a permanent label that provides a way for building owners, homeowners, and others to be able to determine their performance characteristics after the building has been occupied. The 2018 IBC does not require any type of label for garage doors. For products that don’t have permanent labels, it becomes nearly impossible for the owner to determine the structural wind load resistance and/or energy efficiency of the garage doors after they’ve occupied the building. This proposal would simply require some type of permanent marking on the garage door indicating the manufacturer and model/series number, and basic performance characteristics so that the specific performance characteristics could be retrieved at a later date.

This same proposal was submitted for the 2018 IBC but was not approved by the IBC Structural Committee. However, it was nearly unanimously approved at the final action hearings, but did not get the required majority during the OGVC.

For the past 10-15 years, there has been a push towards considering sustainability in the way our buildings are constructed in this country. As a result, building owners and occupants increasingly want more information about the sustainability of the buildings they occupy. Consequently, they need to be provided with ways to determine how critical components are expected to perform in the buildings they use. Garage doors are important components of the building envelope and their performance is critical in preventing wind and water infiltration as well as to maintaining the overall structural integrity of the building.

Some manufacturers already include permanent labels on their products that provide traceability to the manufacture and the product characteristics. The Florida Building Code has required this type of label since the 2007 edition and has continued to require it in subsequent editions. The following is the relevant text from the 6th Edition (2017) Florida Building Code, Building:

1710.5.2.1.1 Garage door labeling. Garage doors shall be labeled with a permanent label provided by the garage door manufacturer. The label shall identify the garage door manufacturer, the garage door model/series number, the positive and negative design pressure rating; indicate impact rated if applicable; the installation instruction drawing reference number; the Florida product approval or Miami-Dade product approval number if applicable; and the applicable test standards. The required garage door components for an approved garage door assembly may be indicated using a checklist form on the label. If a checklist format is used on the label, the door installer or the garage door manufacturer shall mark the selected components on the checklist that are required to assemble an approved garage door system. The installation instructions shall be provided and available on the job site.

Also, Oklahoma Uniform Building Code Commission Rules in their Appendix Y require that garage doors be wind rated to135 mph. Having a permanent label will facilitate verification that the right type of garage door is installed.

Approval of this proposal assure going forward that new or replaced doors will be labeled such that provide building owners and those considering the purchase of buildings with these products will be able to obtain information necessary for determining the expected performance of these critical components of the building envelope.

Cost Impact: The code change proposal will increase the cost of construction

Will impact cost for some manufacturers. The code does not currently require a permanent label. However, some garage door manufacturers voluntarily apply a permanent label that identifies the critical performance characteristics. There will be no cost impact to those manufacturers.

Proposal # 4363

Proposal # 4363
S110-19

IBC: 1709.5.3 (New), 1709.5.3.1 (New), 202 (New)

Proponent: T. Eric Stafford, representing Insurance Institute for Business and Home Safety (testafford@charter.net)

2018 International Building Code

Add new text as follows:

1709.5.3 Wind-borne debris protection Protection of exterior glazed openings in buildings located in windborne debris regions shall be in accordance with Section 1609.1.2.

1709.5.3.1 Impact protective systems testing and labeling Impact protective systems shall be tested for impact resistance by an approved independent laboratory for compliance with ASTM E 1886 and ASTM E 1996. Impact protective systems shall also be tested for design wind pressure by an approved independent laboratory for compliance with ASTM E 330. Required design wind pressures shall be determined in accordance with Section 1609.6 or ASCE 7, and for the purposes of this section, multiplied by 0.6 to convert to allowable stress design. Impact protective systems shall have a label identifying the manufacturer, performance characteristics, and approved inspection agency. Impact protective systems shall have a permanent label applied in accordance with Section 1703.5.4 that provides traceability to the manufacturer, product designation, and performance characteristics.

Add new definition as follows:

IMPACT PROTECTIVE SYSTEM. Construction that has been shown by testing to withstand the impact of test missiles and that is applied, attached, or locked over exterior glazing.

Reason: This proposal is one of several that are addressing labeling of critical components of the building envelope. The primary purpose of this code change is to require that impact protective systems (hurricane shutters) have a permanent label that provides a way for building owners, homeowners, and others to be able to determine their performance characteristics after the building has been occupied. The 2018 IBC does not require any type of label for impact protective systems. However, the 2018 IRC requires impact protective systems to be labeled with similar language as submitted with this proposal. For products that don’t have permanent labels, it becomes nearly impossible for the owner to determine the structural wind load resistance and impact resistance of the products after they’ve occupied the building. This proposal would simply require some type of permanent marking on the impact protective system indicating the manufacturer and model/series number, that provides traceability so specific performance characteristics can be retrieved at a later date. While the permanent label would only need to provide traceability to the product, it could provide all the required information. If the relevant information is not provided on a permanent label, a temporary removable label is required to be applied so that local code officials can verify that the appropriate impact protective system was provided.

For the past 10-15 years, there has been a push towards considering sustainability in the way our buildings are constructed in this country. If this goal is to be successful and building owners and occupants increasingly want more information about the sustainability of the buildings they occupy, they need to be provided ways to be able to determine how critical components are expected to perform in the buildings they use. Impact protective systems are important components of the building envelope and their performance is critical to maintaining the overall structural integrity of the building.

Some manufacturers already include permanent labels on their products that provide traceability to the manufacture and the product characteristics. The Florida Building Code has required a permanent label since the 2007 edition and has continued to require it in subsequent editions. The following is the relevant text from the 5th Edition (2014) Florida Building Code, Building:

1710.8 Impact resistant coverings.

1710.8.1 Labels. A permanent label shall be provided by the product approval holder on all impact-resistant coverings.

1710.8.2 The following information shall be included on the labels on impact-resistant coverings:

1. Product approval holder name and address.

2. All applicable methods of approval. Methods of approval include, but are not limited to Miami-Dade NOA; Florida Building Commission, TDI Product Evaluation; ICC-ES.

3. The test standard or standards specified in Section 1609.1.2, including standards referenced within the test standards specified in Section 1609.1.2 used to demonstrate code compliance.

4. For products with a Florida product approval number or a Miami-Dade County Building and Neighborhood Compliance Department Notice of Acceptance Number (NOA), such numbers shall be included on the label.
This proposal also provides some additional clarification for impact protective systems that is lacking in the IBC. New Section 1709.5.3.1 clarifies that impact protective systems also have to be capable of resisting the required design wind pressure as well as the impact criteria. New language is added to clarify the relationship between design wind loads calculated in accordance with ASCE 7-10 and the wind load testing requirements of ASTM E 330.

Impact protective systems are important components of the building envelope and their performance is critical to maintaining the overall structural integrity of the building. Approval of this proposal will assure, going forward, that new or replaced impact protective systems will be labeled such that building owners and those considering the purchase of buildings with these products will be able to obtain information necessary for determining the expected performance of these critical components used to protect the building envelope in hurricane prone areas.

**Cost Impact:** The code change proposal will increase the cost of construction
A consultant representing the industry estimates the cost of providing labels on impact resistant covering products to be as follows:

a. Water Resistant Self-adhering Permanent Labels approximately $0.15 per label. Such labels would most likely be used on Accordion, Roll, Bahama, and Colonial style shutters.

b. Embossed or ink jet labels used on metal and plastic panels would cost approximately $0.05 per label.
S111-19

IBC®: 1803.5.7

Proponent: 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:

1803.5.7 Excavation near foundations. Where excavation will reduce support from any foundation, a registered design professional shall prepare an assessment of the structure as determined from examination of the structure, the review of available design documents, available subsurface data, and, if necessary, excavation of test pits. The registered design professional shall determine the requirements for underpinning, support and protection of any existing foundation and prepare site-specific plans, details and sequence of work for submission. Such support shall be provided by underpinning, sheeting and bracing, excavation retention systems, or by other means acceptable to the building official.

Reason:

- "Available subsurface data" may include geotechnical investigations of either the adjacent structure or the building under construction.
- The term “support” is broader in scope and includes “underpinning”.
- Requirements for support are described in Section 3307.
- The term “excavation retention systems” encompasses the many methods which are available today, while “sheeting” is an older term which precludes slurry walls, etc.

Click to see the members of the GeoCoalition: http://www.piledrivers.org/2019-geocoalition-members/

Members of the GeoCoalition

The 37 members are structural and geotechnical engineers and contractors from across the country. They are in leadership positions of more than nine organizations including:

- DFI – Deep Foundations Institute
- PDCA – Pile Driving Contractors Association
- ADSC – Association of Drilled Shaft Contractors
- ASCE – American Society of Civil Engineers
- ASTM – American Society of Testing Materials
- ACI – American Concrete Institute
- SAME – Society of American Military Engineers
- NCSEA – National Council of Structural Engineers Associations
- GBA – Geoprofessional Business Association (formerly ASFE)

Leadership positions held include:

- Chair DFI Soil Nailing and Tiebacks Committee
- Chair PDCA Technical Committee
- President of Geo-Institute
- Chair ADSC-DFI Joint Micropile Committee
- President of PDCA
- Chair Earth Retaining Structures of ASCE/G-I
- Manager DFI Technical Committees
- Received five ASTM Standards Development Awards
- Chair DFI Helical Pile Committee
- Chair DFI Codes and Standards Committee
- Director of GBA
- President of DFI
- PDCA Lifetime Achievement Award
- Chair DFI Testing and Evaluation Committee
- ASCE Outstanding Civil Engineer New Orleans
- DFI Distinguished Service Award

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

Proper earth retention systems for protecting existing adjacent structures will eliminate potentially large remediation costs.
2018 International Building Code

Revise as follows:

1804.1 Excavation near foundations. Excavation for any purpose shall not reduce vertical or lateral support for any foundation or adjacent foundation without first underpinning or protecting the foundation against detrimental lateral or vertical movement, or both, in accordance with Section 1803.5.7.

Reason: To include reference to preceding Section which contains the supporting information relative to “Excavation near foundations”. Click here to see the members of the GeoCoalition: http://www.piledrivers.org/2019-geocoalition-members/

Cost Impact: The code change proposal will not increase or decrease the cost of construction. Proper earth retention systems for protecting existing adjacent structures will eliminate potentially large remediation costs.
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 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.
2018 International Building Code

Revised 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 specified 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 but rather provides clarification to indicate service wind load to be used in lieu of nominal load (ultimate wind load).
2018 International Building Code

Add new text as follows:

1807.2.4 Segmental Retaining Walls  Dry-cast concrete units used in the construction of segmental retaining walls shall comply with ASTM C1372.

ASTM

C1372-17: Standard Specification for Dry-Cast Segmental Retaining Wall Units

Reason: ASTM C1372, Standard Specification for Dry-Cast Segmental Retaining Wall Units, was first published in 1997 to establish the minimum physical properties and acceptable constituent materials for segmental retaining wall (SRW) units. Key attributes covered include minimum compressive strength, maximum dimensional tolerances, and freeze-thaw durability performance. As the use of SRW systems continues to grow, material-related failures are unfortunately becoming more commonplace. While ASTM C1372 has been in circulation for more than 20 years, compliance with this standard remains a voluntary option for designers and project specifiers. Introducing a mandatory reference to ASTM C1372 for SRWs into the IBC is intended to address failures due to material deficiencies.

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

Compliance with ASTM C1372 has been an established industry recommendation for SRW units for decades.

Staff Analysis: A review of the standard proposed for inclusion in the code, ASTM C1372-17, with regard to the ICC criteria for referenced standards (Section 3.6 of CP#128) will be posted on the ICC website on or before April 2, 2019.
S116-19

IBC: 1807.2.4 (New), ASTM Chapter 35 (New), NCMA Chapter 35 (New)

Proponent: Jason Thompson, National Concrete Masonry Association, representing National Concrete Masonry Association (jthompson@ncma.org)

2018 International Building Code
Add new text as follows:

1807.2.4 Segmental Retaining Walls
Dry-cast concrete units used in the construction of segmental retaining walls shall comply with ASTM C1372. The design and construction of segmental retaining walls shall comply with NCMA TR127B.

ASTM

C1372-17: Standard Specification for Dry-Cast Segmental Retaining Wall Units

NCMA

TR127B-10: Design Manual for Segmental Retaining Walls

Reason: This code change, if accepted, accomplished two goals:
A) It introduces mandatory compliance for segmental retaining wall units to ASTM C1372, Standard Specification for Dry-Cast Segmental Retaining Wall Units. ASTM C1372 was first published in 1997 to establish the minimum physical properties and acceptable constituent materials for segmental retaining wall (SRW) units. Key attributes covered include minimum compressive strength, maximum dimensional tolerances, and freeze-thaw durability performance.

As the use of SRW systems continues to grow, material-related failures are unfortunately becoming more commonplace. While ASTM C1372 has been in circulation for more than 20 years, compliance with this standard remains a voluntary option for designers and project specifiers. Introducing a mandatory reference to ASTM C1372 for SRWs into the IBC is intended to address failures due to material deficiencies.

B) It introduces a reference to NCMA TR127B, Design Manual for Segmental Retaining Walls. It’s critical to note that NCMA TR127B does not meet the criteria of CP28 as this document is not written in mandatory language. Further, while NCMA TR127B was developed through an industry consensus process, this process does not comply with all aspects of the consensus process established by ANSI. It is, however, the closest to a consensus standard for SRW systems that exists.

NCMA TR127B was first published in 1993 and has undergone multiple updates and revisions as new research, analyses, and information becomes available. NCMA TR127B can be freely accessed at the following link:

http://ncma-br.org/pdfs/masterlibrary/TR-127_SRWDM_5th_printing.pdf


Cost Impact: The code change proposal will increase the cost of construction
Compared to SRW systems that are not designed to any minimum criteria, this change would in theory increase the cost of construction.

Staff Analysis: A review of the standard proposed for inclusion in the code, ASTM C1372-17, NCMA TR127B-10, 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.

Proposal # 5303
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 outdated requirements from the IBC. Current concrete mixes are commonly designed with admixtures to better improve and assure placement using funnel hopper and this set of criteria specifying slump is no longer required in the code. The information in IBC Section 1808.1 is outdated as the slump criteria is only applicable for concrete mix designs not containing admixtures used for proper placement. Where such admixtures are used the slump requirement is likely not to be satisfied.

ACI, a professional technical society, recommends the deletion of this outdated criteria and 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.

This code change eliminates antiquated prescriptive criteria, allowing admixtures to achieve necessary properties and increase affordability.
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 pumppability of concrete while retaining the other necessary properites of the concrete.
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

<table>
<thead>
<tr>
<th>FOUNDATION ELEMENT OR CONDITION</th>
<th>SPECIFIED COMPRESSIVE STRENGTH $f'_c$, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Foundations for structures assigned to Seismic Design Category A, B or C</td>
<td>2,500 psi</td>
</tr>
<tr>
<td>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</td>
<td>2,500 psi</td>
</tr>
<tr>
<td>2b. Foundations for other structures assigned to Seismic Design Category D, E or F</td>
<td>3,000 psi</td>
</tr>
<tr>
<td>3. Precast nonprestressed driven piles</td>
<td>4,000 psi</td>
</tr>
<tr>
<td>4. Socketed drilled shafts</td>
<td>4,000 psi</td>
</tr>
<tr>
<td>5. Micropiles</td>
<td>4,000 psi</td>
</tr>
<tr>
<td>6. Precast prestressed driven piles</td>
<td>5,000 psi</td>
</tr>
</tbody>
</table>

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 changes 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

<table>
<thead>
<tr>
<th>Foundation Element of Condition</th>
<th>2018 IBC $f'_c$, psi</th>
<th>ACI 318 $f'_c$, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Foundations for structures assigned to Seismic Design Category A, B or C</td>
<td>2,500</td>
<td>2,500</td>
</tr>
<tr>
<td>2a. Foundations for two stories or less in height, assigned to Seismic Design Category D, E or F</td>
<td>2,500</td>
<td>3,000</td>
</tr>
<tr>
<td>2b. Foundations for other structures assigned to Seismic Design Category D, E or F2</td>
<td>3,000</td>
<td>5,000</td>
</tr>
</tbody>
</table>

1. Special Structural walls with Grade 60 or 80 reinforcement

2. Special Structural walls with Grade 100 reinforcement

For SI: 1 pound per square inch = 0.00689 MPa.
<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>3. Precast nonprestressed driven piles</td>
<td>4,000 psi</td>
<td>4,000</td>
</tr>
<tr>
<td>4. Socketed drilled shafts</td>
<td>4,000 psi</td>
<td>4,000</td>
</tr>
<tr>
<td>5. Micropiles</td>
<td>4,000 psi</td>
<td>4,000</td>
</tr>
<tr>
<td>6. Precast prestressed driven piles</td>
<td>5,000 psi</td>
<td>5,000</td>
</tr>
</tbody>
</table>

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.
2018 International Building Code

Revise as follows:

**1808.8.2 Concrete cover.** The concrete cover provided for prestressed and nonprestressed reinforcement in 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\(\frac{1}{8}\) 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**

<table>
<thead>
<tr>
<th>FOUNDATION ELEMENT OR CONDITION</th>
<th>MINIMUM COVER</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Shallow foundations</td>
<td>In accordance with Section 20.6 of ACI 318</td>
</tr>
<tr>
<td>2. Precast nonprestressed deep foundation elements Exposed to seawater Not manufactured under plant conditions Manufactured under plant control conditions</td>
<td>2 inches In accordance with Section 20.6.1.3.3 of ACI 318</td>
</tr>
<tr>
<td>3. Precast prestressed deep foundation elements Exposed to seawater Other</td>
<td>2.5 inches In accordance with Section 20.6.1.3.3 of ACI 318</td>
</tr>
<tr>
<td>4. Cast in-place deep foundation elements not enclosed by a steel pipe, tube or permanent casing</td>
<td>2.5 inches</td>
</tr>
<tr>
<td>5. Cast in-place deep foundation elements enclosed by a steel pipe, tube or permanent casing</td>
<td>1 inch</td>
</tr>
<tr>
<td>6. Structural steel core within a steel pipe, tube or permanent casing</td>
<td>2 inches</td>
</tr>
<tr>
<td>7. Cast-in-place drilled shafts enclosed by a stable rock socket</td>
<td>1.5 inches</td>
</tr>
</tbody>
</table>

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.
S121-19

IBC: 1808.8.7(New)

Proponent: Terry Kozlowski, representing Southern Nevada Chapter; Valarie Evans, 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

Add new text as follows:

1808.8.7 Use of non-structural slabs on ground to resist bearing loads. Where bearing loads are proposed to be resisted by non-structural slabs on ground, all of the following conditions shall be satisfied:

1. Structural calculations shall be provided to show the slab can adequately support the proposed load.
2. The maximum allowable subgrade bearing pressure below the slab shall be no greater than 750 psf., with no increases allowed for short duration loads, unless a greater value is justified in a geotechnical investigation report.
3. Presumptive load-bearing values shall apply to class of materials identified in Table 1806.2 as crystalline bedrock; sedimentary and foliated rock; sandy gravel and gravel (GW and GP); sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC).

Reason: This proposal correlates with Appendix I (patio covers) Section I105.2. Non-structural slabs on ground are typically exempt from structural design and special inspection requirements in the code. There may be limited requirements for subgrade preparation below non-structural slabs. Provided that the allowable capacity can be demonstrated through calculation, non-structural slabs on ground can be utilized for architectural, electrical, mechanical and plumbing components. The 750 psf. allowable bearing limit takes into account that a slab on ground would have less than the required 12” embedment depth typical to conventional spread footings.

Bibliography: I105.2 Footings. In areas with a frost depth of zero, a patio cover shall be permitted to be supported on a concrete slab on grade without footings, provided that the slab conforms to the provisions of Chapter 19 of this code and is not less than 31/2 inches (89 mm) thick, and the columns do not support loads in excess of 750 pounds (3.36 kN) per column.

Cost Impact: The code change proposal will increase the cost of construction
This proposal will decrease the cost of construction by eliminating the requirement for footings by using non-structural slabs that comply with this section.

Proposal # 4402

S121-19
1809.5 Frost protection. Except where otherwise protected from frost, foundations and other permanent supports of buildings and structures shall be protected from frost by one or more of the following methods:

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

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

1. Assigned to Risk Category I.
2. Area of 600 square feet (56 m²) or less for light-frame construction or 400 square feet (37 m²) or less for other than light-frame construction.
3. Eave height of 10 feet (3048 mm) or less.

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

Add new text as follows:

**1809.5.1 Frost Protection at Required Exits.** Frost protection shall be provided at exterior landings for all required exits. Frost protection shall only be required to the extent necessary to ensure the unobstructed opening of the required exit doors.

**Exception:** Landings that serve exits which do not have outward swinging doors.

**Reason:** The proposed change is to establish a minimum standard that the exterior landings at required outswinging egress doors (when located in climates subject to frost conditions) be provided with frost protection as required for the primary structure. Adding a section for frost protection at required exits will clarify that the landing areas immediately adjacent to all required egress doors must be provided with the same frost protection systems as that of the building being served by the exit. In cold climate areas, this would help prevent concrete landings (at the exit discharge), from heaving and inevitably compromising the normal operation of the required egress door(s). Such heaving actions have been documented to render an egress door entirely unusable (please see Bibliography for a link to a news report on several compromised doors in Vermont). This creates a dire situation for occupancies permitted to have a single egress. There are numerous conditions that can contribute to concrete heaving, making it impossible to predict when and where such heaving may materialize. Additionally, heaving of concrete landings can significantly impact accessibility to the structure. The proposed language is intended to provide the heave protection only for the area of a landing immediately adjacent to the exit door(s) and only for the area required to allow the door to swing open at least 90 degrees from the closed position. The remaining portions of a larger patio or sidewalk need not be provided with the frost protection. Protection would only be required to assure the required egress door(s) will operate.


**Cost Impact:** The code change proposal will increase the cost of construction.

Additional frost protection of landings will negligibly increase the cost of construction for foundations when compared to the overall cost of the foundation system, bearing in mind the minimal area added for the landings, and the fact that this is only required at required egress doors, and not all exterior doors. Initial installation of frost protected landings is significantly lower than the cost of retrofitting frost protection systems or the maintenance and/or repairs to egress doors when heaving has occurred.
S123-19

Proponent: 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:

1810.3.2.4 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.

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, $\Omega$. In such cases, minimum spiral reinforcement index shall be as specified in Section 13.4.5.6 of ACI 318.

Delete without substitution:

1810.3.8.1.1 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:

4. 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 \(\frac{a}{6}\) 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 1810.3.8.3.2.

**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:

\[
\rho_s = 0.003 \left( \frac{f_{pc}}{f_y} \right) \left( \frac{A_p}{A_v} \right)
\]

where:

- \(A_p\) = Pile cross-sectional area square inches (mm²);
- \(f_{pc}\) = Specified compressive strength of concrete, psi (MPa);
- \(f_y\) = Yield strength of spiral reinforcement ≤ 85,000 psi (586 MPa);
- \(A_v\) = Axial load on pile, pounds (kN), as determined from Equations 16-5 and 16-7;
- \(\rho_s\) = 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, \(\Omega\). 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 36 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 36 feet (10 668 mm), the ductile pile region shall be taken as the greater of 36 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 0 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.
Where the transverse reinforcement consists of circular spirals, the volumetric ratio of spiral transverse reinforcement in the ductile region shall comply with the following:

\[ n = \frac{0.005 f'_c}{\rho A_p} \]

(Equation 18-6)

but not exceed:

\[ n = \frac{f_s}{f_p} \]

(Equation 18-7)

where:

- \( A_p \) = Pile cross-sectional area, square inches (mm²).
- \( f'_c \) = Specified compressive strength of concrete, psi (MPa).
- \( f_s \) = 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.
- \( \rho \) = Volumetric ratio \((\text{vol. spiral/ vol. core})\).

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, \( \Omega \). 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, \( h \), shall conform to:

\[ A = 0.125 f'_c \left( \frac{1}{s} \right)^2 \left[ \frac{A_h + 1.4P}{2} + 1 \right] \]

(Equation 18-8)

but not less than:

\[ A = 0.125 f'_c \left( \frac{1}{s} \right)^2 \left[ \frac{A_h + 1.4P}{2} + 1 \right] \]

(Equation 18-9)

where:

- \( f_s \) = Yield strength of transverse reinforcement \( \leq 70,000 \text{ psi} \) (483 MPa).
- \( h \) = 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).
- \( A_h \) = 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. \( 0.2 f'_c A_p \) for square piles
2. \( 0.4 f'_c A_p \) 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 1810.3.9.5; 1810.3.9.4.
Delete without substitution:

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

\[
\Phi M_c = f'_c \cdot S
\]

(Equation 18.10)

where:

- \( f'_c \) = Specified compressive strength of concrete or grout, psi (MPa).
- \( S \) = 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 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 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 the concrete cover is 3 inches (76 mm) or less 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...
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(e) 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.
  - 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.
  - Section 1810.3.2.1.1: Is covered by Section 18.13.5.4 in ACI 318.
  - Section 1810.3.2.1.2: Is covered by Section 25.7.3.3 in ACI 318.
  - 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.
  - Section 1810.3.8.1→13.4.5.2 and 13.4.5.6
  - Section 1810.3.8.2.1→13.4.5.3
  - Section 1810.3.8.2.2→18.13.5.10.2
  - § Exception remains
  - Section 1810.3.8.2.3→18.13.5.10.3
  - § Exception remains
  - Section 1810.3.8.3.1→13.4.5.4 and 13.4.5.5
  - Section 1810.3.8.3.2→18.13.5.10.4
  - Section 1810.3.8.3.3→18.13.5.10.5
  - 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.
  - Section 1810.3.9.1→13.4.4
  - Section 1810.3.9.2→13.4.4
  - Section 1810.3.9.3→Remains
  - Section 1810.3.9.4→Remains, update reference
  - Section 1810.3.9.4.1→18.13.5.7
  - § Exception→18.13.5.8
  - Section 1810.3.9.4.2→18.13.5.7
  - § Exception→18.13.5.8
  - Section 1810.3.9.4.2.1→18.13.5.5
### IBC 2018

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<th>Section 1810.3.2.1.1</th>
<th>ACI 318</th>
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<td><strong>Seismic hooks.</strong> 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><strong>For structures assigned to SDC C, D, E, or F, hoops, spirals, and ties in deep foundation members shall be terminated with seismic hooks.</strong></td>
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### IBC 2018

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<th>ACI 318</th>
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<tr>
<td><strong>ACI 318 Equation (25.7.3.3).</strong> 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><strong>25.7.3.3</strong> Except for transverse reinforcement in deep foundations, the volumetric spiral reinforcement ratio $p_s$ shall satisfy Eq. (25.7.3.3).</td>
</tr>
</tbody>
</table>

### IBC 2018

<table>
<thead>
<tr>
<th>Section 1810.3.2.2</th>
<th>ACI 318</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Prestressing steel.</strong> Prestressing steel shall conform to ASTM A416.</td>
<td><strong>20.3</strong> Prestressing strands, wires, and bars</td>
</tr>
</tbody>
</table>

#### 20.3 Material properties

<table>
<thead>
<tr>
<th><strong>20.3.1</strong></th>
<th><strong>20.3.1.1</strong> 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):</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
<td>ASTM A416 – strand</td>
</tr>
<tr>
<td>(b)</td>
<td>ASTM A421 – wire</td>
</tr>
<tr>
<td>(c)</td>
<td>ASTM A421 – low-relaxation wire including Supplementary Requirement S1, “Low-Relaxation Wire and Relaxation Testing”</td>
</tr>
<tr>
<td>(d)</td>
<td>ASTM A722 – high-strength bar</td>
</tr>
</tbody>
</table>

### IBC 2018

<table>
<thead>
<tr>
<th>Section 1810.3.8</th>
<th>ACI 318</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Precast concrete piles.</strong> Precast concrete piles shall be designed and detailed in accordance with Sections 1810.3.8.1 through 1810.3.8.3.</td>
<td><strong>13.4.5</strong> Precast concrete piles</td>
</tr>
</tbody>
</table>

#### 13.4.5 Precast concrete piles

| **13.4.5.1** | **13.4.5.2** Longitudinal reinforcement shall be arranged in a symmetrical pattern. |

<table>
<thead>
<tr>
<th><strong>13.4.5.3</strong></th>
<th>For precast non prestressed piles, longitudinal reinforcement shall be provided according to (a) and (b):</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
<td>Minimum of 4 bars</td>
</tr>
<tr>
<td>(b)</td>
<td>Minimum area of $0.008A_g$</td>
</tr>
</tbody>
</table>

#### 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>
<thead>
<tr>
<th><strong>Table 13.4.5.4</strong> Minimum compressive stress in precast prestressed piles</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Pile length (ft)</strong></td>
<td><strong>Minimum compressive stress (psi)</strong></td>
</tr>
<tr>
<td>15</td>
<td>30</td>
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<tr>
<td>16</td>
<td>30</td>
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<tr>
<td>99</td>
<td>30</td>
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<tr>
<td>100</td>
<td>30</td>
</tr>
</tbody>
</table>
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.

1810.3.8.3 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.

<table>
<thead>
<tr>
<th>Pile length ≤ 30</th>
<th>400</th>
</tr>
</thead>
<tbody>
<tr>
<td>30 &lt; Pile length ≤ 50</td>
<td>550</td>
</tr>
<tr>
<td>Pile length &gt; 50</td>
<td>700</td>
</tr>
</tbody>
</table>

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>
<thead>
<tr>
<th>Least horizontal pile dimension-h (in.)</th>
<th>Minimum wire size transverse reinforcement[1]</th>
</tr>
</thead>
<tbody>
<tr>
<td>h ≤ 16</td>
<td>W4, D4</td>
</tr>
<tr>
<td>16 &lt; h &lt; 20</td>
<td>W4.5, D5</td>
</tr>
<tr>
<td>h ≥ 20</td>
<td>W5.5, D6</td>
</tr>
</tbody>
</table>

[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

<table>
<thead>
<tr>
<th>Reinforcement location in the pile</th>
<th>Maximum center-to-center spacing (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>First five ties or spirals at each end of pile</td>
<td>1</td>
</tr>
<tr>
<td>24 in. from each end of pile</td>
<td>4</td>
</tr>
<tr>
<td>Remainder of pile</td>
<td>6</td>
</tr>
</tbody>
</table>

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 ($\phi M_c$) for a cast-in-place deep foundation element not enclosed by a structural steel pipe or tube shall be determined using the following equation:

$$\phi M_c = 3\sqrt{f'_c S_m} \text{ (Equation 18-10)}$$

For SI: $0.25\sqrt{f'_c S_m}$

where:

$f'_c$ = Specified compressive strength of concrete or grout,

13.4.4 Cast-in-place deep foundations

13.4.4.1 Cast-in-place deep foundations that are subject to uplift or where $M_c$ is greater than 0.4 $M_{cr}$ shall be reinforced, unless enclosed by a structural steel pipe or tube.

Note $f_{cr} = 7.5f'_c$
psi (MPa).

\[ S_m = \text{Elastic section modulus, neglecting reinforcement and casing, cubic inches (mm}^3) \]  

### 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.

### 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

### 18.13 Deep Foundations

#### 18.13.5 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.

#### 18.13.5.5 For structures assigned to SDC D, E, or F or located in Site Class E or F, concrete piles, piers, or caissons 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
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 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 $0.4M_{cr} > M_c$.

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 $0.01A_p$ 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.
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:

<table>
<thead>
<tr>
<th>Minimum Longitudinal Reinforcement Ratio</th>
<th>SDC C – All Site Classes</th>
<th>SDC D, E, and F – Site Class A, B, C, and D</th>
<th>SDC D, E, and F – Site Class E and F</th>
</tr>
</thead>
<tbody>
<tr>
<td>(minimum number of bars in accordance with 10.7.3.1)</td>
<td>0.0025</td>
<td>0.005</td>
<td>0.005</td>
</tr>
<tr>
<td>Minimum Reinforced Pile Length</td>
<td>Longest of (a) through (d):</td>
<td>Longest of (a) through (d):</td>
<td>Full length of pile except in accordance with [1] or [2].</td>
</tr>
<tr>
<td>(a) 1/3 pile length</td>
<td>(a) ½ pile length</td>
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<tr>
<td>(b) 10 ft.</td>
<td>(b) 10 ft.</td>
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<td></td>
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<tr>
<td>(c) 3 times the pile diameter</td>
<td>(c) 3 times the pile diameter</td>
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<tr>
<td>(d) flexural length of pile - distance from bottom of pile cap to where $0.4M_u$ exceeds $M_u$.</td>
<td>(d) flexural length of pile - distance from bottom of pile cap to where $0.4M_u$ exceeds $M_u$.</td>
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<td></td>
</tr>
<tr>
<td>Transverse Confinement Reinforcement Zone</td>
<td>Length of Reinforcement Zone</td>
<td>3 times the pile diameter from the bottom of the pile cap</td>
<td>3 times the pile diameter from the bottom of the pile cap.</td>
</tr>
<tr>
<td>Type of Transverse Reinforcement</td>
<td>Closed ties or spirals with a minimum 3/8 in. diameter.</td>
<td>Minimum of No. 3 closed tie or 3/8 in. diameter spiral for piles ≤ 20 in. diameter.</td>
<td>Minimum No. 4 closed tie or 1/2 in. diameter spiral for piles &gt; 20 in. diameter.</td>
</tr>
<tr>
<td>Spacing and Amount of Transverse Reinforcement</td>
<td>Spacing shall not exceed lesser of 6 in. or 8 longitudinal bar diameters</td>
<td>In accordance with 18.7.5.3 and not less than one-half the requirement of Table 18.7.5.4(e)</td>
<td>In accordance with 18.7.5.3 and not less than the requirement of Table 18.7.5.4(e).</td>
</tr>
<tr>
<td>Transverse Reinforcement in Remainder of Reinforced Pile Length</td>
<td>Type of Transverse Reinforcement</td>
<td>Closed ties or spirals with minimum 3/8 in. diameter.</td>
<td>Minimum of No. 3 closed tie or 3/8 in. diameter spiral for piles ≤ 20 in. diameter.</td>
</tr>
</tbody>
</table>
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) \( \frac{1}{2} \) 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.

1813.5.10 Precast concrete piles

1813.5.10.1 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.

1813.5.10.2 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.

1813.5.10.3 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.

1813.5.10.4 For structures assigned to SDC C, precast prestressed concrete piles shall satisfy (a) and (b):

(a) If the transverse reinforcement consists of
$A_p$ = Pile cross-sectional area square inches (mm$^2$).

$f_c' = $ Specified compressive strength of concrete, psi (MPa).

$f_y = $ Yield strength of spiral reinforcement $\leq 85,000$ psi (586 MPa).

$P = $ Axial load on pile, pounds (kN), as determined from Equations 16-5 and 16-7.

$\rho_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, $\Omega$. 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 concrete 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 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.

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, and 6 in.

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:

$$\rho_s = 0.06(\frac{f_y}{f_y})[2.8 + 2.34P/ f_y A_y]$$ (Equation 18-6) but not spirals or circular hoops, the volumetric ratio of transverse reinforcement, $\rho_s$, 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):

$$0.15(\frac{f_y}{f_y})$$ (18.13.5.10.4a)

$$0.04(\frac{f_y}{f_y})[2.8 + 2.3P/ f_y A_y]$$ (18.13.5.10.4b)

and $f_y$ 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.

1813.5.10.5 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, $\rho_s$, 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.

$$0.2(\frac{f_y}{f_y})$$ (18.13.5.10.5a)

$$0.06(\frac{f_y}{f_y})[2.8 + 2.3P/ f_y A_y]$$ (18.13.5.10.5b)

and $f_y$ 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).
exceed: $\rho_s = 0.021$ (Equation 18-7)

where:

$$A_b = \text{Pile cross-sectional area, square inches (mm}^2).$$

$$f'_c = \text{Specified compressive strength of concrete, psi (MPa).}$$

$$f_{sy} = \text{Yield strength of spiral reinforcement } \leq 85,000 \text{ psi (586 MPa).}$$

$$P = \text{Axial load on pile, pounds (kN), as determined from Equations 16-5 and 16-7.}$$

$$\rho_s = \text{Volumetric ratio (vol. spiral/vol. core).}$$

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, $\Omega$. 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:

$$A_{sh} = 0.3sh_c(f'_c/f)\left(A_y/A_{sh} - 1.0\right)[0.5 + 1.4P/(f'_cA_y)]$$

(Equation 18-8)

but not less than:

$$A_{sh} = 0.12sh_c(f'_c/f)h [0.5 + 1.4P/(f'_cA_y)]$$

(Equation 18-9)

where:

$$f_{sy} = \text{yield strength of transverse reinforcement } \leq 70,000 \text{ psi (483 MPa).}$$

$$h_c = \text{Cross-sectional dimension of pile core measured center to center of hoop reinforcement, inch (mm).}$$

$$s = \text{Spacing of transverse reinforcement measured along length of pile, inch (mm).}$$

$$A_{sh} = \text{Cross-sectional area of transverse reinforcement, square inches (mm}^2).$$

$$f'_c = \text{Specified compressive strength of concrete, psi (MPa).}$$

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. \(0.2f'_c A_0\) for square piles
2. \(0.4f'_c A_0\) 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 potential conflicts.
S124-19

IBC®: TABLE 1810.3.2.6

Proponent: 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:

TABLE 1810.3.2.6
ALLOWABLE STRESSES FOR MATERIALS USED IN DEEP FOUNDATION ELEMENTS

Portions of table not shown remain unchanged.

<table>
<thead>
<tr>
<th>MATERIAL TYPE AND CONDITION</th>
<th>MAXIMUM ALLOWABLE STRESSa</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Concrete or grout in compressionb</td>
<td></td>
</tr>
<tr>
<td>Cast-in-place with a permanent casing in accordance with Section 1810.3.2.7 or Section 1810.3.5.3.4</td>
<td>0.4 f'c</td>
</tr>
<tr>
<td>Cast-in-place in a pipe, tube, other permanent casing or rock</td>
<td>0.33 f'c</td>
</tr>
<tr>
<td>Cast-in-place without a permanent casing</td>
<td>0.3f'c</td>
</tr>
<tr>
<td>Precast nonprestressed</td>
<td>0.33f'c</td>
</tr>
<tr>
<td>Precast prestressed</td>
<td>0.33f'c - 0.27 fpc</td>
</tr>
</tbody>
</table>

a. f'c is the specified compressive strength of the concrete or grout; fpc is the compressive stress on the gross concrete section due to effective prestress forces only; fy is the specified yield strength of reinforcement; Fu is the specified minimum yield stress of steel; Fu is the specified minimum tensile stress of structural steel.

b. The stresses specified apply to the gross cross-sectional area within the concrete surface. Where a temporary or permanent casing is used, the inside face of the casing shall be considered to be the concrete surface.

Reason:
1. The code currently allows 0.40 f'c for thin-wall casing (Section 1810.3.2.7) because the concrete is confined. It is reasonable to allow the same 0.40 f'c when the concrete is confined by thicker-wall pipe or tube (in accordance with Section 1810.3.5.3.4).
2. Section 1810.3.2.7 is “Increased allowable compressive stress for cased mandrel-driven cast-in-place elements” which is a legacy requirement mainly for Raymond Step-taper corrugated shell piles which are much weaker than any pipe or tube defined in Section 1810.3.5.3.4.
3. Section 1810.3.5.3.4 is “Steel pipes and tubes” which have better confining ability than corrugated shells.
4. We have not increased the current limit for other permanent casing or rock because of the uncertain nature of “other permanent casing” and because rock is of variable quality.

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
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.

Proposal # 4555
2018 International Building Code

Revised as follows:

<table>
<thead>
<tr>
<th>MATERIAL TYPE AND CONDITION</th>
<th>MAXIMUM ALLOWABLE STRESS(^a)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3. Steel in compression</td>
<td></td>
</tr>
<tr>
<td>Cores within concrete-filled pipes or tubes</td>
<td>(0.5 F_y \leq 32,000 \text{ psi})</td>
</tr>
<tr>
<td>Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8</td>
<td>(0.5 F_y \leq 32,000 \text{ psi})</td>
</tr>
<tr>
<td>Pipes or tubes for microplates</td>
<td>(0.4 F_y \leq 32,000 \text{ psi})</td>
</tr>
<tr>
<td>Other pipes, tubes or H-piles</td>
<td>(0.35 F_y \leq 24,000 \text{ psi})</td>
</tr>
<tr>
<td>Helical piles</td>
<td>(0.6 F_y \leq 0.5 F_{u})</td>
</tr>
<tr>
<td>5. Steel in tension</td>
<td></td>
</tr>
<tr>
<td>Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8</td>
<td>(0.5 F_y \leq 32,000 \text{ psi})</td>
</tr>
<tr>
<td>Other pipes, tubes or H-piles</td>
<td>(0.35 F_y \leq 24,000 \text{ psi})</td>
</tr>
<tr>
<td>Helical piles</td>
<td>(0.6 F_y \leq 0.5 F_{u})</td>
</tr>
</tbody>
</table>

\(f'c\) is the specified compressive strength of the concrete or grout; \(f_p\) is the compressive stress on the gross concrete section due to effective prestress forces only; \(f_y\) is the specified yield strength of reinforcement; \(F_y\) is the specified minimum yield stress of steel; \(F_{u}\) is the specified minimum tensile stress of structural steel.

The stresses specified apply to the gross cross-sectional area within the concrete surface. Where a temporary or permanent casing is used, the inside face of the casing shall be considered to be the concrete surface.

Reason: 1. The proposal does not change the 0.35 \(F_y\) limit, but only increases the allowable upper limit to 24,000 psi because the yield strength for commonly available steel pilings has increased significantly. When the 16,000 psi upper limit was first established, the common steel yield was perhaps only 36,000 psi (e.g. A36). In 2018, yield strengths are normally 50,000 or 60,000 psi, and yields above 70,000 psi are available and in common use for piling. The code should consider the currently available materials to achieve an economic design.

2. Micropiles have a limit of 0.4 \(F_y\) ≤ 32,000 psi, which implies a yield of up to 80,000 psi. The same steel can be used in larger diameter pipes and 0.35\(F_y\) of 80,000 psi is 28,000 psi.

Cost Impact: The code change proposal will decrease the cost of construction. Limiting the upper level of allowable stress to 16,000 psi stress can cause a significant increase in cost of construction on some building projects. Increasing the upper limit will result in a reduction of construction cost.

Proposal # 5077
S126-19

IBC®: TABLE 1810.3.2.6

Proponent: 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:

TABLE 1810.3.2.6
ALLOWABLE STRESSES FOR MATERIALS USED IN DEEP FOUNDATION ELEMENTS

Table 1810.3.2.6

<table>
<thead>
<tr>
<th>MATERIAL TYPE AND CONDITION</th>
<th>MAXIMUM ALLOWABLE STRESSa</th>
</tr>
</thead>
<tbody>
<tr>
<td>4. Nonprestressed reinforcement in tension</td>
<td></td>
</tr>
<tr>
<td>Within micropiles</td>
<td>0.6 $f_y$</td>
</tr>
<tr>
<td>Other conditions</td>
<td></td>
</tr>
<tr>
<td>For load combinations that do not include wind or seismic loads</td>
<td>$0.5 f_y \leq 24,000 - 30,000$ psi</td>
</tr>
<tr>
<td>For load combinations that include wind or seismic loads</td>
<td>$0.5 f_y \leq 40,000$ psi</td>
</tr>
</tbody>
</table>

a. $f'_c$ is the specified compressive strength of the concrete or grout; $f_p$ is the compressive stress on the gross concrete section due to effective prestress forces only; $f_y$ is the specified yield strength of reinforcement; $F_y$ is the specified minimum yield stress of steel; $F_u$ is the specified minimum tensile stress of structural steel.

b. The stresses specified apply to the gross cross-sectional area within the concrete surface. Where a temporary or permanent casing is used, the inside face of the casing shall be considered to be the concrete surface.

Reason: 1. Limiting stresses to 30,000 psi will reduce cracking and the potential for corrosion for permanent load conditions, not including wind or earthquake.

2. Limiting stresses to 40,000 psi for temporary load conditions, including wind or earthquake, is appropriate (see reason 3 below).

3. Section 1901.2 states that structural concrete shall be designed and constructed in accordance with the requirements of IBC Chapter 19 and ACI 318. ACI 318 establishes an upper limit of $F_y$ of 80,000 psi for non-prestressed reinforcing and therefore setting the upper limit at 40,000 psi (50% of 80,000 psi) provides consistency between IBC and ACI 318. ACI 318 design methods will typically produce a service level tension stress between 0.56 $F_y$ and 0.64 $F_y$, therefore the code’s current allowable stress of 0.50 $F_y$ is slightly conservative compared to ACI 318.

4. IBC section 1810.3.1.1 allows concrete deep foundation elements to be designed using approved strength design methods, which in most cases will be ACI 318. The proposed revisions will provide for greater consistency between allowable stress design and strength design methods.

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Cost Impact: The code change proposal will decrease the cost of construction

The 24,000 psi stress limit has led to a significant increase in cost of construction on some building projects. Increasing the upper limit to a level consistent with that allowed by ACI 318 will result in a reduction of construction cost.

Proposal # 5042

S126-19
### IRC: TABLE 1810.3.2.6

**Proponent:** Dale Biggers, P.E. GeoCoalition, representing GeoCoalition (dbiggers@bohbros.com); Lori Simpson, P.E., G.E., representing GeoCoalition (lsimpson@langan.com); Daniel Stevenson, P.E., representing GeoCoalition (dstevenson@berkelapg.com)

#### 2018 International Building Code

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

Revise as follows:

```
<table>
<thead>
<tr>
<th>MATERIAL TYPE AND CONDITION</th>
<th>MAXIMUM ALLOWABLE STRESS ( \text{psi} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Concrete or grout in compression (^\text{a}) Cast-in-place with a permanent casing in accordance with Section 1810.3.2.7 Cast-in-place in a pipe, tube, other permanent casing or rock Cast-in-place without a permanent casing Precast nonprestressed Precast prestressed</td>
<td>( 0.4 f'_c \leq 0.33 f'_c \leq 0.3f'_c \leq 0.33f'<em>c - 0.27 f'</em>{pc} )</td>
</tr>
<tr>
<td>2. Nonprestressed reinforcement in compression</td>
<td>( 0.4 f_y \leq 30,000 \text{ psi} )</td>
</tr>
<tr>
<td>3. Steel in compression Cores within concrete-filled pipes or tubes Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8 Pipes or tubes for micropiles Other pipes, tubes or H-piles Helical piles</td>
<td>( 0.5 F_y \leq 32,000 \text{ psi} 0.4 F_y \leq 32,000 \text{ psi} 0.35 F_y \leq 16,000 \text{ psi} 0.6 F_y \leq 0.5 F_u )</td>
</tr>
<tr>
<td>4. Nonprestressed reinforcement in tension Within micropiles Other conditions</td>
<td>( 0.6 f_y \leq 0.5 f_y \leq 24,000 \text{ psi} )</td>
</tr>
<tr>
<td>5. Steel in tension Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8 Other pipes, tubes or H-piles Helical piles</td>
<td>( 0.5 F_y \leq 32,000 \text{ psi} 0.35 F_y \leq 16,000 \text{ psi} 0.6 F_y \leq 0.5 F_u )</td>
</tr>
<tr>
<td>6. Timber</td>
<td>In accordance with the ANSI/AWC NDS</td>
</tr>
</tbody>
</table>

\( \text{a.} \) \( f'_c \) is the specified compressive strength of the concrete or grout; \( f'_{pc} \) is the compressive stress on the gross concrete section due to effective prestress forces only; \( f_y \) is the specified yield strength of reinforcement; \( F_y \) is the specified minimum yield stress of steel; \( F_u \) is the specified minimum tensile stress of structural steel.

\( \text{b.} \) The stresses specified apply to the gross cross-sectional area within of the concrete surface for precast prestressed piles and to the net cross-sectional area for all other piles. Where a temporary or permanent casing is used, the inside face of the casing shall be considered to be the outer edge of the concrete surface cross-section.

**Reason:** The stresses should apply to the net area ( gross area - steel reinforcement area ) not to the gross sectional area. This would be consistent with other codes.

Click here to see the members of the GeoCoalition: [http://www.piledrivers.org/2019-geocoalition-members/](http://www.piledrivers.org/2019-geocoalition-members/)

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

Proposal # 4742
2018 International Building Code

Delete without substitution:

1810.3.2.7 Increased allowable compressive stress for cased mandrell-driven cast-in-place elements. The allowable compressive stress in the concrete shall be permitted to be increased as specified in Table 1810.3.2.6 for those portions of permanently cased cast-in-place elements that satisfy all of the following conditions:

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

Revise as follows:

TABLE 1810.3.2.6
ALLOWABLE STRESSES FOR MATERIALS USED IN DEEP FOUNDATION ELEMENTS

<table>
<thead>
<tr>
<th>MATERIAL TYPE AND CONDITION</th>
<th>MAXIMUM ALLOWABLE STRESS*</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Concrete or grout in compression&lt;sup&gt;a&lt;/sup&gt;</td>
<td>In accordance with ACI 318</td>
</tr>
<tr>
<td>Cast-in-place with a permanent casing in accordance with Section 1810.3.2.7</td>
<td>(0.4 f'_c)</td>
</tr>
<tr>
<td>Cast-in-place in a pipe, tube, other permanent casing or rock</td>
<td>(0.33 f'_c)</td>
</tr>
<tr>
<td>Cast-in-place without a permanent casing</td>
<td>(0.3 f'_c)</td>
</tr>
<tr>
<td>Precast nonprestressed</td>
<td>(0.33 f'_c)</td>
</tr>
<tr>
<td>Precast prestressed</td>
<td>(0.33 f'_c - 0.27 f_y)</td>
</tr>
<tr>
<td>2. Nonprestressed reinforcement in compression</td>
<td>(0.4 f_y \leq 30,000) psi</td>
</tr>
<tr>
<td>3. Steel in compression</td>
<td>(0.5 F_y \leq 32,000) psi</td>
</tr>
<tr>
<td>Cores within concrete-filled pipes or tubes</td>
<td>(0.5 F_y \leq 32,000) psi</td>
</tr>
<tr>
<td>Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8</td>
<td>(0.4 F_y \leq 32,000) psi</td>
</tr>
<tr>
<td>Pipes or tubes for micropiles</td>
<td>(0.35 F_y \leq 16,000) psi</td>
</tr>
<tr>
<td>Other pipes, tubes or H-piles</td>
<td>(0.6 F_y \leq 0.5 F_u)</td>
</tr>
<tr>
<td>Helical piles</td>
<td>(0.6 f_y)</td>
</tr>
<tr>
<td>4. Nonprestressed reinforcement in tension</td>
<td>(0.5 f_y \leq 24,000) psi</td>
</tr>
<tr>
<td>Within micropiles</td>
<td>(0.6 f_y)</td>
</tr>
<tr>
<td>Other conditions</td>
<td>(0.5 f_y \leq 24,000) psi</td>
</tr>
</tbody>
</table>
5. Steel in tension

Pipes, tubes or H-piles, where justified in accordance with Section 1810.3.2.8

Other pipes, tubes or H-piles

Helical piles

- $F_y \leq 32,000$ psi
- $0.35 F_u \leq 16,000$ psi
- $0.6 F_y \leq 0.5 F_u$

6. Timber

In accordance with the ANSI/AWC NDS

**Reason:** This proposed code change makes four modifications to the 2018 IBC:

1. In Section 1810.3.2.6, ACI 318 is added to Item 1 as the method for design and construction of concrete deep foundations. This aligns the code with the methodology in ACI 318. A comparison of the provisions removed from the 2018 IBC and the requirements in ACI 318 are shown in Table 1.

2. In Section 1810.3.2.7, all text is deleted as this information is provided in ACI 318. Comparison of the text in 2018 IBC and ACI 318 is shown in Table 2. Requirements are identical, except ACI 318 language more clearly communicates that there are other permissible design and construction methods in accordance with Chapter 10 of ACI 318.

**Table 1**

**Comparison of 2018 IBC and ACI 318 Requirements**

<table>
<thead>
<tr>
<th>2018 IBC</th>
<th>ACI 318</th>
</tr>
</thead>
<tbody>
<tr>
<td>1810.3.2.7 Increased allowable compressive stress for cased mandrel-driven cast-in-place elements. The allowable compressive stress in the concrete shall be permitted to be increased as specified in Table 1810.3.2.6 for those portions of permanently cased cast-in-place elements that satisfy all of the following conditions:</td>
<td>13.4.2 Allowable axial capacity</td>
</tr>
<tr>
<td>1. The design shall not use the casing to resist any portion of the axial load imposed.</td>
<td>13.4.2.1 Where concrete deep foundation elements are laterally supported for the entire height and the applied forces cause bending moments no greater than those resulting from accidental eccentricities, structural design of the element using unfactored loads and the allowable capacities specified in Table 13.4.2.2 is permitted. Otherwise, the structural design of concrete deep foundation elements shall be in accordance with Chapter 10.</td>
</tr>
<tr>
<td>2. The casing shall have a sealed tip and be mandrel driven.</td>
<td>13.4.2.2 The maximum allowable axial capacity of deep foundation members shall be in accordance with Table 13.4.2.2.</td>
</tr>
<tr>
<td>3. The thickness of the casing shall not be less than manufacturer's standard gage No.14 (0.068 inch) (1.75 mm).</td>
<td>13.4.2.2.1 The allowable axial capacity for permanently cased cast-in-place concrete deep foundation members that satisfy (a) through (f) shall be permitted to be increased to the value given in Table 13.4.2.2:</td>
</tr>
<tr>
<td>4. The casing shall be seamless or provided with seams of strength equal to the basic material and be of a configuration that will provide confinement to the cast-in-place concrete.</td>
<td>(a) The design shall not use the casing to resist any portion of the axial load imposed.</td>
</tr>
<tr>
<td>5. The ratio of steel yield strength ($F_y$) to specified compressive strength ($f' c$) shall be not less than six.</td>
<td>(b) The casing shall have a sealed tip and be mandrel-driven.</td>
</tr>
<tr>
<td>6. The nominal diameter of the element shall not be greater than 16 inches (406 mm).</td>
<td>(c) The thickness of the casing shall not be less than manufacturer's standard gage No.14 (0.068 inch).</td>
</tr>
</tbody>
</table>

(f) The nominal diameter of the element shall be not greater than 16-in.

**Table 2**

**Comparison of 2018 IBC TABLE 1810.3.2.6 and ACI 13.4.2.2**
ALLOWABLE STRESSES FOR MATERIALS USED IN DEEP FOUNDATION ELEMENTS

<table>
<thead>
<tr>
<th>Material Type and Condition</th>
<th>IBC 2018 Maximum allowable Stress</th>
<th>ACI 318 MAXIMUM CAPACITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Concrete or grout in compression</td>
<td>0.4 ( f'_c )</td>
<td>( P_n = 0.4f'_c A_g )</td>
</tr>
<tr>
<td>Cast-in-place with a permanent casing in accordance with Section 1810.3.2.7</td>
<td>0.33 ( f'_c )</td>
<td>( P_n = 0.33f'_c A_f + 0.4f A_s )</td>
</tr>
<tr>
<td>Cast-in-place in a pipe, tube, other permanent casing or rock</td>
<td>0.3 ( f'_c )</td>
<td>( P_n = 0.3f'_c A_g + 0.4f A_s )</td>
</tr>
<tr>
<td>Cast-in-place without a permanent casing</td>
<td>0.33 ( f'_c )</td>
<td>( P_n = 0.33f'_c A_g + 0.4f A_s )</td>
</tr>
<tr>
<td>Precast nonprestressed</td>
<td>0.33 ( f'<em>c ) - 0.27 ( f</em>{pc} )</td>
<td>( P_n = (0.33f'<em>c - 0.27f</em>{pc})A_g )</td>
</tr>
</tbody>
</table>

The reference to ACI 318 is not a new concept for obtaining information for deep foundations. The IBC currently refers to the American Wood Council for provisions for deep timber foundations.

ACI, a professional technical society, supports these revisions to better align the IBC with current design and construction methodologies addressed in ACI 318 and to better communicate to the user that there additional methods that could result in lower initial costs and for conditions not addressed in 2018 IBC. ACI recommends that the committee approve this code change as submitted.

**Cost Impact:** The code change proposal will not increase or decrease the cost of construction
There is no increase in cost of construction. This proposal aligns the IBC with the methods used for concrete design and construction in accordance with ACI 318.
S129-19

IBC®: 1810.3.3.1

**Proponent:** 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:

**1810.3.3.1 Allowable axial load.** The allowable axial load on a deep foundation element shall be determined in accordance with Sections 1810.3.3.1.1 through 1810.3.3.1.9.

**Exception:** Load testing is not required where approved by the building official.

**Reason:** The load test waiver provision is at the discretion of the building official and was added to cover the case where applicable local knowledge and experience exist, such as nearby load test data on similar piles in similar subsurface conditions.

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**Cost Impact:** The code change proposal will decrease the cost of construction.

It will decrease the cost of construction if the load test can be waived based on previous experience.
S130-19

IBC®: 1810.3.3.1.9

Proponent: 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:

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

\[
P_a = 0.5 P_u
\]

(Equation 18-4)

where \( P_u \) is the least value of:

1. Base capacity plus shaft resistance of the helical pile. The base capacity is equal to the sum \( \sum \) of the areas of the helical bearing plates times the ultimate bearing capacity of the soil or rock comprising the bearing stratum. The shaft resistance only above the uppermost helical bearing plate shall be considered.

2. Ultimate capacity determined from well-documented correlations with installation torque.

3. Ultimate capacity determined from load tests when required by Section 1810.3.3.1.2.

4. Ultimate axial capacity of pile shaft.

5. Ultimate axial capacity of pile shaft couplings.

6. Sum of the ultimate axial capacity of helical bearing plates affixed to pile.

Reason: Value 1: Larger helical pile elements are now common and shaft friction can play an important role for larger shaft diameters. This addition allows for shaft resistance to be taken into account. “Shaft resistance” is the term used to be consistent with Section 1810.3.3.1.4.

Value 3: This item has been misinterpreted to always require load tests. Load testing is costly for small residential projects where helical piles are often used. The requirement for load testing of all piles is covered in Section 1810.3.3.1.2

Cost Impact: The code change proposal will decrease the cost of construction. This change will reduce the cost of construction for those helical piles where shaft friction can be taken into account and for those situations which do not require a load test.
2018 International Building Code

Revise as follows:

1810.3.4 Subsiding soils or strata. Where deep foundation elements are installed through subsiding soils or other subsiding strata and derive support from underlying firmer materials, consideration shall be given to the downward frictional forces potentially imposed on the elements by the subsiding upper strata. Where the influence of subsiding soils or strata is considered as imposing loads on the element, the allowable stresses specified in this chapter shall be permitted to be increased where satisfactory substantiating data are submitted.

Reason:
· The title of the current Section is “Subsiding soils” – changes are made in the text to match the title.
· Subsiding material includes more than just fill – the change to “soils” includes native soils and fills. Other subsiding strata include manmade material that can subside.
· The title is changed to reflect the existing text which mentions “strata”.

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

The change has no cost impact because it is only a clarification of the code.
S132-19

IBC®: 1810.3.5.3.1, 1810.3.11.2

Proponent: 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 $\frac{3}{8}$ 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 unlined 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."


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.
S133-19
IBC: 1810.3.6

Proponent: 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 and 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.

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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.
2018 International Building Code

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 of ACI 318.

Exceptions:

1. Or precast prestressed piles in Seismic Design Category C, the minimum spiral reinforcement required by Section 18.13.5.10.4 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 or Section 2.4.5 and the applicable overstrength factor, Ω. In such cases, minimum spiral reinforcement index shall be as specified in Section 13.4.5.6 of ACI 318.

2. For precast prestressed piles in Seismic Design Categories D through F, the minimum spiral reinforcement required by Section 18.13.5.10.5(c) 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 or Section 2.4.5 and the applicable overstrength factor, Ω. In such cases, minimum spiral reinforcement 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 0.259 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:

\[
\rho_s = 0.060 \frac{f'c}{f_y} \left( \frac{s}{A_p} \right) \]

(Equation 18-5)

where:

- \( A_p \) = Pile cross-sectional area, square inches (mm²).
- \( f'c \) = Specified compressive strength of concrete, psi (MPa).
- \( f_y \) = 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.
- \( \rho_s \) = 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, \( \Omega \). 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:

\[
\rho_s = 0.060 \frac{f'c}{f_y} \left( \frac{s}{A_p} \right) \]

(Equation 18-6)

but not exceed:

\[
\rho_s \leq 0.10 \]

(Equation 18-7)

where:

- \( A_p \) = Pile cross-sectional area, square inches (mm²).
- \( f'c \) = Specified compressive strength of concrete, psi (MPa).
- \( f_y \) = 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.
- \( \rho_s \) = 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, \( \Omega \). 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, \( A_p \), shall conform to:
but not less than:

\[ A_d = \frac{0.122h_c}{s} \left( 1 - 1.4\left( \frac{f'}{f_{yc}} \right)^2 \right) \]

where:

- \( f' \) = yield strength of transverse reinforcement ≤ 70,000 psi (483 MPa).
- \( h_c \) = Cross-sectional dimension of pile core measured center to center of hoop reinforcement, inch (mm).
- \( s \) = Spacing of transverse reinforcement measured along length of pile, inch (mm).
- \( A_w \) = Cross-sectional area of transverse reinforcement, square inches (mm²).
- \( f_{yc} \) = 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. \( 0.2f' A' \) for square piles
2. \( 0.4f' A' \) for circular or octagonal piles

Reason: Section 1810.3.8 of the IBC, along with its subsections, is mostly being deleted because similar provisions have been approved for inclusion in the 2019 edition of ACI 318. Deletion of these provisions from the IBC is necessary to eliminate any potential conflict between the 2021 IBC and ACI 318-19.

Two exceptions for precast prestressed piles are currently present in Sections 1810.3.8.3.2 (for SDC C) and 1810.3.8.3.3 Item 5 (for SDCs D through F). These are being retained because corresponding provisions have not been added to ACI 318-19. The exception statements are similar to other overstrength statements in this code and referenced load and material standards. The exceptions recognize that the volumetric ratio of spiral reinforcement need not be greater than that required for driving and handling stresses when the pile foundation system is designed for load combinations including overstrength. The minimum spiral reinforcement required per Section 13.4.5.6 of ACI 318-19 for driving and handling stresses is the minimum spiral reinforcement required for Seismic Design Categories A and B. In summary, when design includes the effect of overstrength, the increased axial forces, shear forces, and bending moments in the pile provide a large factor of safety against nonlinear pile behavior.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This code change proposal will not affect the cost of construction since it is removing duplicate provisions that already appear in a referenced standard.
2018 International Building Code

Revise as follows:

1810.3.11 Pile caps. Pile caps shall conform with ACI 318 and this section. Pile caps shall be of reinforced concrete, and shall include all elements to which vertical deep foundation elements are connected, including grade beams and mats. The soil immediately below the pile cap shall not be considered as carrying any vertical load, with the exception of a combined pile raft. The tops of vertical deep foundation elements shall be embedded not less than 3 inches (76 mm) into pile caps and the caps shall extend not less than 4 inches (102 mm) beyond the edges of the elements. The tops of elements shall be cut or chipped back to sound material before capping.

Delete without substitution:

1810.3.11.1 Seismic Design Categories C through F. For structures assigned to Seismic Design Category C, D, E, or F, concrete deep foundation elements shall be connected to the pile cap by embedding the element reinforcement or field placed dowels anchored in the element into the pile cap for a distance equal to their development length in accordance with ACI 318. It shall be permitted to connect precast prestressed piles to the pile cap by developing the element prestressing strands into the pile cap provided that the connection is ductile. For deformed bars, the development length is the full development length for compression, or tension in the case of uplift, without reduction for excess reinforcement in accordance with Section 25.4.10 of ACI 318. Alternative measures for laterally confining concrete and maintaining toughness and ductile-like behavior at the top of the element shall be permitted provided that the design is such that any hinging occurs in the confined region. The minimum transverse steel ratio for confinement shall be not less than one half of that required for columns.

For resistance to uplift forces, anchorage of steel pipes, tubes or H-piles to the pile cap shall be made by means other than concrete bond to the bare steel section. Concrete-filled steel pipes or tubes shall have reinforcement of not less than 0.01 times the cross-sectional area of the concrete fill developed into the cap and extending into the fill a length equal to two times the required cap embedment, but not less than the development length in tension of the reinforcement.

Revise as follows:

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.

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: 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 ACI, a 501(c)3 professional technical society, recommends approval as submitted to help avoid confusion and potential conflicts where similar requirements exist in both the IBC and ACI 318.
**1810.3.11.1** Seismic Design Categories C through F. For structures assigned to Seismic Design Category C, D, E or F, concrete deep foundation elements shall be reinforcement or field-placed dowels anchored in the element into the pile cap for a distance equal to their development length in accordance with ACI 318. It shall be permitted to connect precast prestressed piles to the pile cap by developing the element prestressing strands into the pile cap provided that the connection is ductile. For deformed bars, the development length is the full development length for compression, or tension in the case of uplift, without reduction for excess reinforcement in accordance with Section 25.4.10 of ACI 318. Alternative measures for laterally confining concrete and maintaining toughness and ductile-like behavior at the top of the element shall be permitted provided that the design is such that any hinging occurs in the confined region.

The minimum transverse steel ratio for confinement shall be not less than one-half of that required for columns. For resistance to uplift forces, anchorage of steel pipes, tubes or H-piles to the pile cap shall be made by means other than concrete bond to the bare steel section. Concrete-filled steel pipes or tubes shall have reinforcement of not less than 0.01 times the cross-sectional area of the concrete fill developed into the cap and extending into the fill a length equal to two times the required cap embedment, but not less than the development length in tension of the reinforcement.

**18.13.6** Anchorage of piles, piers, and caissons

**18.13.6.1** For structures assigned to SDC C, D, E, or F, the longitudinal reinforcement in piles, piers, or caissons resisting tension loads shall be detailed to transfer tension forces within the pile cap to supported structural members.

**18.13.6.2** For structures assigned to SDC C, D, E, or F, concrete piles and concrete filled pipe piles shall be connected to the pile cap by embedding the pile reinforcement in the pile cap a distance equal to the development length or by the use of field-placed dowels anchored in the concrete pile. For deformed bars, the compression development length is used if the pile is in compression. In the case of uplift, the tension development length is used without reduction in length for excess reinforcement.

**18.13.6.3** For structures assigned to SDC D, E, or F, if tension forces induced by earthquake effects are transferred between pile cap or mat foundation and precast pile by reinforcing bars grouted or post-installed in the top of the pile, the grouting system shall have been demonstrated by testing to develop at least 1.25f of the bar.

**Cost Impact:** The code change proposal will not increase or decrease the cost of construction. The is not cost increase or decrease, as the technical requirements remain essential unchanged. The proposal eliminates content from the IBC that is addressed in ACI 318.
Proponent: Stephen Szoke, representing American Concrete Institute (steve.szoke@concrete.org)

2018 International Building Code

Revise as follows:

1810.3.12 Grade beams. For structures assigned to Seismic Design Category D, E or F, grade beams shall comply with the provisions in Section 18.13.3 of ACI 318 for grade beams, except where they are.

   Exception: Grade beams designed to resist the seismic load effects including overstrength factor in accordance with Section 2.3.6 or 2.4.5 of ASCE 7.

1810.3.13 Seismic ties. For structures assigned to Seismic Design Category C, D, E or F, individual deep foundations shall be interconnected by ties. Unless it can be demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils or very dense granular soils, ties shall be capable of carrying, in tension or compression, a force equal to the lesser of the product of the larger pile cap or column design gravity load times the seismic coefficient, SDS, divided by 10, and 25 percent of the smaller pile or column design gravity load. Seismic ties shall comply with the provisions of ACI 318.

   Exception: In Group R-3 and U occupancies of light-frame construction, deep foundation elements supporting foundation walls, isolated interior posts detailed so the element is not subject to lateral loads or exterior decks and patios are not subject to interconnection where the soils are of adequate stiffness, subject to the approval of the building official.

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 1810.3.12→18.13.3
- Section 1810.3.13→18.13.4

<table>
<thead>
<tr>
<th>IBC 2018</th>
<th>ACI 318</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1810.3.12</strong> Grade beams. For structures assigned to Seismic Design Category D, E or F, grade beams shall comply with the provisions in Section 18.13.3 of ACI 318 for grade beams, except where they are designed to resist the seismic load effects including overstrength factor in accordance with Section 2.3.6 or 2.4.5 of ASCE 7.</td>
<td><strong>18.13.3</strong> Grade beams and slabs-on-ground</td>
</tr>
<tr>
<td><strong>18.13.3.1</strong> For structures assigned to SDC D, E, or F, grade beams and beams that are part of a mat foundation subjected to flexure from columns that are part of the seismic-force-resisting system shall be in accordance with 18.6.</td>
<td><strong>18.13.3.1</strong> For structures assigned to SDC D, E, or F, grade beams and beams that are part of a mat foundation subjected to flexure from columns that are part of the seismic-force-resisting system shall be in accordance with 18.6.</td>
</tr>
<tr>
<td><strong>18.13.3.2</strong> For structures assigned to SDC C, D, E, or F, slabs-on-ground that resist in-plane earthquake forces from walls or columns that are part of the seismic-force-resisting system shall be designed as diaphragms in accordance with 18.12. The construction documents shall clearly indicate that the slab-on-ground is a structural diaphragm and part of the seismic-force-resisting system.</td>
<td><strong>18.13.3.2</strong> For structures assigned to SDC C, D, E, or F, slabs-on-ground that resist in-plane earthquake forces from walls or columns that are part of the seismic-force-resisting system shall be designed as diaphragms in accordance with 18.12. The construction documents shall clearly indicate that the slab-on-ground is a structural diaphragm and part of the seismic-force-resisting system.</td>
</tr>
</tbody>
</table>

| **1810.3.13** Seismic ties. For structures assigned to Seismic Design Category C, D, E or F, individual deep foundations shall be interconnected by ties. Unless it can be demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils or very dense | **18.13.4** Foundation Seismic Ties |
| **18.13.4.1** For structures assigned to SDC C, D, E, or F, individual pile caps, piers, or caissons shall be interconnected by foundation seismic ties in orthogonal directions, unless it can be demonstrated that equivalent restraint is provided | **18.13.4.1** For structures assigned to SDC C, D, E, or F, individual pile caps, piers, or caissons shall be interconnected by foundation seismic ties in orthogonal directions, unless it can be demonstrated that equivalent restraint is provided |
granular soils, ties shall be capable of carrying, in tension or compression, a force equal to the lesser of the product of the larger pile cap or column design gravity load times the seismic coefficient, SDS, divided by 10, and 25 percent of the smaller pile or column design gravity load.

Exception: In Group R-3 and U occupancies of lightframe construction, deep foundation elements supporting foundation walls, isolated interior posts detailed so the element is not subject to lateral loads or exterior decks and patios are not subject to interconnection where the soils are of adequate stiffness, subject to the approval of the building official.

18.13.4.2 For structures assigned to SDC D, E, or F, individual spread footings founded on soil defined in ASCE 7 as Site Class E or F shall be interconnected by foundation seismic ties.

18.13.4.3 Where required, foundation seismic ties shall have a design strength in tension and compression at least equal to $0.1S \times D$ times the greater of the pile cap or column factored dead load plus factored live load unless it is demonstrated that equivalent restraint will be provided by (a), (b), (c), or (d):

(a) reinforced concrete beams within the slab-on-ground
(b) reinforced concrete slabs-on-ground
(c) confinement by competent rock, hard cohesive soils, or very dense granular soils
(d) other means approved by the building official.

18.13.4.4 For structures assigned to SDC D, E, or F, grade beams designed to act as horizontal foundation seismic ties between pile caps or footings shall have continuous longitudinal reinforcement that shall be developed within or beyond the supported column or anchored within the pile cap or footing at all discontinuities and shall satisfy (a) and (b):

(a) The smallest cross-sectional dimension of the grade beam shall be at least equal to the clear spacing between connected columns divided by 20, but need not exceed 18 in.
(b) Closed tie transverse reinforcement shall be provided at a spacing not to exceed the lesser of 0.5 times the smallest orthogonal cross-sectional dimension and 12 in.

ACI, a 501(c)3 professional technical society recommends approval as submitted to avoid confusion by eliminating criteria addressed in ACI 318 from the IBC. ACI, a 501(c)3 professional technical society recommends approval as submitted to avoid confusion by eliminating criteria addressed in ACI 318 from the IBC.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. There is no increase or decrease in cost of construction. Provisions of ACI 318 are applicable and code change proposal removes requirements from IBC that are also addressed in ACI 318.
S137-19

IBC®: 1810.4.1.2

Proponent: Daniel Stevenson, P.E., representing GeoCoalition (dstevenson@berkelapg.com); Dale Biggers, P.E. GeoCoalition, representing GeoCoalition (dbiggers@bohbros.com); Lori Simpson, representing GeoCoalition (lsimpson@langan.com)

2018 International Building Code

Revise as follows:

1810.4.1.2 Casing. Shafts in unstable soils. Where cast-in-place deep foundation elements are formed through unstable soils and concrete is placed in an open-drilled hole, a casing shall be inserted in the hole, the open hole shall be stabilized by a casing, suitable slurry, or other approved method prior to placing the concrete. Where the casing is withdrawn during concreting, the level of concrete shall be maintained above the bottom of the casing at a sufficient height to offset any hydrostatic or lateral soil pressure. Driven casings shall be mandrel driven their full length in contact with the surrounding soil.

Reason: 1. There are other commonly used means of stabilizing unstable soils besides casing, such as the use of drilling slurry.
2. Because there are means other than casings to stabilize the hole, the title is changed to “Shafts in unstable soils” because the focus of the section is stabilizing shafts in unstable soils.
3. The current code version says in the first sentence “concrete is placed in an open hole”. Those words are redundant, and thus removed, because the sentence begins with the context of a cast-in-place element. The “open hole…prior to placing the concrete” wording is maintained in the revision.

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
This change can lower construction cost by allowing other commonly used stabilization methods.

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

Proposal # 4561
S138-19

IBC®: 1810.4.1.3

Proponent: 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. 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.

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.

Proposal # 4584

Click here to see the members of the GeoCoalition: [http://www.piledrivers.org/2019-geocoalition-members/](http://www.piledrivers.org/2019-geocoalition-members/)
S139-19

IBC®: 1810.4.1.3

Proponent: 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 the concrete surface in any completed element rises or drops, the element shall be replaced. Driven uncased deep foundation elements shall not be installed in soils that could cause heave.

Reason: The deleted sentence has noting to do with the subject of this code section, the title of which is "Driving near uncased concrete." Prior to 2009, this sentence was contained in a code section that pertained only to the installation of driven uncased piles (e.g. 2006 IBC 1810.4.3 Installation under 1810.4 Driven uncased piles). However there is no longer a code section which specifically addresses the installation of driven uncased piles.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. It only removes antiquated language.
IBC®: 1810.4.5

Proponent: Dale Biggers, P.E. GeoCoalition, representing GeoCoalition (dbiggers@bohbros.com); Daniel Stevenson, P.E., representing GeoCoalition (dстevenson@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.
S141-19

IBC®: 1810.4.11

Proponent: 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:

1810.4.11 Helical piles. Helical piles shall be installed to specified embedment depth and torsional resistance criteria as determined by a registered design professional. The torque applied during installation shall not exceed the manufacturer's rated maximum allowable installation torque resistance of the helical pile.

Reason: The term “manufacturer’s rated maximum installation torque resistance” is consistent with the language that appears in many evaluation reports published by ICC-ES, which reflect the testing per acceptance criteria AC358 of ICC-ES. The term “maximum allowable” has created confusion and is not defined.

Click here to see the members of the GeoCoalition: http://www.piledrivers.org/2019-geocoalition-members/

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

The code change proposal will not increase the cost of construction since it is a clarification to the code

Proposal # 5145
Delete without substitution:

[BS] DETAILED PLAIN CONCRETE STRUCTURAL WALL. See Section 1905.1.1.

[BS] ORDINARY PRECAST STRUCTURAL WALL. See Section 1905.1.1.

[BS] ORDINARY REINFORCED CONCRETE STRUCTURAL WALL. See Section 1905.1.1.

[BS] ORDINARY STRUCTURAL PLAIN CONCRETE WALL. See Section 1905.1.1.

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.

1905.1.1 ACI 318, Section 2.3. Modify existing definitions and add the following definitions to ACI 318, Section 2.3.

DESIGN DISPLACEMENT. Total lateral displacement expected for the design-basis earthquake, as specified by Section 12.8.6 of ASCE 7.

SPECIAL STRUCTURAL WALL. A cast-in-place or precast wall complying with the requirements of 18.2.4 through 18.2.8, 18.10 and 18.11, as applicable, in addition to the requirements for ordinary reinforced concrete structural walls or ordinary precast structural walls, as applicable. Where ASCE 7 refers to a “special reinforced concrete structural wall,” it shall be deemed to mean a “special structural wall.”

Delete without substitution:

DETAILED PLAIN CONCRETE STRUCTURAL WALL. A wall complying with the requirements of Chapter 14, including 14.6.2.

ORDINARY PRECAST STRUCTURAL WALL. A precast wall complying with the requirements of Chapters 1 through 13, 15, 16 and 19 through 26.

ORDINARY REINFORCED CONCRETE STRUCTURAL WALL. A cast-in-place wall complying with the requirements of Chapters 1 through 13, 15, 16 and 19 through 26.

ORDINARY STRUCTURAL PLAIN CONCRETE WALL. A wall complying with the requirements of Chapter 14, excluding 14.6.2.

Revise as follows:

1905.1.6 1901.2.1 ACI 318, Section 14.6. Detailed plain concrete structural wall. Modify ACI 318, Section 14.6 by adding new Section 14.6.2 to read as follows:

- 14.6.2 — Detailed plain concrete structural walls:
  - 14.6.2.1 — Detailed plain concrete structural walls are walls conforming to the requirements of ordinary structural plain concrete walls and 14.6.2.2.
  - 14.6.2.2 — Reinforcement shall be provided as follows:
    Detailed plain concrete structural walls shall comply with the requirements of Chapter 14 of ACI 318 with reinforcement provided as follows:
    
    (a) Vertical reinforcement of at least 0.20 square inch (129 mm²) in cross-sectional area shall be provided continuously from support to support at each corner, at each side of each opening and at the ends of walls. The continuous vertical bar required beside an opening is permitted to substitute for one of the two No. 5 bars required by Section 14.6.1 of ACI 318.
    
    (b) Horizontal reinforcement at least 0.20 square inch (129 mm²) in cross-sectional area shall be provided:
1. Continuously at structurally connected roof and floor levels and at the top of walls.

2. At the bottom of load-bearing walls or in the top of foundations where doweled to the wall.

3. At a maximum spacing of 120 inches (3048 mm).

Reinforcement at the top and bottom of openings, where used in determining the maximum spacing specified in Item 3 above, shall be continuous in the wall.

**Reason:** There is no technical change to the requirements for design and construction of structural concrete. This change improves the clarity of the code and the coordination with ACI 318 by:

1) removing redundant language that advises concrete shall be in accordance with “this chapter” and “Section 1905” where the latter is part of the chapter.

2) relocating the provisions for detailed plain concrete structural wall from IBC Section 1905.1.6 to new Section 1901.2.1 for clarity and to facilitate use.

3) removing definition of ordinary precast concrete wall which has the same definition as ordinary reinforced concrete wall and not differentiated in ACI 318 or in the IBC.

4) removing definitions for other walls systems defined in ACI 318 and transcribed from ACI 318 simply to have all wall system definitions in one place when adding a definition for “detailed plain concrete structural wall.”

In addition, because the current language is presented as a modification, it portrays that criteria in ACI 318 for such walls are being modified. ACI Committee 318 does not recognize and thus does not address detailed plain concrete structural walls. This should not be presented as a modification to ACI 318, but included as specific language that permits detailed plain concrete structural walls in the IBC.

**Deletion of “as amended in Section 1905 of this code.”** In Section 1901.2 “as amended in Section 1905 of this code” is removed because it is redundant. The same sentence where this text occurs advises that “structural concrete be designed and constructed in accordance with the requirements of this chapter and ACI 318.” There is no need for the superfluous language: “as amended in Section 1905 of this code.” Language already advises that both ACI 318 and Chapter 19 shall be satisfied and IBC administration sections advise that Chapter 19 shall governor over ACI 318.

**Detailed plain concrete structural wall.** The current organization forces the user to flip back and forth between the IBC and ACI 318, only to discover the differences in the two are only applicable to “detailed plain concrete structural walls” which are not addressed in ACI 318. The new proposed language makes it clear to the use the provisions only where “detailed plain concrete structural walls” are being used in the project. This code change proposal removed redundant and superfluous language and clarifies the code. This change removes text transcribed from ACI simply to have all definitions in one place.

The technical requirements (text) is moved from Section 1905.1.6 new Section 1901.2.1 titled “detailed plain concrete structural wall” with minor editorial modifications necessary to accommodate the relocation.

**Ordinary precast concrete wall.** The definition for “ordinary precast structural wall” is deleted. The definition is identical to that for ordinary reinforced concrete structural wall and ACI does not differentiate between ordinary reinforced and precast structural walls. The language in the 2018 edition of the IBC creates confusion because the definition in the IBC encourages the user to seek specific language and criteria identified for ordinary precast concrete wall where it does not exist in ACI 318. ACI 318 is clear that both are addressed in ACI 318 Chapter 11. ACI 318 Section 11.1. Scope states:

“11.1.1 This chapter shall apply to the design of nonprestressed and prestressed walls including (a) through (c):

(a) Cast-in-place

(b) Precast in-plant

(c) Precast on-site including tilt-up”

**Removal of repetitive definitions in the IBC.** Several definitions in the 2018 IBC are similar to those in ACI 318. Continuation of carrying definitions in both documents requires additional coordination in Sections of ACI 318. The transcription of these definitions in the IBC was to have all structural wall definitions together when the IBC was simply adding criteria for “detailed plain concrete structural wall.” Having the new Section 1901.2.1 addresses “detailed plain concrete structural wall” and eliminates potential confusion on the definitions and applicable section of ACI 318. This will reduce the need for subsequent code change proposals simply to coordinate ACI 318 and the IBC. The definitions in both documents are
show in the table below.

**Removal of points called definitions in Section 202.** Pointers are removed from Section 202 for the definitions being removed from Section 1905.1.1

<table>
<thead>
<tr>
<th>2018 IBC 1905.1.1</th>
<th>ACI 318</th>
</tr>
</thead>
<tbody>
<tr>
<td>Definition for “structural wall: is absent from the 2018 IBC.</td>
<td><strong>structural wall</strong>—wall proportioned to resist combinations of shears, moments, and axial forces in the plane of the wall; a shear wall is a structural wall.</td>
</tr>
<tr>
<td>DETAILED PLAIN CONCRETE STRUCTURAL WALL. A wall complying with the requirements of Chapter 14, including 14.6.2.</td>
<td>Detailed plain concrete structural walls are not addressed in ACI 318. IBC text is moved from Section 1905.1.6 to new section 1901.2.1. Commentary to the IBC should be added to indicate that detailed plain concrete structural walls are not recognized by ACI Committee 318 and thus not addressed in ACI 318 Building Code Requirements for Structural Concrete.</td>
</tr>
<tr>
<td>ORDINARY PRECAST STRUCTURAL WALL. A precast wall complying with the requirements of Chapters 1 through 13, 15, 16 and 19 through 26.</td>
<td><strong>Ordinary precast structural wall.</strong> ACI 318 does not differentiate between ordinary precast structural walls and ordinary reinforced structural walls. Note that the definitions in the IBC are identical for both Ordinary Precast Structural Wall and Ordinary Reinforced Concrete Structural Wall.</td>
</tr>
<tr>
<td>ORDINARY REINFORCED CONCRETE STRUCTURAL WALL. A cast-in-place wall complying with the requirements of Chapters 1 through 13, 15, 16 and 19 through 26.</td>
<td>structural wall, ordinary reinforced concrete—a wall complying with Chapter 11.</td>
</tr>
<tr>
<td>ORDINARY STRUCTURAL PLAIN CONCRETE WALL. A wall complying with the requirements of Chapter 14, excluding 14.6.2.</td>
<td>Structural wall, ordinary plain concrete—a wall complying with Chapter 14.</td>
</tr>
<tr>
<td>SPECIAL STRUCTURAL WALL. A cast-in-place or precast wall complying with the requirements of 18.2.4 through 18.2.8, 18.10 and 18.11, as applicable, in addition to the requirements for ordinary reinforced concrete structural walls or ordinary precast structural walls, as applicable. Where ASCE 7 refers to a special reinforced concrete structural wall, it shall be deemed to mean a “special structural wall.”</td>
<td>Section 14.6.2 does not exist in ACI 318 and is a modification to ACI 318 in IBC Section 1905.1.6 which provides new Section 14.6.2. This is very confusing. Text from 1905.1.6 identifying and adding a non-existing Section 14.6.2 in ACI 318 is revised to eliminate reference to ACI 318 and moves to new Section 1901.2.1.</td>
</tr>
<tr>
<td>IBC has no provisions for intermediate precast walls.</td>
<td>structural wall, special—a cast-in-place structural wall in accordance with 18.2.3 through 18.2.8 and 18.10; or a precast structural wall in accordance with 18.2.3 through 18.2.8 and 18.11.</td>
</tr>
<tr>
<td></td>
<td>structural wall, intermediate precast—a wall complying with 18.5.</td>
</tr>
</tbody>
</table>

**ACI, a 501.C.3 professional technical society, encourages the approval of this code change proposal to improve the IBC by more clearly advising the user that there are provisions in the IBC for “detailed plain concrete structural walls” which are not addressed in ACI 318. The change removes transcription from ACI 318 and eliminates the need for frequent code change proposals to coordinate referenced ACI sections cited in the IBC.**

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

No change to technical requirements

Proposal #: 4444

S142-19
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 7-16.

Reason: This information has been coordinated between ASCE 7 and ACI 318. ACI 318-19 now contains duplicate provisions as Section 14.2.4 of ASCE 7-16. Future versions of ASCE 7 plan to remove the duplicate language. Therefore, ACI 318 should be the design reference for precast concrete diaphragms in buildings assigned to Seismic Design Category C, D, E or F.

ACI, a 501.C.3, professional technical society, encourages the approval of this code change proposal to improve the IBC by avoiding unnecessary duplication of text in code and referenced standards.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. Design and construction requirements are unaltered, change only removes text that is no longer necessary.
S144-19
IBC: 1901.2, SECTION 1907, 1907.1, 1907.2, 1907.3

Proponent: 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 1/4 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 m²) 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 1/4 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 m²) 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: 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 construction, change places slab related criteria in one section.
2018 International Building Code

Revise as follows:

1901.3 Anchoring to concrete. Anchoring to concrete shall be in accordance with ACI 318 as amended in Section 1905, and applies to cast-in (headed bolts, headed studs and hooked J- or L-bolts); post-installed expansion (torque-controlled and displacement-controlled); undercut and adhesive; and screw anchors.

Reason: This code change adds screws conforming to the requirements of ACI 318 as permissible anchoring devices. This makes the IBC more current and reflects technological advancements integrated into standardization. Further the use of screws adds flexibility for design and construction.

ACI, a 501.C.3 professional society encourages the approval of this code change proposal to improve the IBC by adding increased flexibility by adding screws as acceptable elements for anchoring to concrete.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. While there is no quantitative data, the addition of another method of anchorage improves flexibility in design and construction which may reduce initial cost. The addition of screws as a method for attachment will not increase cost.
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.

<table>
<thead>
<tr>
<th>IBC Requirements</th>
<th>ACI Requirements</th>
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<tbody>
<tr>
<td>Loads used in design</td>
<td>Compressive strength of concrete</td>
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<tr>
<td>Design work delegated to contractor</td>
<td>Compressive strength of concrete</td>
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<td>Aggregates</td>
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<td>Admixtures</td>
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<td>If water reducing – amount of modification</td>
<td>Exposure Class C – chloride ion limits</td>
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<td>If retarding – modification in setting time</td>
<td>Exposure Class S – types of cement</td>
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<tr>
<td>Where expansive cements are used – admixture compatibility</td>
<td>Density of lightweight aggregate</td>
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<td>Steel fiber reinforcement</td>
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<tr>
<td>Requirement</td>
<td>Specification</td>
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<td>----------------------------------------------------------------------------</td>
<td>-------------------------------------------------------------------------------</td>
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<tr>
<td>Volumetric fracture of aggregates where required in design</td>
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<td>Where used for shear – requirements for steel-fiber reinforced concrete</td>
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<td>Exposure class at option of engineer</td>
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<td>Compressive strength as various phases at option of engineer.</td>
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<td>Concrete mix proportions</td>
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<td>Material storage</td>
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<td>Concrete batching, mixing, and transport/delivery</td>
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<td>Pump pipe requirements</td>
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<td>Concrete placement</td>
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<td>Vertical lift requirements</td>
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<td>Field cured specimens if required</td>
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<td>Temperature of high early strength concrete</td>
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<td>Accelerated curing requirements if employed</td>
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<td>Protection and curing concrete</td>
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<td>Cold weather concrete procedures if applicable</td>
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<td>Hot weather concrete procedures if applicable</td>
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<tr>
<td>Locations where slab column interfaces are integrated</td>
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<td>Locations where steel-fiber reinforcement is required</td>
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<td>Saw cutting locations</td>
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<td>Strength or grade of reinforcement</td>
<td>Designation and grade of reinforcement</td>
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<td>Size and location of elements</td>
<td>Size and location of members</td>
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<td>Tolerance of members</td>
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<td>Size and location of reinforcement</td>
<td>Size and location of reinforcement</td>
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<td>Tolerances for reinforcement</td>
<td>Tolerances for reinforcement</td>
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<td>Designation of protective coatings</td>
<td>Designation of protective coatings</td>
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<td>Mill reports</td>
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<td>Field bending of reinforcement</td>
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<tr>
<td>Provisions for dimensional change</td>
<td>Provisions for dimensional change</td>
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<tr>
<td>Qualifications of anchors</td>
<td>Qualifications of anchors</td>
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<tr>
<td>Type, size, location requirements, effective embedment depth, and installation requirements for anchors</td>
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<tr>
<td>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</td>
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<tr>
<td>Qualifications for anchor installers</td>
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<tr>
<td>Corrosion protect for exposed anchors</td>
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<td>Type, size, details, and location of embedments</td>
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<tr>
<td>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.</td>
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<tr>
<td>Magnitude and location of prestressing forces</td>
<td>Magnitude and location of prestressing forces</td>
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<tr>
<td>Anchor and lap splice lengths</td>
<td>Anchor and lap splice lengths</td>
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<tr>
<td>Type and location of welded and mechanical splices</td>
<td>Type and location of welded and mechanical splices</td>
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<tr>
<td>Type and location of end-bearing splices</td>
<td></td>
</tr>
</tbody>
</table>
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.
2018 International Building Code

Add new text as follows:

1901.7 Tolerances for structural concrete. Where not indicated in construction documents, structural tolerances for concrete structural elements shall be in accordance with this section.

1901.7.1 Cast-in-place concrete tolerances. Structural tolerances for cast-in-place concrete structural elements shall be in accordance with ACI 117.

Exceptions:

1. Group R-3 detached one or two-family dwellings are not required to comply with this section.
2. Shotcrete is not required to comply with this section.

1901.7.2 Precast concrete tolerances. Structural tolerances for precast concrete structural elements shall be in accordance with ACI ITG-7.

Exception: Group R-3 detached one or two-family dwellings are not required to comply with this section.

ACI

117-10: Specification for Tolerances for Concrete Construction and Materials

ACI

ITG-7-09: Specification for Tolerances for Precast Concrete

Reason: ACI staff receive frequent technical inquiries regarding the allowable tolerances of structural concrete elements when tolerances are not indicated in construction documents. Adding these reference standards to the IBC provides the user with the information necessary for structural elements to perform as intended.

ACI, a 501(c)3 professional technical society, recommends approval as submitted to help assure that the appropriate tolerances for structural concrete elements applicable where not included in construction documents.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. There is no cost change associated with these requirements which are routinely cited in construction documents.

Staff Analysis: A review of the standard proposed for inclusion in the code, ACI 117-1 and ITG-7-09, 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.
SECTION 1902
definitions coordination of terms

1902.1 General. The words and terms defined in ACI 318 shall, for the purposes of this chapter and as used elsewhere in this code for concrete construction, have the meanings shown in ACI 318 as modified by Section 1905.1.1. Coordination of terminology used in ACI 318 and ASCE 7 shall be as follows:

Add new text as follows:

1902.1.1 Design displacement Design displacement shall be the total lateral displacement expected for the design-basis earthquake, as specified by Section 12.8.6 of ASCE 7.

1902.1.2 Special structural wall. Special structural walls made of cast-in-place or precast concrete shall comply with the requirements of Sections 18.2.4 through 18.2.8, 18.10 and 18.11 of ACI 318, as applicable, in addition to the requirements for ordinary reinforced concrete structural walls or ordinary precast structural walls, as applicable. Where ASCE 7 refers to a "special reinforced concrete shear wall," it shall be deemed to mean a "special structural wall."

Revise as follows:

1905.1.1 ACI 318, Section 2.3. Modify existing definitions and add the following definitions to ACI 318, Section 2.3.

DESIGN DISPLACEMENT. Total lateral displacement expected for the design-basis earthquake, as specified by Section 12.8.6 of ASCE 7.

SPECIAL STRUCTURAL WALL. A cast-in-place or precast wall complying with the requirements of Sections 18.2.4 through 18.2.8, 18.10 and 18.11 of ACI 318, as applicable, in addition to the requirements for ordinary reinforced concrete structural walls or ordinary precast structural walls, as applicable. Where ASCE 7 refers to a "special reinforced concrete structural wall," it shall be deemed to mean a "special structural wall."

(portions of section 1905.1.1 not shown are unchanged)

Reason: There is no change to the requirements for design and construction of structural concrete. This change improves the clarity of the code and the coordination with ACI 318 by:

1) removing redundant language that advises concrete shall be in accordance with "this chapter" and "Section 1905" where the latter is part of the chapter.

2) relocating the provisions for "design displacement" from IBC Section 1905.1.6 to new Section 1901.2.1 for clarity and to facilitate use. This appropriately removes criteria from a definition and places the criteria in a section.

3) relocating the provisions for "special structural wall" from IBC Section 1905.1.6 to new Section 1901.2.1 for clarity and to facilitate use. This appropriately removes criteria from a definition and places the criteria in a section. The definition and criteria for special structural wall in the IBC and ACI 318 are shown in Table 1

Deletion of “as amended in Section 1905 of this code.” In Section 1901.2 “as amended in Section 1905 of this code” is removed because it is redundant. The same sentence where this text occurs advises that “structural concrete be designed and constructed in accordance with the requirements of this chapter and ACI 318.” There is no need for the superfluous language: “as amended in Section 1905 of this code.”
already advises that both ACI 318 and Chapter 19 shall be satisfied and IBC administration sections advise that Chapter 19 shall govern over ACI 318.

**Removal of repetitive definitions in the IBC.** Several definitions in the 2018 IBC are similar to those in ACI 318. This will reduce the need for subsequent code change proposals simply to coordinate ACI 318 and the IBC.

**Elimination of pointers.** Unnecessary pointers in Section 202 Definitions are removed.

Table 1 - Comparison of IBC and ACI 318 definitions and criteria for special structural walls.

<table>
<thead>
<tr>
<th>2018 IBC 1905.1.1</th>
<th>ACI 318</th>
</tr>
</thead>
<tbody>
<tr>
<td>SPECIAL STRUCTURAL WALL. A cast-in-place or precast wall complying with the requirements of 18.2.4 through 18.2.8, 18.10 and 18.11, as applicable, in addition to the requirements for ordinary reinforced concrete structural walls or ordinary precast structural walls, as applicable. Where ASCE 7 refers to a special reinforced concrete structural wall, it shall be deemed to mean a “special structural wall.”</td>
<td>structural wall, special—a cast-in-place structural wall in accordance with 18.2.3 through 18.2.8 and 18.10; or a precast structural wall in accordance with 18.2.3 through 18.2.8 and 18.11.</td>
</tr>
<tr>
<td>IBC has no provisions for intermediate precast walls.</td>
<td>structural wall, intermediate precast—a wall complying with 18.5.</td>
</tr>
</tbody>
</table>

ACI, a 501.C.3 professional technical society, encourages the approval of this code change proposal to improve the IBC by more clearly advising the user that there are provisions in the IBC for “detailed plain concrete structural walls” which are not addressed in ACI 318. The change removes transcription from ACI 318 and eliminates the need for frequent code change proposals to coordinate referenced ACI sections cited in the IBC.

**Cost Impact:** The code change proposal will not increase or decrease the cost of construction. This change does not alter design or construction criteria but makes the code more user friendly, reducing the potential for errors or conflicts.
S149-19

IBC®: 1903.1

Proponent: 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:

1903.1 General. Materials used to produce concrete, concrete itself and testing thereof shall comply with the applicable standards listed in ACI 318.

Exception: The following standards as referenced in Chapter 35 shall be permitted to be used:

1. ASTM C150
2. ASTM C595
3. ASTM C1157

Reason: This language was introduced when there was concern that reference to the re-formatted edition of ACI 318-14 might not be approved for inclusion as a referenced standard in the 2015 edition of the International Building Code (IBC). The re-formatted edition of ACI 318 was included in the IBC and thus these cement standards, as referenced in ACI 318, are part of the IBC because language in Chapter 19 advises that:

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.

Further, ACI 318 permits other cementitious materials and the exception has implied to some users that these are the only cementitious materials permitted for concrete. All permissible cement standard specifications are listed in ACI 318:

C150/C150M-12—Standard Specification for Portland Cement
C595/C595M-14—Standard Specification for Blended Hydraulic Cements
C618-12a—Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete
C845/C845M-12—Standard Specification for Expansive Hydraulic Cement
C989/C989M-13—Standard Specification for Slag Cement for Use in Concrete and Mortars
C1240-14—Standard Specification for Silica Fume Used in Cementitious Mixtures

ACI, a 501.C.3 professional society, encourages the approval of this code change proposal as submitted to remove redundant and potentially misleading language.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. This code change proposal continues to permit the use of cement cited in the the 2018 edition of the IBC, and may reduce costs on specific projects by expanding the acceptable types of cement in accordance with those permitted in ACI 318.
SEISMIC DESIGN REQUIREMENTS

1905.1.1 Seismic design category A. Structures assigned to Seismic Design Category A shall not be required to satisfy the requirements of Chapter 18 of ACI 318.

1905.1.2 Seismic design categories B, C, D, E and F. Structures assigned to Seismic Design Category B, C, D, E or F shall satisfy 18.2.1.3 through 18.2.1.7 of ACI 318, as applicable.

1905.1.3 Structural plain concrete. Structural elements of plain concrete are prohibited in structures assigned to Seismic Design Category C, D, E or F.

14.1.4 — Structural elements of plain concrete are prohibited in structures assigned to Seismic Design Category C, D, E or F.
(b) Isolated footings of plain concrete supporting pedestals or columns are permitted, provided the projection of the footing beyond the face of the supported member does not exceed the footing thickness.

Exception: In detached one- and two-family dwellings three stories or less in height, the projection of the footing beyond the face of the supported member is permitted to exceed the footing thickness.

(c) Plain concrete footings supporting walls are permitted, provided the footings have at least two continuous longitudinal reinforcing bars. Bars shall not be smaller than No. 4 and shall have a total area of not less than 0.002 times the gross cross-sectional area of the footing. For footings that exceed 8 inches (203 mm) in thickness, a minimum of one bar shall be provided at the top and bottom of the footing. Continuity of reinforcement shall be provided at corners and intersections.

Exceptions:

1. In Seismic Design Categories A, B and C, detached one- and two-family dwellings three stories or less in height constructed with stud-bearing walls are permitted to have plain concrete footings without longitudinal reinforcement.
2. For foundation systems consisting of a plain concrete footing and a plain concrete stemwall, a minimum of one bar shall be provided at the top of the stemwall and at the bottom of the footing.
3. Where a slab on ground is cast monolithically with the footing, one No. 5 bar is permitted to be located at either the top of the slab or bottom of the footing.

Add new text as follows:

1905.1.4 Seismic force resisting system. Structural systems designated as part of the seismic force-resisting system shall be restricted to those permitted by ASCE 7. Except for Seismic Design Category A, for which Chapter 18 of ACI 318 does not apply, the following provisions shall be satisfied for each structural system designated as part of the seismic force-resisting system, regardless of the seismic design category:

1. Ordinary moment frames shall satisfy Section 18.3 of ACI 318.
2. Ordinary reinforced concrete structural walls and ordinary precast structural walls need not satisfy any provisions in Chapter 18 of ACI 318.
3. Intermediate moment frames shall satisfy Section 18.4 of ACI 318.
4. Intermediate precast structural walls shall satisfy Section 18.5 of ACI 318.
5. Special moment frames shall satisfy Sections 18.6 through 18.9 of ACI 318.
6. Special structural walls shall satisfy Section 18.10 of ACI 318.
7. Special structural walls constructed using precast concrete shall satisfy Section 18.11 of ACI 318.

1905.1.5 Special structural elements. Special moment frames and special structural walls shall also satisfy Sections 18.2.4 through 18.2.8 of ACI 318.

Revise as follows:

1905.1.6 ACI 318, Section 18.11. Precast special structural concrete walls. Modify ACI 318, Section 18.11.2.1 to read as follows:

1905.1.7 ACI 318, Section 18.13.1.1. Seismic force resisting foundations. Modify ACI 318, Section 18.13.1.1 to read as follows:

Foundations resisting earthquake-induced forces or transferring earthquake-induced forces between a structure and ground shall comply with the requirements of 18.13 and other applicable provisions of ACI 318 unless modified by Chapter 18 of the International Building Code.

Delete without substitution:

1905.1.3 ACI 318, Section 18.5. Modify ACI 318, Section 18.5 by adding new Section 18.5.2.2 and renumbering existing Sections 18.5.2.2 and 18.5.2.3 to become 18.5.2.3 and 18.5.2.4, respectively:

18.5.2.2 — Connections that are designed to yield shall be capable of maintaining 80 percent of their design strength at the deformation induced by the design displacement or shall use Type 2 mechanical splices.
18.5.2.3 — Elements of the connection that are not designed to yield shall develop at least 1.5 $S_d$.
18.5.2.4 — In structures assigned to SDC D, E or F, wall piers shall be designed in accordance with 18.10.8 or 18.14 in ACI 318.

Add new text as follows:

1905.1.8 Connections. Connections shall comply with Sections 1905.8.1 and 1905.8.2.

1905.1.8.1 Connections designed to yield. Connections that are designed to yield shall be capable of maintaining 80 percent of their design strength at the deformation induced by the design displacement or shall use Type 2 mechanical splices.
1905.1.8.2 Elements of connections not designed to yield. Elements of the connection that are not designed to yield shall develop at least $1.5 S_e$.

1905.1.9 Wall piers. In structures assigned to Seismic Design Category D, E or F, wall piers shall be designed in accordance with ACI 318 Section 18.10.8 or 18.14 of ACI 318.

Reason: These proposed revisions do not alter the criteria in the IBC, but instead make it clear to the user what systems and applications are being addressed. Rather than having the user read sections 14.1.4, 14.1.4.1 etc., the revisions clearly advise the user what is being addressed in the section. The user can more easily determine if the criteria are applicable to the project.

Modifications shown as new section 1905.1. These revisions make it clear to the user what systems and applications are being addressed. Rather than having the user read sections 14.1.4, 14.1.4.1 etc., the revisions clearly advise the user what is being addressed in the section. The user can more easily determine if the criteria are applicable to the project.

Modifications shown as new section 1905.1.2. Retains the criteria for structures assigned to SDC B, C, D, E, and F but clearly identifies that criteria as only being applicable to those SDCs. This change makes the code more user friendly, especially where used for projects in SDC A.

NOTE: Requirements of 2018 IBC 1905.1.2 are relocated to new Section 1905.1.4 Seismic force resisting system.

Modifications shown as new section 1905.1.3. This revision more clearly presents the requirements and

exceptions for plain concrete used in seismic design categories in an appropriately identified section. The criteria of section 1905.1.7 is moved to new section 1906.1.3. This also eliminates a pointer to other sections of the code.

NOTE: Requirements of 2018 IBC 1905.1.3 addressing connections are relocated to new Section 1905.1.8 Connections; and requirements in 2018 IBC 1905.1.3 addressing wall piers are relocated to new Section 1905.1.9 Wall piers.

Modifications shown as new section 1905.1.4. This revision identifies the section topic as “Seismic force resisting systems” in lieu of ACI 318 Section “18.2.1.6.” This provides clarity and improves direction to the user.

Modifications shown as new section 1905.1.5. This revision identifies the section topic as “Special structural elements.” and not as ACI 318, Section 18.11. This provides clarity and improves direction to the user.

Modification shown as new section 1905.1.6. This revision identifies the section as “Precast special structural concrete walls.” in lieu of 1905.1.4 ACI 318, Section 18.11. This provides clarity and direction to the user. This revision makes it clear that the user need not be concerned with these criteria where projects do not involve precast concrete. This modification also updates the referenced section to align with the provisions of ACI 318-19.

Modification shown as new section 1905.1.7. This revision identifies the section as “Seismic force-resisting foundations.” in lieu of “ACI 318, Section 18.13.1.1.” This provides clarity and direction to the user. This revision makes it clear that the user need not be concerned with these criteria where projects do not involve seismic force-resisting foundations.

Modification shown as new section 1905.1.8. This revision identifies the section as “Connections” in lieu of “ACI 318, Section 18.5.” This provides clarity and direction to the user. This revision makes it clear that the user need not be concerned with these criteria are applicable to connections and further clearly delineates between elements designed to yield and those not designed to yield.

Modification shown as new section 1905.1.9. This revision identifies the section as “Wall piers” in lieu of “ACI “18.5.2.4.” To the user it is unclear whether to search IBC or ACI 318 for Section 18.5.2.4. Further, if wall piers are not employed on the project, the user can easily identify that these requirements may not be applicable. Finally, the revised language makes it clearer that the cited references are both in ACI 318.

ACI, a 501.C.3 professional technical society, encourages the approval of this code change proposal to improve the IBC by more clearly advising the user that the provisions addressed in this section are related to seismic design category requirements, reduces transcription/duplication of language in IBC and ACI 318, reduces confusion by eliminating multiple indicators that are solely section numbers from ACI 318.

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

The requirements are not changed. Text is reorganized and presented in a more clear manner identifying specifically to what applications criteria pertain rather than just listing section numbers of ACI 318 which is very confusing.

This proposal could result in cost reductions by reducing confusion as presented in the 2018 IBC.

Proposal # 4532

S150-19
S151-19

IBC: 1905.1.8, 1906 (New), 1906.1 (New), 1906.1.2 (New), 1906.1.2.1 (New), 1906.1.2.2 (New), 1906.1.2.3 (New)

Proponent: 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

Delete without substitution:

1905.1.8

Modify ACI 318, Sections 17.2.3.4.2, 17.2.3.4.3(d) and 17.2.3.5.2 to read as follows:

17.2.3.4.2 – Where the tensile component of the strength-level earthquake force applied to anchors exceeds 20 percent of the total factored anchor tensile force associated with the same load combination, anchors and their attachments shall be designed in accordance with 17.2.3.4.3. The anchor design tensile strength shall be determined in accordance with 17.2.3.4.4.

Exception: Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 shall be deemed to satisfy Section 17.2.3.4.3(d).

17.2.3.4.3(d) – The anchor or group of anchors shall be designed for the maximum tension obtained from design load combinations that include $E$, with $E$ increased by $\Omega$. The anchor design tensile strength shall be calculated from 17.2.3.4.4.

17.2.3.5.2 – Where the shear component of the strength-level earthquake force applied to anchors exceeds 20 percent of the total factored anchor shear force associated with the same load combination, anchors and their attachments shall be designed in accordance with 17.2.3.5.3. The anchor design shear strength for resisting earthquake forces shall be determined in accordance with 17.5.

Exceptions:

1. For the calculation of the in-plane shear strength of anchor bolts attaching wood sill plates of bearing or nonbearing walls of light-frame wood structures to foundations or foundation stem walls, the in-plane shear strength in accordance with 17.5.2 and 17.5.3 need not be computed and 17.2.3.5.3 shall be deemed to be satisfied provided all of the following are met:
   1.1. The allowable in-plane shear strength of the anchor is determined in accordance with ANSI/AWC NDS Table 12E for lateral design values parallel to grain.
   1.2. The maximum anchor nominal diameter is 5/8 inch (16 mm).
   1.3. Anchor bolts are embedded into concrete a minimum of 7 inches (178 mm).
   1.4. Anchor bolts are located a minimum of 4 5/8 inches (46 mm) from the edge of the concrete parallel to the length of the wood sill plate.
   1.5. Anchor bolts are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the wood sill plate.
   1.6. The sill plate is 2-inch (51 mm) or 3-inch (76 mm) nominal thickness.

2. For the calculation of the in-plane shear strength of anchor bolts attaching cold-formed steel track of bearing or nonbearing walls of light-frame construction to foundations or foundation stem walls, the in-plane shear strength in accordance with 17.5.2 and 17.5.3 need not be computed and 17.2.3.5.3 shall be deemed to be satisfied provided all of the following are met:
   2.1. The maximum anchor nominal diameter is 5/8 inch (16 mm).
   2.2. Anchors are embedded into concrete a minimum of 7 inches (178 mm).
   2.3. Anchors are located a minimum of 4 5/8 inches (46 mm) from the edge of the concrete parallel to the length of the track.
   2.4. Anchors are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the track.
   2.5. The track is 33 to 68 mil (0.84 mm to 1.73 mm) designation thickness.

Allowable in-plane shear strength of exempt anchors, parallel to the edge of concrete, shall be permitted to be determined in accordance with AISI S100, Section E3.3.1.

In light-frame construction bearing or nonbearing walls, shear strength of concrete anchors less than or equal to 1 inch (25 mm) in diameter attaching sill plate or track to foundation or foundation stem wall need not satisfy 17.2.3.5.3(a) through (c) when the design strength of the anchors is determined in accordance with 17.5.2.1(c).

Add new text as follows:
SECTION 1906
ANCHORS TO CONCRETE

1906.1 General Anchors to concrete shall be designed and installed in accordance with Chapter 17 of ACI 318 and the provisions of this section.

1906.1.1 Anchors resisting out-of-plane forces. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 shall be deemed to satisfy Section 17.2.3.4.3(d) of ACI 318.

Add new text as follows:

1906.1.2 Anchorage of light frame walls to concrete. For the calculation of the in-plane shear strength of anchor bolts attaching wood sill plates or cold-formed steel track of bearing or nonbearing walls of light-frame structures to foundations or foundation stem walls, the in-plane shear strength in accordance with 17.5.2 and 17.5.3 of ACI 318 need not be computed and 17.2.3.5.3 of ACI 318 shall be deemed to be satisfied where the requirements of Sections 1906.1.2.1 and 1906.1.2.2 are met.

1906.1.2.1 Wood light frame walls. For anchor bolts attaching wood sill plates of light frame wood walls to foundations or foundation stem walls:
1. The allowable in-plane shear strength of the anchor is determined in accordance with ANSI/AWC NDS Table 12E for lateral design values parallel to grain.
2. Anchor bolts are embedded into concrete a minimum of 7 inches (178 mm).
3. Anchor bolts are located a minimum of 13/4 inches (45 mm) from the edge of the concrete parallel to the length of the wood sill plate.
4. Anchor bolts are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the wood sill plate.
5. The sill plate is 2-inch (51 mm) or 3-inch (76 mm) nominal thickness.

Add new text as follows:

1906.1.2.2 Cold-formed steel light frame walls. For anchor bolts attaching cold-formed steel track of light frame construction to foundations or foundation stem walls:
1. The maximum anchor nominal diameter is 5/8 inch (16 mm).
2. Anchors are embedded into concrete a minimum of 7 inches (178 mm).
3. Anchors are located a minimum of 13/4 inches (45 mm) from the edge of the concrete parallel to the length of the track.
4. Anchors are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the track.
5. The track is 33 to 68 mil (0.84 mm to 1.73 mm) designation thickness.

1906.1.2.3 Anchors 1 inch (25 mm) or less in diameter. In light-frame construction bearing or nonbearing walls, shear strength of concrete anchors less than or equal to 1 inch [25 mm] in diameter attaching sill plate or track to foundation or foundation stem wall need not satisfy 17.10.6.3(a) through (c) of ACI 318 when the design strength of the anchors is determined in accordance with 17.7.2.1(c) of ACI 318.

Reason: This code change proposal:
1) More clearly identifies the subject matter as anchors to concrete.
2) Removes duplicative text transcribed from ACI 318 to which the chapter already requires compliance. Deleted text from the IBC is shown with text in ACI 318 in the table below.
3) More clearly indicates where exceptions for light-frame anchorage to concrete are applicable and that the exceptions are only applicable to light-frame.
4) Moves anchor requirement in one section.

<table>
<thead>
<tr>
<th>2018 IBC</th>
<th>ACI 318</th>
</tr>
</thead>
<tbody>
<tr>
<td>17.2.3.4.2 – Where the tensile component of the strength-level earthquake force applied to anchors exceeds 20 percent of the total factored anchor tensile force associated with the same load combination, anchors and their attachments shall be designed in accordance with 17.2.3.4.3. The anchor design tensile strength shall be determined in accordance with 17.2.3.4.4.</td>
<td>17.2.3.4.2 Where the tensile component of the strength level earthquake force applied to anchors exceeds 20 percent of the total factored anchor tensile force associated with the same load combination, anchors and their attachments shall be designed in accordance with 17.2.3.4.3. The anchor design tensile strength shall be determined in accordance with 17.2.3.4.4. Comment: Criteria are the same and text is recommended for deletion from IBC. Further such a change avoids routine code change proposals to coordinate section of ACI 318 referenced in the IBC.</td>
</tr>
<tr>
<td>17.2.3.4.3(d) – The anchor or group of anchors shall be designed for the maximum tension obtained from design load combinations that include E</td>
<td>(d) The anchor or group of anchors shall be designed for the maximum tension obtained from design load combinations that include E, with the</td>
</tr>
</tbody>
</table>

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with $E$ increased by 0. The anchor design tensile strength shall be calculated from 17.2.3.4.4.

horizontal component of $E$ increased by $\Omega_0$. The anchor design tensile strength shall satisfy the tensile strength requirements of 17.2.3.4.4

Comment: Criteria are the same and text is recommended for deletion from IBC. Further such a change avoids routine code change proposals to coordinate section of ACI 318 referenced in the IBC.

17.2.3.5.2 – Where the shear component of the strength-level earthquake force applied to anchors exceeds 20 percent of the total factored anchor shear force associated with the same load combination, anchors and their attachments shall be designed in accordance with 17.2.3.5.3. The anchor design shear strength for resisting earthquake forces shall be determined in accordance with 17.5

17.2.3.5.2 Where the shear component of the strength level earthquake force applied to anchors exceeds 20 percent of the total factored anchor shear force associated with the same load combination, anchors and their attachments shall be designed in accordance with 17.2.3.5.3. The anchor design shear strength for resisting earthquake forces shall be determined in accordance with 17.5

Comment: Criteria are the same and text is recommended for deletion from IBC. Further such a change avoids routine code change proposals to coordinate section of ACI 318 referenced in the IBC.

This code change also removes unnecessary transcription from ACI 318 to further improve clarity.

ACI, a 501.C.3 professional technical society, encourages the approval of this code change proposal to improve the IBC by removing transcription from ACI 318, as the transcription makes the user think there is something different that must be addressed when the chapter already requires compliance with ACI 318. Further this proposed revision places anchor requirements in one section.

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

This code change does not alter any technical requirements, but make the code more user-friendly by clearly communicating where deviations from ACI 318 are permitted.
**S152-19**

**IBC®: SECTION 1906, 1906.1**

**Proponent:** 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:

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**SECTION 1906**

**STRUCTURAL PLAIN CONCRETE FOOTINGS FOR LIGHTFRAME CONSTRUCTION**

1906.1 **Scope.** Plain concrete footings. The design and construction of structural plain concrete, both cast in place and precast, shall comply with the minimum requirements of ACI 318, as modified in Section 1905. **Exception:** For Group R-3 occupancies and buildings of other occupancies less than two stories above grade plane of light-frame construction, the required footing thickness of ACI 318 plain concrete footings is permitted to be reduced to 6 inches (152 mm), provided that the footing does not extend more than 4 inches (102 mm) on either side of the supported wall.

**Reason:** This code change removes unnecessary text and clearly indicate to the user that the provisions of this sections are restricted to light-frame construction. Sections 1905 and 1901 already advise that structural plain concrete must follow the ACI 318 and the appropriate sections of the IBC. This redundant language is eliminated. Further text is editorially modified to alter language presented as an “exception” to be presented as an appropriate provision.

ACI, a 501.C.3 professional technical 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 footing supporting light-frame construction.

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

No change to criteria, improved language for clarify and direction.

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Proposal # 4537
**S153-19**

**IBC®: 1907.1, ASTM Chapter 35 (New)**

**Proponent:** 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; Valarie Evans

**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\(\frac{1}{2}\) inches (89 mm). A 6-mil-10-mil (0.006-0.010 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**

**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.
S154-19

IBC: 1907.2 (New)

Proponent: Amy Dowell, Post-Tensioning Institute, representing Post-Tensioning Institute (amy.dowell@post-tensioning.org); Stephen Szoke (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.
2018 International Building Code

SECTION 1908
SHOTCRETE

Revise as follows:

1908.1 General. Shotcrete is mortar or concrete that is pneumatically projected at high velocity onto a surface. Except as specified in this section, shotcrete shall conform to the requirements of this chapter for plain or reinforced concrete, shall be in accordance with the requirements of ACI 318.

Delete without substitution:

1908.2 Proportions and materials. Shotcrete proportions shall be selected that allow suitable placement procedures using the delivery equipment selected and shall result in finished in-place hardened shotcrete meeting the strength requirements of this code.

1908.3 Aggregate. Coarse aggregate, if used, shall not exceed 3/4 inch (19.1 mm).

1908.4 Reinforcement. Reinforcement used in shotcrete construction shall comply with the provisions of Sections 1908.4.1 through 1908.4.4.

1908.4.1 Size. The maximum size of reinforcement shall be No. 5 bars unless it is demonstrated by preconstruction tests that adequate encasement of larger bars will be achieved.

1908.4.2 Clearance. Where No. 5 or smaller bars are used, there shall be a minimum clearance between parallel reinforcement bars of 2 inches (64 mm). When bars larger than No. 5 are permitted, there shall be a minimum clearance between parallel bars equal to six diameters of the bars used. Where two curtains of steel are provided, the curtain nearer the nozzle shall have a minimum spacing equal to 12 bar diameters and the remaining curtain shall have a minimum spacing of six bar diameters.

Exception: Subject to the approval of the building official, required clearances shall be reduced where it is demonstrated by preconstruction tests that adequate encasement of the bars used in the design will be achieved.

1908.4.3 Splices. Lap splices of reinforcing bars shall utilize the noncontact lap splice method with a minimum clearance of 2 inches (51 mm) between bars. The use of contact lap splices necessary for support of the reinforcing is permitted where approved by the building official, based on satisfactory preconstruction tests that show that adequate encasement of the bars will be achieved, and provided that the splice is oriented so that a plane through the center of the spliced bars is perpendicular to the surface of the shotcrete.

1908.4.4 Spirally tied columns. Shotcrete shall not be applied to spirally tied columns.

1908.5 Preconstruction tests. Where preconstruction tests are required by Section 1908.4, a test panel shall be shot, cured, cored or sawn, examined and tested prior to commencement of the project. The sample panel shall be representative of the project and simulate job conditions as closely as possible. The panel thickness and reinforcing shall reproduce the thickest and most congested area specified in the structural design. It shall be shot at the same angle, using the same nozzleman and with the same concrete mix design that will be used on the project. The equipment used in preconstruction testing shall be the same equipment used in the work requiring such testing, unless substitute equipment is approved by the building official. Reports of preconstruction tests shall be submitted to the building official as specified in Section 1704.5.

1908.6 Rebound. Any rebound or accumulated loose aggregate shall be removed from the surfaces to be covered prior to placing the initial or any succeeding layers of shotcrete. Rebound shall not be used as aggregate.

1908.7 Joints. Except where permitted herein, unfinished work shall not be allowed to stand for more than 30 minutes unless edges are sloped to a thin edge. For structural elements that will be under compression and for construction joints shown on the approved construction documents, square joints are permitted. Before placing additional material adjacent to previously applied work, sloping and square edges shall be cleaned and wetted.

1908.8 Damage. In-place shotcrete that exhibits sag, sloughs, segregation, honeycombing, sand pockets or other obvious defects shall be removed and replaced. Shotcrete above sags and sloughs shall be removed and replaced while still plastic.

1908.9 Curing. During the curing periods specified herein, shotcrete shall be maintained above 40°F (4°C) and in moist condition.
1908.1 Initial curing. Shotcrete shall be kept continuously moist for 24 hours after shotcreting is complete or shall be sealed with an approved curing compound.

1908.2 Final curing. Final curing shall continue for seven days after shotcreting, or for three days if high-early-strength cement is used, or until the specified strength is obtained. Final curing shall consist of the initial curing process or the shotcrete shall be covered with an approved moisture-retaining cover.

1908.3 Natural curing. Natural curing shall not be used in lieu of that specified in this section unless the relative humidity remains at or above 85 percent, and is authorized by the registered design professional and approved by the building official.

1908.10 Strength tests. Strength tests for shotcrete shall be made by an approved agency on specimens that are representative of the work and that have been water soaked for not fewer than 24 hours prior to testing. Where the maximum size aggregate is larger than $\frac{3}{8}$ inch (9.5 mm), specimens shall consist of not less than three 3-inch-diameter (76 mm) cores or 3-inch (76 mm) cubes. Where the maximum size aggregate is $\frac{3}{8}$ inch (9.5 mm) or smaller, specimens shall consist of not less than 2-inch-diameter (51 mm) cores or 2-inch (51 mm) cubes.

1908.10.1 Sampling. Specimens shall be taken from the in-place work or from test panels, and shall be taken not less than once each shift, but not less than one for each 50 cubic yards (38.2 m$^3$) of shotcrete.

1908.10.2 Panel criteria. Where the maximum size aggregate is larger than $\frac{3}{8}$ inch (9.5 mm), the test panels shall have minimum dimensions of 18 inches by 18 inches (457 mm by 457 mm). Where the maximum size aggregate is $\frac{3}{8}$ inch (9.5 mm) or smaller, the test panels shall have minimum dimensions of 12 inches by 12 inches (305 mm by 305 mm). Panels shall be shot in the same position as the work, during the course of the work and by the nozzlemen doing the work. The conditions under which the panels are cured shall be the same as the work.

1908.10.3 Acceptance criteria. The average compressive strength of three cores from the in-place work or a single test panel shall equal or exceed 0.85 $f_{cmax}$ with no single core less than 0.75 $f_{cmax}$. The average compressive strength of three cubes taken from the in-place work or a single test panel shall equal or exceed $f_{cmax}$ with no individual cube less than 0.88 $f_{cmax}$. To check accuracy, locations represented by erratic core or cube strengths shall be retested.

Reason: The current criteria in the International Building Code (IBC) is based on American Concrete Institute (ACI) Guide to Fiber-Reinforced Shotcrete (ACI 506.1R). The guide was last updated in 2008 and much of the information in the current edition of the IBC is based on recommendations published in the 1998 edition of ACI 506.1R. The current criteria in the IBC is for the most part archaic and does not reflect shotcrete that is readily available today. Mandatory criteria for the design and construction of shotcrete is now integrated into ACI Building Code Requirements for Structural Concrete and Commentary (ACI 318). ACI 318 includes shotcrete along with plain and reinforced cast-in-place concrete and precast and prestressed concrete:

"4.2.1.1 Design properties of shotcrete shall conform to the requirements of concrete except as modified by specific provisions of the Code."

The provisions unique to shotcrete as shown below under “Shotcrete provisions included in ACI 318,” demonstrating a fully comprehensive effort by ACI Committee 318 to integrate shotcrete into ACI 318 Building Code Requirements for Structural Concrete. These provisions are in addition to all exiting applicable provisions of ACI 318. Among the significant differences between the current language in the 2018 edition of the IBC and ACI 318 are:

1) New durability requirements added to Chapter 19

2) Criteria that allow for additional spacings of reinforcement to improve economy added to Chapter 25.

3) Additional criteria for reinforcement and splices to better assure life safety and desired performance added to Chapter 25.

4) Criteria for inspection and quality assurance specific to shotcrete added to ACI 318 Chapter 26.

With ACI 318 being the premier document for design and construction of structural concrete, this inclusion elevates the overall acceptance of shotcrete thereby providing owners, developers, and designers with increased confidence when using shotcrete. This in turn allows owners, developers and designers to more readily use the most economical concrete solutions for their projects. Inclusion in ACI 318 also provides all relevant design and construction criteria in mandatory language required for design and construction of shotcrete to assure an acceptable level of life safety and performance while more appropriately addressing current industry practice.

Further ACI 318 is referenced as applicable to plain and reinforced concrete in Section 1901.2: “Structural concrete shall be designed and constructed in accordance with the requirements of this chapter and ACI 318...” Since shotcrete may be a type of structural concrete, the removal of the criteria in the IBC, in addition to updating the requirements to current technology and practice, will help remove confusion and eliminate errors.

This code change proposal:

1) Replaces general language in Section 1908.1 and simply directs the user to ACI 318.
2) Removes archaic criteria from the IBC in favor of current criteria applicable to current shotcrete products, design, construction, and inspection as addressed in ACI 318.

3) Removes pointers for inspection from Table 1705.3 Required Special Inspections and Tests of Concrete Construction, as these pointers are no longer required where compliance is in accordance with ACI 318.

4) Adds ACI 318 as a reference to Section 1908 of the IBC.

As a not-for-profit professional society, ACI recommends approval of this code change proposal as submitted to reflect current products and design and construction practices for the benefit of the public and to improve design and construction flexibility and lower costs.

Shotcrete provisions included in ACI 318

Chapter 2 – Notation and Terminology

panel, shotcrete mockup—a shotcrete specimen that simulates the size and detailing of reinforcement in a proposed structural member for preconstruction evaluation of the nozzle operator's ability to encase the reinforcement.

panel, shotcrete test—a shotcrete specimen prepared in accordance with ASTM C1140 for evaluation of shotcrete. shotcrete — mortar or concrete placed pneumatically by high velocity projection from a nozzle onto a surface.

shotcrete, dry mix—shotcrete in which most of the mixing water is added to the concrete ingredients at the nozzle.

shotcrete, wet mix—shotcrete in which the concrete ingredients, including water, are mixed before introduction into the delivery hose.

Chapter 4 – Structural System Requirements

4.2—Materials

4.2.1.1 Design properties of shotcrete shall conform to the requirements for concrete except as modified by specific provisions of the Code.

Chapter 19—Concrete: Design and Durability Requirements

19.3.3.3 Wet-mix shotcrete subject to freezing-and-thawing Exposure Classes F1, F2, or F3 shall be air entrained. Dry-mix shotcrete subject to freezing-and-thawing Exposure Class F3 shall be air entrained. Except as permitted in 19.3.3.6, air content shall conform to Table 19.3.3.3.

<table>
<thead>
<tr>
<th>Mixture Type</th>
<th>Target air content, percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wet-mix shotcrete, before placement</td>
<td>F1  5.0  6.0  6.0</td>
</tr>
<tr>
<td>Dry-mix shotcrete (in place)</td>
<td>N/A  N/A  4.5</td>
</tr>
</tbody>
</table>

19.3.3.4 Wet-mix shotcrete shall be sampled in accordance with ASTM C172, and air content shall be measured in accordance with ASTM C231 or ASTM C173.

19.3.3.5 Dry-mix shotcrete shall be sampled and air content shall be measured as directed by the licensed design professional.

19.3.3.6 For $f'_c$ exceeding 5000 psi, reduction of air content indicated in Tables 19.3.3.1 and 19.3.3.3 by 1.0 percentage point is permitted.

Chapter 25—Reinforcement Details

25.2—Minimum spacing of reinforcement

25.2.7 For parallel nonprestressed reinforcement in shotcrete members, the clear spacing shall be in accordance with (a) or (b):

(a) The clear spacing between bars shall be at least the greater of $6d_b$ and 2½ in.

(b) If two curtains of reinforcement are provided, the clear spacing between bars in the curtain nearer the nozzle shall be at least $12d_b$. The clear spacing between bars in the remaining curtain shall conform to (a).
25.2.7.1 It shall be permitted to use a clear spacing that does not meet 25.2.7(a) or 25.2.7(b) provided shotcrete mockup panels are used to 
demonstrate proper reinforcement encasement in accordance with (a) and (b):

(a) The shotcrete mockup panels shall be representative of the most complex reinforcement configurations to be encountered.

(b) The licensed design professional shall specify the shotcrete mock-up panel quantity, frequency of shooting per nozzleman and member type, 
and panel thickness to verify reinforcement encasement.

25.2.8 For prestressed strands in shotcrete members, minimum center-to-center spacing s shall satisfy 25.2.4, except as permitted in 25.2.6.

25.2.9 For prestressed wire in shotcrete members, minimum center-to-center spacing s shall satisfy the requirements for wire in 25.2.5, except as 
permitted in and 25.2.6.

25.2.10 For ties, hoops, and spiral reinforcement in columns to be placed with shotcrete, minimum clear spacing shall be 3 in.

25.2.10.1 It shall be permitted to use a clear spacing other than 3 in. provided shotcrete mockup panels are used to demonstrate proper 
encasement of the reinforcement in accordance with 25.2.7.1

25.5—Splices

25.5.1.6 Non-contact lap splices for reinforcement in shotcrete shall have clear spacing in accordance with (a) or (b):

(a) For No. 6 and smaller bars, the clear spacing between bars shall be at least the greater of 6dₚ and 2½ in.

(b) For No. 7 and larger bars, the clear spacing shall be established using a shotcrete mockup panel to demonstrate that the reinforcement is 
properly encased.

25.5.1.7 Contact lap splices for reinforcement in shotcrete shall be oriented with the plane of the spliced bars perpendicular to the surface of the 
shotcrete and approved by the licensed design professional based on a shotcrete mockup panel to demonstrate that the reinforcement is properly 
encased.

Chapter 26 – Construction Documents and Inspection

26.3—Member information

26.3.1 (b) Members to be constructed using shotcrete

26.3.2(a) Use of shotcrete for structural members not identified in the construction documents as required to be placed by shotcrete shall be 
permitted in accordance with the project contract documents.

26.4—Concrete materials and mixture requirements

26.4.1.2.1 Compliance requirements:

(e) For shotcrete, the aggregate gradation shall comply with ASTM C1436.

26.4.1.4 Admixtures

26.4.1.4.1 Compliance requirements:

(e) Admixtures used in shotcrete shall conform to ASTM C1141.

26.4.1.6 Packaged, pre-blended, dry, combined materials for shotcrete

26.4.1.6.1 Compliance requirements:

(a) Packaged, pre-blended, dry, combined materials for shotcrete shall conform to ASTM C1480.

26.4.2 Concrete mixture requirements

26.4.2.1 Design information:
For shotcrete, the nominal maximum size of coarse aggregate shall not exceed 1/2 in.

26.4.3 Proportioning of concrete mixtures

26.4.3.1 Compliance requirements:

(e) Shotcrete mixture proportions shall be established so that shotcrete satisfies (1) through (3):

(1) Can be placed without segregation and fully encase reinforcement.

(2) Meets durability requirements given in the construction documents.

(3) Conforms to strength test requirements for shotcrete.

26.4.4 Documentation of concrete mixture characteristics

26.4.4.1 Compliance requirements:

(d) Documentation of shotcrete mixture characteristics shall be submitted for review by the licensed design professional before the mixture is used and before making changes to mixtures already approved. Evidence of the ability of the proposed shotcrete mixture to comply with the concrete mixture requirements in the construction documents shall be included in the documentation.

26.5.2 Concrete placement and consolidation

26.5.2.1 Compliance requirements:

(j) Prior to placement of a new layer of shotcrete, rebound and overspray from adjacent placements shall be removed.

(k) Cuttings and rebound shall not be incorporated into the Work.

(l) Shotcrete surfaces intended to receive subsequent shotcrete placement shall be roughened to a full amplitude of approximately ¼-in. before the shotcrete has reached final set.

(m) Before placing additional material onto hardened shotcrete, laitance shall be removed, joints shall be cleaned, and the surface dampened.

(n) In-place fresh shotcrete that exhibits sags, sloughs, segregation, honeycombing, or sand pockets shall be removed and replaced.

(o) A certified shotcrete nozzle operator shall place all shotcrete.

(p) If a project-specific shotcrete mockup panel is required, each nozzle operator shall have demonstrated the ability to shoot an approved shotcrete mockup panel.

26.5.3 Curing

26.5.3.2 Compliance requirements:

(f) Shotcrete shall be cured in accordance with (1) through (3).

1. For 24 hours from completion of placement, initial curing shall be provided by one of the following methods:

i. Ponding, fogging, or continuous sprinkling;

ii. Absorptive mat, fabric, or other protective covering kept continuously moist;


2. After 24 hours from completion of placement, final curing shall be provided by one of the following methods:

i. Same method used in the initial curing process;

ii. Sheet materials;
iii. Other moisture-retaining covers kept continuously moist.

3. Final curing shall be maintained for a minimum duration of not less than the following:

i. 7 days,

ii. 3 days if high-early-strength cement or an accelerating admixture is used.

26.5.6 Construction, contraction, and isolation joints

26.5.6.1 Design information:

26.5.6.1(f) For shotcrete, location of construction joints for which square joints are permitted.

26.5.6.2 Compliance information:

26.5.6.2(g) For shotcrete, construction joint surfaces shall be cut at a 45-degree angle to the finished surface, unless a square joint is designated in the construction documents.

26.5.6.2(h) For shotcrete, construction joints proposed at locations not shown on the construction documents shall be submitted to the licensed design professional for approval prior to shotcrete placement.

26.12—Concrete evaluation and acceptance

26.12.1 General

26.12.1.1 Compliance requirements:

(b) For shotcrete, a strength test shall be the average strength of at least three 3-in. diameter cores taken from a test panel prepared in accordance with ASTM C1140 and tested at 28 days from time of placement or at test age designated for $f'_c$.

26.12.2 Frequency of testing

26.12.2.1 Compliance requirements:

(d) For shotcrete, prepare a shotcrete test panel for each mixture and each nozzle operator at least once per day or for every 50 yd$^3$ placed, whichever results in the greater number of panels.

26.12.4 Acceptance for shotcrete

26.12.4.1 Compliance requirements:

(a) Specimens for acceptance tests shall be in accordance with (1) and (2):

(1) Test panels shall be prepared in the same orientation and by the same nozzle operator placing shotcrete.

(2) Cores shall be obtained, conditioned, and tested in accordance with ASTM C1604.

(b) Strength level of a shotcrete mixture shall be acceptable if (1) and (2) are satisfied:

(1) Every arithmetic average of the strengths from three consecutive test panels equals or exceeds $f'_c$.

(2) The average compressive strength of three cores from a single test panel is not less than 0.85 $f'_c$ with no core having a strength less than 0.75 $f'_c$.

(c) If either of the requirements of 26.12.4.1(b) are not satisfied, steps shall be taken to increase the average of subsequent strength results.

(d) Requirements for investigating low strength-test results shall apply if the requirements of 26.12.4.1(b)(2) are not met.

Table 1 - Comparison of 2018 IBC and ACI 318 Provisions
2018 IBC Provisions

Chapter 2 – Notation and Terminology

**shotcrete** — mortar or concrete placed pneumatically by high velocity projection from a nozzle onto a surface.

**shotcrete, dry mix** — shotcrete in which most of the mixing water is added to the concrete ingredients at the nozzle.

**shotcrete, wet mix** — shotcrete in which the concrete ingredients, including water, are mixed before introduction into the delivery hose.

1908.2 Proportions and materials. Shotcrete proportions shall be selected that allow suitable placement procedures using the delivery equipment selected and shall result in finished in-place hardened shotcrete meeting the strength requirements of this code.

Chapter 4 – Structural System Requirements

4.2—Materials

4.2.1.1 Design properties of shotcrete shall conform to the requirements for concrete except as modified by specific provisions of the Code.

26.4.3 Proportioning of concrete mixtures

26.4.3.1 Compliance requirements:

(e) Shotcrete mixture proportions shall be established so that shotcrete satisfies (1) through (3):

(1) Can be placed without segregation and fully encase reinforcement.

(2) Meets durability requirements given in the construction documents.

(3) Conforms to strength test requirements for shotcrete.

26.4.4.1 Compliance requirements:

(d) Documentation of shotcrete mixture characteristics shall be submitted for review by the licensed design professional before the mixture is used and before making changes to mixtures already approved in use. Evidence of the ability of the proposed shotcrete mixture to comply with the concrete mixture requirements in the construction documents shall be included in the documentation.

Comment: ACI 318-19 addresses durability in addition to strength and placement.

1908.3 Aggregate. Coarse aggregate, if used, shall not exceed 3/4 inch (19.1 mm).

26.4.2 Concrete mixture requirements

26.4.2.1 Design information:

(a)(17) For shotcrete, the nominal maximum size of coarse aggregate shall not exceed ½ in.

Comment: The ACI 318-19 provisions more appropriately limit the maximum aggregate size for shotcrete to ½ inch in lieu of ¾ inch as allowed in the IBC.

1908.4 Reinforcement. Reinforcement used in shotcrete construction shall comply with the provisions of Sections 1908.4.1 through 1908.4.4.

1908.4.1 Size. The maximum size of reinforcement shall be No. 5 bars unless it is demonstrated by preconstruction tests that adequate encasement of larger bars will be achieved.

Chapter 25—Reinforcement Details

25.2—Minimum spacing of reinforcement

25.2.7 For parallel nonprestressed reinforcement in shotcrete members, the clear spacing shall be in accordance with (a) or (b):
(a) The clear spacing between bars shall be at least the greater of 6d₆ and 2½ in.

(b) If two curtains of reinforcement are provided, the clear spacing between bars in the curtain nearer the nozzle shall be at least 12d₆. The clear spacing between bars in the remaining curtain shall conform to (a).

25.2.7.1 It shall be permitted to use a clear spacing that does not meet 25.2.7(a) or 25.2.7(b) provided shotcrete mockup panels are used to demonstrate proper reinforcement encasement in accordance with (a) and (b):

(a) The shotcrete mockup panels shall be representative of the most complex reinforcement configurations to be encountered.

(b) The licensed design professional shall specify the shotcrete mock-up panel quantity, frequency of shooting per nozzleman and member type, and panel thickness to verify reinforcement encasement.

Comment: To reflect current state-of-the-art, minimum bar sizes are No. 6 in ACI 318-19.

25.2.8 For prestressed strands in shotcrete members, minimum center-to-center spacing s shall satisfy 25.2.4, except as permitted in 25.2.6.

25.2.9 For prestressed wire in shotcrete members, minimum center-to-center spacing s shall satisfy the requirements for wire in 25.2.5, except as permitted in and 25.2.6.

Comment: ACI 318-19 appropriately addresses prestressing strand and wire use in shotcrete members.

1908.5 Preconstruction tests. Where preconstruction tests are required by Section 1908.4, a test panel shall be shot, cured, cored or sawn, examined and tested prior to commencement of the project.

The sample panel shall be representative of the project and simulate job conditions as closely as possible. The panel thickness and reinforcing shall reproduce the thickest and most congested area specified in the structural design.

panel, shotcrete test—a shotcrete specimen prepared in accordance with ASTM C1140 for evaluation of shotcrete.

panel, shotcrete mockup—a shotcrete specimen that simulates the size and detailing of reinforcement in a proposed structural member for preconstruction evaluation of the nozzle operator’s ability to encase the reinforcement.

26.5.2 Concrete placement and consolidation

26.5.2.1 Compliance requirements:

(o) A certified shotcrete nozzle operator shall place all shotcrete.

(p) If a project-specific shotcrete mockup panel is required, each nozzle operator shall have demonstrated the ability to shoot an approved shotcrete mockup panel.

Comment: ACI 318-19 uses the current terminology of mockup panel versus sample panel. ACI 318-19 has clear language addressing size and detailing of reinforcement.
**Comment: ACI 318-19 has specific requirements for sampling and testing cores**

It shall be shot at the same angle, using the same nozzlemans and with the same concrete mix design that will be used on the project. The equipment used in preconstruction testing shall be the same equipment used in the work requiring such testing, unless substitute equipment is approved by the building official.

26.12.4.1 Compliance requirements:

(a) Specimens for acceptance tests shall be in accordance with (1) and (2):

1. Test panels shall be prepared in the same orientation and by the same nozzle operator placing shotcrete.
2. Cores shall be obtained, conditioned, and tested in accordance with ASTM C1604.

**Comment: ACI 318-19 adds requirements for placement of shotcrete to be performed by the same nozzle operator doing the work.**

Reports of preconstruction tests shall be submitted to the building official as specified in Section 1704.5.

**Comment: ACI 318-18 contains more specific requirements including but not limited to frequency of tests.**

<table>
<thead>
<tr>
<th>1908.4.2 Clearance</th>
<th>25.2.10 For ties, hoops, and spiral reinforcement in columns to be placed with shotcrete, minimum clear spacing shall be 3 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Where No. 5 or smaller bars are used, there shall be a minimum clearance between parallel reinforcement bars of 2½ inches (64 mm). When bars larger than No. 5 are permitted, there shall be a minimum clearance between parallel bars equal to six diameters of the bars used. Where two curtains of steel are provided, the curtain nearer the nozzle shall have a minimum spacing equal to 12 bar diameters and the remaining curtain shall have a minimum spacing of six bar diameters.</td>
<td>25.2.10.1 It shall be permitted to use a clear spacing other than 3 in. provided shotcrete mockup panels are used to demonstrate proper encasement of the reinforcement in accordance with 25.2.7.1</td>
</tr>
<tr>
<td><strong>Exception:</strong> Subject to the approval of the building official, required clearances shall be reduced where it is demonstrated by preconstruction tests that adequate encasement of the bars used in the design will be achieved.</td>
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</tbody>
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<table>
<thead>
<tr>
<th>1908.4.3 Splices</th>
<th>25.5—Splices</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lap splices of reinforcing bars shall utilize the noncontact lap splice method with a minimum clearance of 2 inches (51 mm) between bars. The use of contact lap splices necessary for support of the reinforcing is permitted where approved by the building official, based on satisfactory preconstruction tests that show that adequate encasement of the bars will be achieved, and provided that the splice is oriented so that a plane through the center of the spliced bars is perpendicular to the surface of the shotcrete.</td>
<td>25.5.1.6 Non-contact lap splices for reinforcement in shotcrete shall have a clear spacing in accordance with (a) or (b):</td>
</tr>
<tr>
<td>(a) For No. 6 and smaller bars, the clear spacing between bars shall be at least the greater of 6d₆ and 2½ in.</td>
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<tr>
<td>(b) For No. 7 and larger bars, the clear spacing shall be established using a shotcrete mockup panel to demonstrate that the reinforcement is properly encased.</td>
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</tr>
<tr>
<td>25.5.1.7 Contact lap splices for reinforcement in shotcrete shall be oriented with the plane of the spliced bars perpendicular to the surface of the shotcrete and approved by the licensed design professional based on a shotcrete mockup panel to demonstrate that the reinforcement is properly encased.</td>
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</tbody>
</table>

1908.4.4 Spirally tied columns. Shotcrete shall not be applied to spirally tied columns.

<table>
<thead>
<tr>
<th>25.2.10 For ties, hoops, and spiral reinforcement in columns to be placed with shotcrete, minimum clear spacing shall be 3 in.</th>
</tr>
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<tbody>
<tr>
<td>25.2.10.1 It shall be permitted to use a clear spacing other than 3 in. provided shotcrete mockup panels are used to demonstrate proper encasement of the reinforcement in accordance with 25.2.7.1</td>
</tr>
</tbody>
</table>

**Chapter 19—Concrete: Design and Durability Requirements**

19.3.3.4 Wet-mix shotcrete shall be sampled in accordance with ACI 318-19.
with ASTM C172, and air content shall be measured in accordance with ASTM C231 or ASTM C173.

19.3.3.5 Dry-mix shotcrete shall be sampled and air content shall be measured as directed by the licensed design professional.

Comment: ACI 318-19 includes specific requirements for sampling wet-mix and dry-mix shotcrete

19.3.3.6 For f’c exceeding 5000 psi, reduction of air content indicated in Tables 19.3.3.1 and 19.3.3.3 by 1.0 percentage point is permitted.

Comment: ACI 318-19 reflects the durability and performance of higher strength concrete by relaxing the requirements for air content.

1908.6 Rebound. Any rebound or accumulated loose aggregate shall be removed from the surfaces to be covered prior to placing the initial or any succeeding layers of shotcrete. Rebound shall not be used as aggregate.

Comment: ACI 318-19 clearly address overspray, rebound and cuttings.

26.5.2 Concrete placement and consolidation

26.5.2.1 Compliance requirements:

(i) Prior to placement of a new layer of shotcrete, rebound and overspray from adjacent placements shall be removed.

(k) Cuttings and rebound shall not be incorporated into the Work.

Comment: ACI 318-19 includes requirements for placement of subsequent concrete not addressed in the 2018 IBC.

1908.7 Joints. Except where permitted herein, unfinished work shall not be allowed to stand for more than 30 minutes unless edges are sloped to a thin edge. For structural elements that will be under compression and for construction joints shown on the approved construction documents, square joints are permitted. Before placing additional material adjacent to previously applied work, sloping and square edges shall be cleaned and wetted.

Comment: ACI 318-19 includes requirements for placement of subsequent concrete not addressed in the 2018 IBC.

1908.7 Joints. Except where permitted herein, unfinished work shall not be allowed to stand for more than 30 minutes unless edges are sloped to a thin edge. For structural elements that will be under compression and for construction joints shown on the approved construction documents, square joints are permitted. Before placing additional material adjacent to previously applied work, sloping and square edges shall be cleaned and wetted.

Comment: ACI 318-19 includes requirements for placement of subsequent concrete not addressed in the 2018 IBC.

1908.8 Damage. In-place shotcrete that exhibits sags, sloughs, segregation, honeycombing, sand pockets or other obvious defects shall be removed and replaced. Shotcrete above sags and sloughs shall be removed and replaced while

26.5.2 Concrete placement and consolidation

26.5.2.1 Compliance requirements:

(m) Before placing additional material onto hardened shotcrete, laitance shall be removed, joints shall be cleaned, and the surface dampened.

Comment: ACI 318-19 includes requirements for placement of subsequent concrete not addressed in the 2018 IBC.
### 1908.9 Curing

During the curing periods specified herein, shotcrete shall be maintained above 40°F (4°C) and in moist condition.

*Comment: This requirement in ACI 318-19 is applicable to all concrete and not specifically called out for shotcrete.*

#### 1908.9.1 Initial curing

Shotcrete shall be kept continuously moist for 24 hours after shotcreting is complete or shall be sealed with an approved curing compound.

26.5.3 Curing concrete and shotcrete

26.5.3.2 Compliance requirements:

(f) Shotcrete shall be cured in accordance with (1) through (3).

1. For 24 hours from completion of placement, initial curing shall be provided by one of the following methods:

   i. Ponding, fogging, or continuous sprinkling;
   
   ii. Absorptive mat, fabric, or other protective covering kept continuously moist;
   

2. After 24 hours from completion of placement, final curing shall be provided by one of the following methods:

   i. Same method used in the initial curing process;
   
   ii. Sheet materials;
   
   iii. Other moisture-retaining covers kept continuously moist.

*Comment: ACI 318-19 reflects specific requirements addressed in the appropriate ASTM product specifications for shotcrete.*

#### 1908.9.2 Final curing

Final curing shall continue for seven days after shotcreting, or for three days if high early-strength cement is used, or until the specified strength is obtained. Final curing shall consist of the initial curing process or the shotcrete shall be covered with an approved moisture-retaining cover.

26.5.3 Curing concrete and shotcrete

26.5.3.2 Compliance requirements:

3. Final curing shall be maintained for a minimum duration of not less than the following:

   i. 7 days,
   
   ii. 3 days if high early-strength cement or an accelerating admixture is used.

*Comment: ACI 318-19 provides time-period for curing but appropriately defers to the ASTM specifications for methods.*

#### 1908.9.3 Natural curing

Natural curing shall not be used in lieu of that specified in this section unless the relative humidity remains at or above 85 percent, and is authorized by the registered design professional and approved by the building official.

*Comment: ACI 318-19 appropriates defers curing methods as provided in the applicable ASTM product specifications.*

#### 1908.10 Strength tests

Strength tests for shotcrete shall be made by an approved agency on specimens that are representative of the work and that have been water soaked for not fewer than 24 hours prior to testing. Where the maximum size aggregate is larger than 3/8 inch (9.5 mm), specimens shall consist of not less than three 3-inch-diameter (76 mm) cores or 3-inch (76 mm) cubes. Where the maximum-size aggregate is 3/8 inch (9.5 mm) or smaller, specimens shall consist of not less than 2-inch-diameter (51 mm) cores or 2-inch (51 mm) cubes.

Cores shall be obtained, conditioned, and tested in accordance with ASTM C1604

26.12.4 Acceptance for shotcrete

26.12.4.1 Compliance requirements:

(a) Specimens for acceptance tests shall be in accordance with (1) and (2):
(1) Test panels shall be prepared in the same orientation and by the same nozzle operator placing shotcrete.

(2) Cores shall be obtained, conditioned, and tested in accordance with ASTM C1604.

(b) Strength level of a shotcrete mixture shall be acceptable if (1) and (2) are satisfied:

(1) Every arithmetic average of the strengths from three consecutive test panels equals or exceeds $f'_{c}$.

(2) The average compressive strength of three cores from a single test panel is not less than 0.85 $f'_{c}$ with no core having a strength less than 0.75 $f'_{c}$.

(c) If either of the requirements of 26.12.4.1(b) are not satisfied, steps shall be taken to increase the average of subsequent strength results.

(d) Requirements for investigating low strength-test results shall apply if the requirements of 26.12.4.1(b)(2) are not met.

19.3.3.3 Wet-mix shotcrete subject to freezing-and-thawing Exposure Classes F1, F2, or F3 shall be air entrained. Dry-mix shotcrete subject to freezing-and-thawing Exposure Class F3 shall be air entrained. Except as permitted in 19.3.3.6, air content shall conform to Table 19.3.3.3

Comment: ACI 318-19 appropriately references ASTM C1604 Obtaining and Testing Drilled Cores of Shotcrete for sampling and testing shotcrete cores.

1908.10.1 Sampling. Specimens shall be taken from the in-place work or from test panels, and shall be taken not less than once each shift, but not less than one for each 50 cubic yards (38.2 m$^3$) of shotcrete.

26.12.2 Frequency of testing

26.12.2.1 Compliance requirements:

(d) For shotcrete, prepare a shotcrete test panel for each mixture and each nozzle operator at least once per day or for every 50 yd$^3$ placed, whichever results in the greater number of panels

1908.10.2 Panel criteria. Where the maximum-size aggregate is larger than 3/8 inch (9.5 mm), the test panels shall have minimum dimensions of 18 inches by 18 inches (457 mm by 457 mm). Where the maximum-size aggregate is 3/8 inch (9.5 mm) or smaller, the test panels shall have minimum dimensions of 12 inches by 12 inches (305 mm by 305 mm).


Panels shall be shot in the same position as the work, during the course of the work and by the nozzlemen doing the work

The conditions under which the panels are cured shall be the same as the work.


1908.10.3 Acceptance criteria. The average compressive strength of three cores from the in-place work or a single test panel shall equal or exceed 0.85 $f'_{c}$ with no single core less than 0.75 $f'_{c}$. The average compressive strength of three cubes taken from the in-place work or a single test panel shall equal or exceed $f'_{c}$ with no
individual cube less than 0.88 $f'_c$. To check accuracy, locations represented by erratic core or cube strengths shall be retested.

(b) For shotcrete, a strength test shall be the average strengths of at least three 3-in. diameter cores taken from a test panel prepared in accordance with ASTM C1140 and tested at 28 days from time of placement or at test age designated for $f'_c$.

26.12.4 Acceptance for shotcrete

26.12.4.1 Compliance requirements:

(a) Specimens for acceptance tests shall be in accordance with (1) and (2):

(1) Test panels shall be prepared in the same orientation and by the same nozzle operator placing shotcrete.

(2) Cores shall be obtained, conditioned, and tested in accordance with ASTM C1604.

(b) Strength level of a shotcrete mixture shall be acceptable if (1) and (2) are satisfied:

(1) Every arithmetic average of the strengths from three consecutive test panels equals or exceeds $f'_c$.

(2) The average compressive strength of three cores from a single test panel is not less than 0.85 $f'_c$ with no core having a strength less than 0.75 $f'_c$.

(c) If either of the requirements of 26.12.4.1(b) are not satisfied, steps shall be taken to increase the average of subsequent strength results.

(d) Requirements for investigating low strength-test results shall apply if the requirements of 26.12.4.1(b)(2) are not met.

19.3.3.3 Wet-mix shotcrete subject to freezing-and-thawing Exposure Classes F1, F2, or F3 shall be air entrained. Dry-mix shotcrete subject to freezing-and-thawing Exposure Class F3 shall be air entrained. Except as permitted in 19.3.3.6, air content shall conform to Table 19.3.3.3.

Comment: ACI 318-19 provides general requirements and appropriately references ASTM C1140 Standard Practice for Preparing and Testing Specimens from Shotcrete Test Panel

Chapter 26 – Construction Documents and Inspection

26.3—Member information

26.3.1 (b) Members to be constructed using shotcrete

26.3.2(a) Use of shotcrete for structural members not identified in the construction documents as required to be placed by shotcrete shall be permitted in accordance with the project contract documents.

26.4—Concrete materials and mixture requirements

26.4.1.2.1 Compliance requirements:

(ee) For shotcrete, the aggregate gradation shall comply with ASTM C1436.

26.4.1.4 Admixtures
26.4.1.4.1 Compliance requirements:
(e) Admixtures used in shotcrete shall conform to ASTM C1141.

26.4.1.6 Packaged, pre-blended, dry, combined materials for shotcrete

26.4.1.6.1 Compliance requirements:
(a) Packaged, pre-blended, dry, combined materials for shotcrete shall conform to ASTM C1480.
(d) Documentation of shotcrete mixture characteristics shall be submitted for review by the licensed design professional before the mixture is used and before making changes to mixtures already in use. Evidence of the ability of the proposed shotcrete mixture to comply with the concrete mixture requirements in the construction documents shall be included in the documentation.

Comment: ACI 318-19 includes material and mixture requirements not addressed in the 2018 IBC.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
No increase to initial cost of construction. This change removes antiquated criteria for the International Building Code (IBC) and simply refers to updated, but comparable, criteria in American Concrete Institute Building Code Requirements for Structural Concrete and Commentary (ACI 318). The provisions of ACI 318 are more inclusive of design and construction methods conditions and provide increased flexibility for designers and contractors. In many instances this increased flexibility has the potential to reduce costs.

Proposal # 4539

S155-19
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).

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, liquefaction 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 repellent (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 constituents 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):**

<table>
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<tr>
<th>Sample</th>
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<th>Permeance [ng/Pa s m²]</th>
<th>Permeability [ng/Pa s m]</th>
<th>US Perms</th>
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</table>

**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*


Building with Earth: Design and Technology of Sustainable Architecture. Gernot Minke, Birkhauser (Bern, 2009)
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.
2018 International Building Code

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 \( \frac{3}{4} \) 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.

Add new text as follows:

2109.2.4.8.1 Conditions where lathing is not required. For unstabilized adobe walls finished with clay-lime plaster, lathing shall be allowed to be omitted at the discretion of the Building Official when evidence of adequate mechanical bonding is demonstrated to and approved by the building official.

2109.2.4.8.2 Lime Plaster. Lime plaster is any plaster with a binder composed of calcium hydroxide, \((\text{CaOH})\) 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.

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. Lime plasters are frequently recommended for use on earthen and other monolithic masonry systems, however they are currently not expressly permitted by the IBC. This proposal includes accepted industry best practices in allowing the use of lime plaster for adobe wall systems.

Other comments related to this proposal:

- Necessity: Lime plasters are a desirable finishing system that is relatively durable, vapor permeable, and with somewhat less embodied carbon than conventional cement stuccos. However, as they have different properties from other plasters, specific requirements for this material are necessary.
- Definition and standards: Lime plasters have accepted definitions that have been developed by industry associations. Those definitions are included here for purposes of clarity; this text is identical to that included in IRC Appendix S.
- Omission of lath at Building Official’s Discretion: In some cases and due to the specific characteristics of a plaster, substrate, and the skill of the installer, plaster may be successfully installed over substrates that would ordinarily require lath. This section gives discretion to the building official to permit such installations where evidence of sufficient bonding can be shown.
- Lath requirements for unstabilized adobe with lime plaster: The New Mexico Earthen Materials Building code requires, (and conservation practice often recommends) that lath not be used, and instead the head joints of adjacent adobe walls be left open allow keying of the applied plaster and the wall assembly – this practice has detrimental structural implications that cannot be adequately addressed within the scope of this proposal and is not allowed by the proposed language.
- Permeability of various finished (per Straube):
Table 2.3: Results of Vapor Permeance Test Results [Straube, 2000]

<table>
<thead>
<tr>
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<th>US Perms</th>
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Bibliography:
- 2015 New Mexico Earthen Building Materials Code
- 2015 International Residential Code Appendix S - Strawbale Construction
- Building with Earth: Design and Technology of Sustainable Architecture. Gernot Minke, Birkhauser (Bern, 2009)
- Sustainable Building with Earth. Horst Schroeder, Springer International Publishing (Switzerland, 2016)

Cost Impact: The code change proposal will not increase or decrease the cost of construction. This proposal offers an alternative to portland cement plasters, and as such does not represent an increase or decrease in the cost of construction.
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.

Add new text as follows:

2109.2.4.8.1 Cement-lime plaster. Cement-lime plaster shall be any plaster mix type CL, F or FL, as described in ASTM C926.

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.

This proposal includes language for cement-lime finishes and is informed by code provisions and guidance from IRC Strawbale Construction Appendix S, the 2015 New Mexico Earthen Building Materials Code, and ASTM 2392-10 Standard Guide for Design of Earthen Wall Building Systems. Definitions for Cement-lime plasters shown here are according to accepted industry terminology.

Permeability of various finished (per Straube):

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<tr>
<th>Sample</th>
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Table 2.3: Results of Vapor Permeance Test Results [Straube, 2000]

The cement-lime plaster types allowed in the new section, as described in ASTM 926, each contain between 75-200% as much lime as cement, ensuring a vapor permeance of at least 7 perms.

2015 International Residential Code Appendix S - Strawbale Construction
Cost Impact: The code change proposal will not increase or decrease the cost of construction.

This proposal offers an alternative to portland cement plasters, and as such does not represent an increase or decrease in the cost of construction.
S159-19

IBC: 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)

**Proponent:** Ben Loescher, representing Self (bloescher@lmarchitectsinc.com); Martin Hammer, representing Martin Hammer, Architect (mhammer@pacbell.net); David Eisenberg, representing DCAT (strawnet@gmail.com)

**2018 International Building Code**

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.

Add new text as follows:

2109.2.4.8.1 Clay Plaster Clay plaster shall comply with this section.

2109.2.4.8.2 General. Clay plaster shall be any plaster having a clay or clay subsoil binder. Such plaster shall contain sufficient clay to fully bind the sand, fine aggregate or other granular material, and shall be permitted to contain reinforcing fibers. Acceptable reinforcing fibers include chopped straw, sisal, and animal hair.

2109.2.4.8.3 Clay subsoil requirements. The suitability of clay subsoil shall be determined in accordance with the Figure 2 Ribbon Test and the Figure 3 Bail Test in the appendix of ASTM 2392/E2392M.

2109.2.4.8.4 Weather exposed locations. Clay plaster exposed to water from direct or wind-driven rain, snow, or irrigation spray shall be finished with a clay-lime plaster, lime plaster, or other approved erosion-resistant finish. The use of clay plasters shall not be permitted on weather exposed parapets.

2109.2.4.8.5 Prohibited finish coat. Plaster containing Portland cement shall not be permitted as a finish over clay plaster.

2109.2.4.8.6 Conditions where lathing is not required. For unstabilized adobe walls finished with unstabilized clay plaster, lathing shall not be required.

**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.

This proposal includes language allowing limited use of clay plasters in exterior applications. This language is based on provisions that have been successfully used in New Mexico, and are somewhat more restrictive than those found in the 2015 New Mexico Earthen Building Materials Code, the bulk of which have been in use in that State since the 1980s.

Notes related to this proposal:

- **Necessity:** Clay plasters are a desirable finishing system that is readily available, low cost, low-embodied carbon, and vapor permeable. However, due to the susceptibility of clay plasters to erosion, specific requirements for this material are necessary.
- **Constituents:** As clay plasters are for the most part made by the applicator from available materials of varying properties, some guidance on these constituent elements is required.
- **Clay content:** This language asserts that clay plasters must use clay as a binder, rather than some other material that would have different qualities and might have different requirements.
- **Reinforcing Fibers:** Reinforcing fibers are frequently added to clay plasters to improve their fabric strength as well as to inhibit and control cracking. This proposal includes frequently used fibers also referenced in IRC Appendix S (chopped straw being the most common pervasive of these), but does not restrict the use of other fibers. Additionally, as clay plasters may be successfully installed without reinforcing fiber (dependent on the material qualities of the clay/sand/aggregate mix), this proposal does not require them.
- **Clay subsoil requirements:** Some relatively rare types of clay are not suitable for use in clay plasters as they are too expansive, or do not provide sufficient binding characteristics. This section proposes the use of a simple field test from ASTM-2392 to assess suitability.
- **Thickness and coats:** A minimum thickness is required to provide the desired weather protection benefits anticipated elsewhere in this code section. The 7/8” Minimum matches industry practice and is minimum thickness is used in the New Mexico Earthen Materials Building Code.
- **Weather exposed locations:** When directly exposed to weather, clay plasters are susceptible to erosion. This section prescribes minimum requirements for protection of clay plasters within these conditions. This section is adapted from language in IRC Appendix S. Lime plasters and linseed oil surface applications have been successfully used to inhibit the erosion of rain exposed clay plasters.
- **Use prohibition on parapets:** Adobe parapet walls are more susceptible to weather damage than other building surfaces due to their exposed location, more complicated detailing and potential for contact with embanked snow. Lime and linseed oil treated clay plasters do not provide...
sufficient protection for these conditions; as such use of clay plasters in these locations is prohibited.

- Prohibited finish coat: Cement plasters, including soil cement plasters, have not been demonstrated to perform adequately when applied over clay plasters - their use is prohibited here.
- Permeability of various finished (per Straube) follows, noting: “Earth plasters are generally more permeable than even lime plasters. The addition of straw increases the permeability further. A 38 mm (1.5”) thick earth plaster can have a permeance of over 1200 metric perms (over 20 US Perms), in the same order as building papers and housewraps.”

<table>
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<th>Sample</th>
<th>t [mm]</th>
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</table>

**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*
- Building with Earth: Design and Technology of Sustainable Architecture. Gernot Minke, Birkhauser (Bern, 2009)
- Sustainable Building with Earth. Horst Schroeder, Springer International Publishing (Switzerland, 2016)

**Cost Impact:** The code change proposal will decrease the cost of construction.
This proposal offers an less expensive alternative to portland cement plasters for specific building conditions and exposures, and as such will in some cases represent a decrease in the cost of construction.
2018 International Building Code

Revise as follows:

2205.2.1.1 Seismic Design Category B or C. Structures assigned to Seismic Design Category B or C shall be of any construction permitted in Section 2205. Where a response modification coefficient, $R$, in accordance with ASCE 7, Table 12.2-1, is used for the design of structures assigned to Seismic Design Category B or C, the structures shall be designed and detailed in accordance with the requirements of AISC 341. Beam-to-column moment connections in special moment frames and intermediate moment frames shall be prequalified in accordance with AISC 341 Section K1, qualified by testing in accordance with AISC 341 Section K2, or shall be prequalified in accordance with AISC 358.

Exception: The response modification coefficient, $R$, designated for “Steel systems not specifically detailed for seismic resistance, excluding cantilever column systems” in ASCE 7, Table 12.2-1, shall be permitted for systems designed and detailed in accordance with AISC 360, and need not be designed and detailed in accordance with AISC 341.

2205.2.1.2 Seismic Design Category D, E or F. Structures assigned to Seismic Design Category D, E or F shall be designed and detailed in accordance with AISC 341, except as permitted in ASCE 7, Table 15.4-1. Beam-to-column moment connections in special moment frames and intermediate moment frames shall be prequalified in accordance with AISC 341 Section K1, qualified by testing in accordance with AISC 341 Section K2, or shall be prequalified in accordance with AISC 358.

Add new text as follows:

358-16/s1-18: Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications, Including Supplement No. 1

Reason: The purpose of this proposal is to introduce a new reference to an existing AISC standard in Section 2205. First issued in 2005, AISC 358 includes specific requirements for a number of prequalified connections for intermediate special moment frames and special moment frames. In years past, it has been acceptable for AISC 358 to be a direct reference in AISC 341. However, supplements are now being processed for AISC 358 more frequently than new editions of AISC 341. This has the net effect of not recognizing these newer supplements in the building code, thus leading to confusion for building officials, registered design professionals and manufacturers of the prequalified connections. Introduction of a direct reference in the IBC permits the most up-to-date edition of the standard to be referenced.

This proposal adopts a new supplement for AISC 358, which is not recognized by AISC 341-16. AISC 358-16 Supplement 1(2018) adds a new prequalified moment connection, the proprietary SlottedWeb Moment Connection, in a new Chapter 14. Additionally, Chapter 11 covering the SidePlate Moment Connection has been expanded to include HSS columns and to permit bolted connections. Finally, Chapter 10 covering the ConXtech CONXL Moment Connection has been revised to address a manufacturing safety issue.

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

This code change proposal provides a direct reference within the IBC to a standard that was previously a secondary reference via AISC 341. The addition of AISC 358 as a direct reference will provide building officials, designers and manufacturers with access to the most recent edition of the standard, and provide additional options for prequalified connections.

Staff Analysis: A review of the standard proposed for inclusion in the code, AISC 358-16/S1-18, 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.
2018 International Building Code

Revise as follows:

[BS] STORAGE RACKS

Cold-formed or hot-rolled steel structural members which are formed into steel storage racks, including pallet storage racks, movable-shelf racks, rack-supported systems, automated storage and retrieval systems (stacker racks), push-back racks, pallet-flow racks, case-flow racks, pick modules and rack-supported platforms. Other types of racks, such as drive-in or drive-through racks, cantilever racks, portable racks or racks made of materials other than steel, are not considered storage racks for the purpose of this code.

Add new text as follows:

202 DEFINITIONS [BS] STORAGE RACKS, STEEL CANTILEVERED

A framework or assemblage comprised of cold-formed or hot-rolled steel structural members, primarily in the form of vertical columns, extended bases, horizontal arms projecting from the faces of the columns, and longitudinal (down-aisle) bracing between columns. There may be shelf beams between the arms, depending on the products being stored; this definition does not include other types of racks such as pallet storage racks, drive-in racks, drive-through racks, or racks made of materials other than steel.

Revise as follows:

2209.1 Storage Steel storage racks. The design, testing and utilization of steel 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 steel storage racks shall be in accordance with Section 15.5.3 of ASCE 7.

2209.2 Cantilevered steel Steel cantilevered storage racks. The design, testing, and utilization of steel 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 steel cantilevered steel storage racks shall be in accordance with Section 15.5.3 of ASCE 7.

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

Reason: These code changes align definitions in IBC section 202 and ASCE 7-16 section 11.2 concerning steel storage racks. Adding the term “steel” to “storage racks” emphasizes that the racks must be made of steel, not wood, or another material. The addition of the steel cantilevered storage rack definition acknowledges that this common type of storage rack has different loading and design requirements than a simple steel storage rack. The proposed definition for steel cantilevered storage racks is identical to the one found in ASCE 7-16 section 11.2. Since IBC section 2209.2 specifically addresses the design of steel cantilevered storage racks, it makes sense to add the corresponding definition in this section of the code.

The changes in IBC 1705.12.7 Storage racks clarify that periodic special inspection is required for steel storage racks, regular or cantilevered, that are 8 feet or greater in height in Seismic Design Categories D, E, or F.

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

The code change is simply a clarification of what requirements apply to steel storage racks, not a change in requirements.

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

The code change is simply a clarification of what requirements apply to steel storage racks, not a change in requirements.
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 stairs, ladders and open edge guards for use with material handling structures shall be in accordance with ANSI/MH32.1.

MH28.3-2018: Design, Testing and Utilization of Industrial Steel Work Platforms

MH28.2-2018: Design, Testing and Utilization of Industrial Boltless Steel Shelving

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.

Proposal # 5183
2018 International Building Code

2210.1.1 Steel decks. The design and construction of cold-formed steel decks shall be in accordance with this section.

2210.1.1.1 Noncomposite steel floor decks. Noncomposite steel floor decks shall be permitted to be designed and constructed in accordance with ANSI/SDI-NC1.0.

2210.1.1.2 Steel roof deck. Steel roof decks shall be permitted to be designed and constructed in accordance with ANSI/SDI-RD1.0.

2210.1.1.3 Composite slabs on steel decks. Composite slabs of concrete and steel deck shall be permitted to be designed and constructed in accordance with SDI-C.

Add new text as follows:

2210.1.1.4 Construction documents. The construction documents for steel decks shall include the following information:

1. The deck type, profile, and number of spans.
2. The slab depth and metal thickness.
3. Whether deck is galvanized.
4. Deck attachment to supports:
   1. Attachment pattern for welds and screws.
   2. Size of welds and screws.
   3. Deck side lap attachments

Reason: Information concerning Metal Decks and their attachment to supporting framing are of critical importance in determining the response of buildings to their intended loads. Not only does it indicate the capacity for support of gravity loads, but also the strength and serviceability of diaphragms within buildings to resist lateral loads. Therefore, this information needs to be included on the Construction Documents to assist Jurisdictions in evaluating Code Compliance.

Bibliography: Chris Snidow is Professional Engineer and is also a Commercial Plan Reviewer for the County of Henrico.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. The purpose of this proposal is nothing more than a requirement for additional information on the Construction Documents. The information merely describes the work that is to be done and therefore has no impact on the Cost of Construction.
S164-19

IBC®: 2211.1

Proponent: Jon-Paul Cardin, American Iron and Steel Institute, representing American Iron and Steel Institute (JCardin@steel.org)

2018 International Building Code

Revise as follows:

2211.1 Structural framing. For cold-formed steel light-frame construction, the design, manufacture, installation, and installation quality of the following structural framing systems, including their members and connections, shall be in accordance with AISI S240, and Sections 2211.1.1 through 2211.1.3, as applicable:

1. Floor and roof systems.
2. Structural walls.
3. Shear walls, strap-braced walls and diaphragms that resist in-plane lateral loads.
4. Trusses.

Reason: The intent of this code change proposal is to update the charging language of Section 2211 to more accurately reflect the content of the section and the scope of AISI S240. The updated language provides the user more clarification regarding the subject matter of the section and the reference document.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
This code change proposal does not make any technical changes to the provisions of the IBC. It simply serves to provide clarification to the user.
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.
IBC®: 2303.2

Proponent: 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, the flame front shall not progress more than 10 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.
S167-19

IBC: 2303.2.3, 2303.3.2.3.1 (New)

Proponent: 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.
PERMANENT INDIVIDUAL TRUSS MEMBER RERAINT (PITMR) Restraint that is used to prevent local buckling of an individual truss chord or web member because of the axial forces in the individual truss member.

PERMANENT INDIVIDUAL TRUSS MEMBER DIAGONAL BRACING (PITMDB) Structural member or assembly intended to permanently stabilize the PITMRs.

INDIVIDUAL TRUSS MEMBER A truss chord or truss web.

Revise as follows:

2303.4.1.1 Truss design drawings. The written, graphic and pictorial depiction of each individual truss shall be provided to the building official for approval prior to installation. Truss design drawings shall be provided with the shipment of trusses delivered to the job site. Truss design drawings shall include, at a minimum, the following information:

1. Slope or depth, span and spacing.
2. Location of all joints and support locations.
3. Number of plies if greater than one.
4. Required bearing widths.
5. Design loads as applicable, including:
   5.1. Top chord live load.
   5.2. Top chord dead load.
   5.3. Bottom chord live load.
   5.4. Bottom chord dead load.
   5.5. Additional loads and locations.
   5.6. Environmental design criteria and loads (such as wind, rain, snow, seismic).
6. Other lateral loads, including drag strut loads.
7. Adjustments to wood member and metal connector plate design value for conditions of use.
8. Maximum reaction force and direction, including maximum uplift reaction forces where applicable.
9. Joint connection type and description, such as size and thickness or gage, and the dimensioned location of each joint connector except where symmetrically located relative to the joint interface.
10. Size, species and grade for each wood member.
11. Truss-to-truss connections and truss field assembly requirements.
12. Calculated span-to-deflection ratio and maximum vertical and horizontal deflection for live and total load as applicable.
13. Maximum axial tension and compression forces in the truss members.
14. Required permanent individual truss member restraint location and the method and details of restraint and diagonal bracing to be used in accordance with Section 2303.4.1.2.

2303.4.1.2 Permanent individual truss member restraint (PITMR) and permanent individual truss member diagonal bracing (PITMDB). Where permanent restraint of truss members is required on the truss design drawings designate the need for permanent individual truss member restraint, it shall be accomplished by one of the following methods:

1. Permanent individual truss member restraint and diagonal bracing shall be PITMR and PITMDB installed using standard industry lateral restraint and diagonal bracing details in accordance with generally TPI 1 section 2.3.3.1.1, accepted engineering practice, or Figures 2303.4.1.2(1a), (2a), and (3). Locations for lateral restraint shall be identified on the truss design drawing.
2. Individual truss member reinforcement in place of the specified lateral restraints (i.e., buckling reinforcement such as T-reinforcement, L-reinforcement, proprietary reinforcement, etc.) such that the buckling of any individual truss member is resisted internally by the individual truss through suitable means (for example, buckling reinforcement by T-reinforcement or L-reinforcement, proprietary reinforcement, etc.) such as T-reinforcement. The buckling reinforcement of individual truss members of the trusses shall be installed as shown on the truss design drawing or on supplemental truss member buckling reinforcement details provided by the truss designer or in accordance with Figures 2303.4.1.2(1b) and (2b).
3. A project-specific permanent individual truss member restraint/bracing design shall be permitted to be specified PITMR and PITMDB design provided by any registered design professional.
Add new text as follows:

2303.4.1.2.1 New Code Section Trusses installed without a diaphragm on the top or bottom chord shall require a project specific PITMR and PITMDB design prepared by a registered design professional.

Exception: Group U occupancies.

Revise as follows:

2303.4.1.3 Trusses spanning 60 feet or greater. The owner or the owner’s authorized agent shall contract with any qualified registered design professional for the design of the temporary installation restraint and diagonal bracing and the permanent individual truss member restraint bracing PITMR and PITMDB for all trusses with clear spans 60 feet (18 288 mm) or greater.

Add new text as follows:
FIGURE 2303.4.1.2.(1a) PITMR AND PITMDB FOR TRUSS WEB MEMBERS REQUIRING ONE ROW OF PITMR

S289
FIGURE 2303.4.1.2 (1b) ALTERNATIVE INSTALLATION USING BUCKLING REINFORCEMENT FOR TRUSS WEB MEMBERS IN LIEU OF ONE ROW OF PITMR
2303.4.1.2

FIGURE 2303.4.1.2.(2a) PITMR AND PITMDB FOR TRUSS WEB MEMBERS REQUIRING TWO ROWS OF PITMR

Section (Example of Double Row of PITMR with PITMDB on Web Members)
FIGURE 2303.4.1.2.2b ALTERNATIVE INSTALLATION USING BUCKLING REINFORCEMENT FOR TRUSS WEB MEMBERS IN LIEU OF TWO ROWS OF PITMR

<table>
<thead>
<tr>
<th>NUMBER OF ROWS OF PITMR SPECIFIED ON WEB MEMBER</th>
<th>SIZE OF TRUSS WEB</th>
<th>TYPE AND SIZE OF WEB FOR I OR U REINFORCEMENT</th>
<th>GRADE OF WEB REINFORCEMENT</th>
<th>MINIMUM LENGTH OF WEB REINFORCEMENT</th>
<th>MINIMUM CONNECTION OF WEB REINFORCEMENT TO WEB</th>
</tr>
</thead>
<tbody>
<tr>
<td>TWO</td>
<td>2x4</td>
<td>(2) - 2x4</td>
<td>Same species and grade or better than web member</td>
<td>90% of web or extend to within 6&quot; of end of web member, whichever is greater</td>
<td>(0.131&quot; x 3&quot;) nails at 6&quot; on-center</td>
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<td></td>
<td>2x6</td>
<td>(2) - 2x6</td>
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<td>2x8</td>
<td>(2) - 2x8</td>
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</table>

*Maximum allowable web length is 14"*
The purpose of this proposal is to bring clarity on how to physically install permanent individual truss member restraint and diagonal bracing of wood truss members. The current predominant industry standard of care for the installation of permanent individual truss member restraint and diagonal bracing of both wood truss chords and webs, is to have the truss installer (framer) rely on “standard industry details”. This is understood to mean; and is stated as such in TPI 1, section 2.3.3.1.1; to be the use of Building Component Safety Information (BCSI) – B3: Permanent Restrain/Bracing of Chords & Web Members. The reality in the field however, and has been documented by practicing structural engineers, is the truss installers / framers are: a) not familiar with BCSI-B3; b) are sometimes not provided a copy of that document with the trusses; and c) do not have the technical expertise required to interpret the various conditions and options listed in the BCSI – B3 document. The Owner, the Building Designer; the Truss Designer, the Truss Manufacturer and the Building Official are all relying on the ability of the Truss Installer / framer to accurately and completely interpret where and how to install the required restraint bracing and diagonal bracing whenever pre-engineered wood trusses are used. There is sufficient concern in the industry that the current practice is deficient and could lead to truss failures.

To remedy this situation, the Structural Building Components Association (SBCA), along with the National Council of Structural Engineers’ Associations (NCSEA), with input from the Truss Plate Institute (TPI) and the National Framers Alliance, have collaboratively prepared this Code Change Proposal as outlined below.
1. Definitions for an Individual Truss Member; a Permanent Individual Truss Member Restraint (PITMR); and Permanent Individual Truss Member Diagonal Bracing (PITMDB) are proposed. This will eliminate some current confusion within both the design community and on the job site with respect to what the specific members are to be called and what their purpose is. Currently terms such as bracing; bridging; continuous lateral brace (CLB); x-bracing; etc. are used and are do not necessarily mean the same thing to everyone.

2. We are proposing to add some clarification figures into the Code to assist both the Truss Installers and Building Inspectors to more clearly and easily understand when and how PITMR’s and PITMDB’s are to be installed.

3. The figures and associated connections are prescriptive, and the consensus among the groups preparing this CCP is that having these in the code is better than what is currently being used. The following are some engineering analysis items that need to be discussed to fully understand the rational that went into the preparation of this figures:

   a) There is a mutually understood, open question regarding “what is the actual lateral restraint force required to brace a web member that is in compression?”. The range of answers is from the traditional (albeit conservative) 2% of the axial compression force in the member; to 1% (currently used by AISI in light gage truss design), to even a possible value less than 1% depending on the end fixity provided by the top and bottom chords and associated diaphragms. To more accurately determine what the actual lateral restraint force is, there will need to be testing and further analysis required.

   b) The overwhelming consensus among the group involved in this CCP, was that having something installed was better than nothing, and we therefore strived to be consistent with the current field practices being used, but at the same time trying to provide a logical load path. By utilizing a minimum of (4) 10d (0.131 x 3”) nails connected to blocking each end that would go “into compression and bearing against adjacent trusses” would create a considerable lateral restraint capacity, and at the same time is a very similar to what a framer is currently installing.

   c) The vast majority of projects utilizing wood trusses will have both a plywood / OSB roof diaphragm, and a gypsum board ceiling diaphragm. It was agreed that with a roof and ceiling diaphragm installed, it allows us to limit this CCP to addressing web members only.

   d) For specialty projects where there is no diagram on the top or bottom chords, a project specific PITMR and PITMDB design will be required. This is critical for projects with spans less than 60 feet that have dropped ceilings and thus no ceiling diaphragm (schools, offices, etc.). While we acknowledge this will add some additional cost to those types of projects, the occupancy exposure clearly justifies the effort to make sure the trusses are properly installed. We also have provided an exception for Group U Occupancies that will allow industry standard details to still be used for those low risk projects.

For Building Departments that have experienced and trained field inspectors familiar with wood trusses, this CCP does not change their current process and ability to allow the use BCSI – B3. For smaller jurisdictions with limited personnel however, this CCP will provide more information and a tool to assist the Building Official in inspecting the truss installation.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
The cost of construction will not change for typical projects, since permanent individual truss member restraint and diagonal bracing of wood truss members is already required by the Code. For certain projects without ceiling diaphragms the cost of construction will increase slightly by the amount of the project specific design, but this increased cost is minimal and justified based on the occupancy risk typically associated with these types of projects.
2018 International Building Code

Revise as follows:

2303.7 Shrinkage. Consideration shall be given in design to the possible effects of wood cross-grain dimensional changes considered vertically that may occur in lumber fabricated in a green condition as a result of changes in the wood moisture content after installation.

Reason: This change removes the existing language “fabricated in a green condition” and clarifies that consideration is to be given to effects of cross-grain dimensional change resulting from changes in moisture content after installation. The proposed change broadens applicability beyond “lumber fabricated in a green condition” (i.e. where “green condition” is associated with lumber moisture content greater than 19% at time of installation) because design considerations for dimensional change in reference design documents apply for both “green” and “dry” material. Examples of design and detailing criteria are in documents such as AITC 104, NDS, the Forest Products Laboratory Wood Handbook - Wood as an Engineering Material (General Technical Report FPL-GTR-190) and manufacturer’s literature for engineered wood products. These documents include information on detailing at connections, adjustment factors for connection design, and guidance on consideration of shrinkage effects to facilitate level top edge surface after drying.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. The proposal is not intended to change how shrinkage considerations are handled so there is no cost impact.
2304.10.1 Connection fire resistance rating. Fire resistance ratings for connections in Type IV-A, IV-B, or IV-C construction shall be determined by one of the following:

1. Testing in accordance with Section 703.2 where the connection is part of the fire resistance test.
2. Engineering analysis that demonstrates that the temperature rise at any portion of the connection is limited to an average temperature rise of 250°F (139°C), and a maximum temperature rise of 325°F (181°C), for a time corresponding to the required fire resistance rating of the structural element being connected. For the purposes of this analysis, the connection includes connectors, fasteners, and portions of wood members included in the structural design of the connection.

Reason: IBC Sections 704.2 and 704.3 require connections of columns and other primary structural members to be protected with materials that have the required fire-resistance rating. This proposed change provides two options for demonstrating compliance with this requirement for connections in Types IV-A, IV-B and IV-C construction: a testing option and a calculation option.

Types IV-A, IV-B and IV-C construction utilize mass timber elements that have inherent fire resistance. The new provisions which added these construction types have explicit fire-resistance ratings and protection requirements. Option 1 allows connections that are part of a successful ASTM E119 fire resistance test to be considered acceptable evidence of meeting the requirements of Sections 704.2 and 704.3.

Some connections used in Types IV-A, IV-B and IV-C construction are not part of the mass timber element or assembly testing. For those connections, an engineering analysis is required. Analysis procedures have been developed that allow the protection of these connections to be designed based on test results of E119 fire tests from protection configurations using the wood member outside of the connection, additional wood cover, and/or gypsum board. The analysis procedures must demonstrate that the protection will limit the temperature rise at any portion of the connection, including the metal connector, the connection fasteners, and portions of the wood member that are necessary for the structural design of the connection. The average temperature rise limit of 250°F (139°C) and maximum temperature rise limit of 325°F (181°C) represent the fire separation and thermal protection requirements for wall and floor assemblies tested per ASTM E119 and ensure that the connection retains most of its initial strength throughout the fire-resistance rating time. Please note the Celsius values in parentheses are for temperature rise calculated as the difference between the final temperature and the initial temperature, not a direct conversion of a Fahrenheit temperature.

IBC 722 permits structural fire-resistance ratings of wood members to be determined using Chapter 16 of the National Design Specification® (NDS®) for Wood Construction. Where a wood connection is required to be fire-resistance rated, NDS Section 16.3 requires all components of the wood connection, including the steel connector, the connection fasteners, and the wood needed in the structural design of the connection, to be protected for the required fire-resistance rating time. NDS permits the connection to be protected by wood, gypsum board or other approved materials. AWC publication Technical Report 10: Calculating the Fire Resistance of Wood Members and Assemblies (https://www.awc.org/codes-standards/publications/tr10), which is referenced in the NDS Commentary to Chapter 16, has been specifically updated to provide guidance on and examples of connection designs meeting the requirements of IBC 704 and NDS 16.3.

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 not increase or decrease 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.
S171-19

IBC: 2304.10, 2304.10.8 (New)

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

2018 International Building Code

Revise as follows:

2304.10 Connectors and fasteners. Connectors and fasteners shall comply with the applicable provisions of Sections 2304.10.1 through 2304.10.7, 2304.10.8.

Add new text as follows:

2304.10.8 Bottom (sill) plate anchorage. Where field conditions preclude the placement of the minimum bottom plate anchors specified in Table 2304.10.1, a registered design professional shall provide a design for the attachment in accordance with accepted engineering practice.

Reason: Bottom plate is preferentially referenced in lieu of sill plate to match that same evolution in the IBC and AF&PA references. In residential construction many times there are short length wall framing for door openings, exterior built-up columns and post framing and similar construction where it is impractical to comply with the 2308.3 completely. The exception is provided to explicitly allow a design professional the ability to design appropriate attachment for these conditions. Insertion within the General Construction Requirements, section 2304 is to clarify that this change applies to both designed and conventional construction.

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

This proposal simply adds an additional accepted method of attachment by allowing a registered design professional to determine placement of foundation anchorage based on field conditions.
2018 International Building Code

Revise as follows:

2304.9 Lumber decking. Lumber decking shall be designed and installed in accordance with the general provisions of this code and Sections 2304.9.1 through 2304.9.5.3. Other lumber decking patterns and connection designs shall be substantiated through engineering analysis.

2304.9.1 General. Each piece of lumber decking shall be square-end trimmed. Where random lengths are furnished, each piece shall be square end trimmed across the face so that not less than 90 percent of the pieces are within 0.5 degrees (0.00873 rad) of square. The ends of the pieces shall be permitted to be beveled up to 2 degrees (0.0349 rad) from the vertical with the exposed face of the piece slightly longer than the opposite face of the piece. Tongue-and-groove decking shall be installed with the tongues up on sloped or pitched roofs with pattern faces down.

2304.9.2 Layup patterns. Lumber decking is permitted to be laid up following one of five standard patterns as defined in Sections 2304.9.2.1 through 2304.9.2.5. Other patterns are permitted to be used provided that they are substantiated through engineering analysis.

Reason: The proposed addition is intended to clarify that both alternative layup patterns as well as alternative fastening options which are substantiated by engineering analysis can be permitted. A current general statement indicating this is slightly modified and moved from Section 2304.9.2 to the charging language for the entire section on lumber decking, Section 2304.9. In this way, it is made clear that it applies to all the subsections, including, for instance, fastening prescribed in 2304.9.3.2 and Table 2304.9.3.2, not just to layup patterns. Alternative design can always be used in accordance with Section 104.11 if substantiated by engineering analysis and approved by the code official.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. The proposal adds a clarifying statement indicated alternative designs substantiated by engineering analysis may be accepted.
TABLE 2304.10.1
FASTENING SCHEDULE

<table>
<thead>
<tr>
<th>DESCRIPTION OF BUILDING ELEMENTS</th>
<th>NUMBER AND TYPE OF FASTENER</th>
<th>SPACING AND LOCATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood structural panels (WSP), subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framing&lt;sup&gt;a&lt;/sup&gt;</td>
<td>6d common or deformed (2&quot; × 0.113&quot;) (subfloor and wall)</td>
<td>Edges (inches) Intermediate supports (inches)</td>
</tr>
<tr>
<td>30. 3/8&quot; – 1/2&quot;</td>
<td>8d common or deformed (2 1/2&quot; × 0.131&quot;) (roof) or RSRS-01 (2 3/8&quot; × 0.113&quot;) nail (roof)&lt;sup&gt;d&lt;/sup&gt;</td>
<td>6</td>
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<tr>
<td>31. 19/32&quot; – 3/4&quot;</td>
<td>8d common (2 1/2&quot; × 0.131&quot;); or 6d deformed (2&quot; × 0.113&quot;) (subfloor and wall)</td>
<td>6</td>
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<tr>
<td>32. 7/8&quot; – 11/14&quot;</td>
<td>10d common (3&quot; × 0.148&quot;); or 8d deformed (2 1/2&quot; × 0.131&quot;)</td>
<td>6</td>
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</tbody>
</table>

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. Tabulated fastener requirements apply where the ultimate design wind speed is less than 140 mph. For wood structural panel roof sheathing attached to gable end roof framing and to intermediate supports within 48 inches of roof edges and ridges, nails shall be spaced at 4 inches on center where the ultimate design wind speed is greater than 130 mph in Exposure B or greater than 110 mph in Exposure C.

f. Where the ultimate design wind speed is less than or equal to 110 mph, roof sheathing attachment using the specified fasteners shall be installed 3 inches on center at all supports.

Reason: AWC is submitting a proposal to update roof sheathing nailing in Table 2304.10.1 to be based on ASCE7-16 wind loads and to agree with roof sheathing nailing in the 2018 Wood Frame Construction Manual. Wind uplift nailing requirements for common species of roof framing with specific gravities of 0.42 or greater (e.g. SPF, Hem-Fir) are the basis of the proposed nail spacing requirements in Table 2304.10.1 to meet the wind uplift loading requirements per ASCE 7-16 without being overly complex in specification of roof sheathing nailing. The basic nailing proposed is 6" o.c. at panel edges and 6" o.c. at intermediate supports in the field of the panel. As shown in the boxed cells of 2018 WFCM, Table 3.10A for the common case of roof framing spaced at 24 inches on center, nailing at intermediate supports in the interior portions of the roof is 6" o.c. for wind speeds within the scope of IBC 2308. The 6" o.c. spacing is also appropriate for edge zones except where ultimate wind speeds equal or exceed 130 mph in Exposure B and 110 mph in Exposure C where 4" o.c. nailing is needed. These special cases are addressed by the proposed modification to footnote "e".
To update the alternative fastening to uplift loading requirements per ASCE 7-16 without being overly complex in specification of roof sheathing attachment schedules, footnote “f” was added. The reference calculation leading to use of 3” o.c. spacing at all locations is based on the 0.113” diameter nail shank withdrawal from wood framing with specific gravity equal to 0.42 and pre-calculated wind uplift loads in WFCM Table 3.10. The use of a single 3” spacing at all supports was extended to staples based on the assumption that the ASCE 7-16 load increase would similarly require reduced spacing. This assumption was applied to staples because a withdrawal value is not available for staples in the NDS.

### Table 3.10A  Roof Sheathing Attachment Requirements for Wind Loads (7/16”, PANEL SG=0.50)

(Prescriptive Alternative to Table 3.10)

<table>
<thead>
<tr>
<th>Wind Speed 3-second gust (mph) (See Figure 1.1)</th>
<th>90</th>
<th>95</th>
<th>100</th>
<th>105</th>
<th>110</th>
<th>115</th>
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<td><strong>Sheathing Location</strong></td>
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<td><strong>Rafter/Truss Specific Gravity, G</strong></td>
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<td><strong>Maximum Nail Spacing for 8d Common Nails, or 10d Box Nails (inches, o.c.)</strong></td>
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<td>6</td>
<td>12</td>
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<tr>
<td>19.2</td>
<td>6</td>
<td>12</td>
<td>6</td>
<td>12</td>
<td>6</td>
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<td>6</td>
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<td>6</td>
<td>12</td>
<td>6</td>
<td>12</td>
<td>6</td>
<td>12</td>
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<td>6</td>
<td>12</td>
<td>6</td>
<td>12</td>
<td>6</td>
<td>12</td>
</tr>
</tbody>
</table>

**Exceeds capacity of 7/16" sheathing**

**Exceeds capacity of 7/16" sheathing**

| **Gable Endwall Rake or Rake Truss with up to 90° Rake Overhang** |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |
| 0.49                                                           |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |
| **Perimeter Edge Zone**                                        |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |
| 0.42                                                           |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |

**Exceeds capacity of 7/16" sheathing**

**Exceeds capacity of 7/16" sheathing**

| **Exposure B** |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |
|                |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |    |

**Board Sheathing**

<table>
<thead>
<tr>
<th>Sheathing Size</th>
<th>Rafter/Truss Spacing (in.)</th>
<th>Minimum Number of 8d Common Nails Per Support</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1×6 or 1×8 Sheathing</td>
<td>12×19.2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>1×10 or Larger Sheathing</td>
<td>12×19.2</td>
<td>3</td>
<td>3</td>
</tr>
</tbody>
</table>

**Notes:**
1. Nail spacing at panel edges (in.)
2. Nail spacing at intermediate supports in the panel field (in.)
3. For roof sheathing within 4 feet of the perimeter edge of the roof, including 4 feet on each side of the roof peak, the 4 foot perimeter edge zone attachment requirements shall be used.
4. For wind speeds greater than 130 mph, blocking is required which transfers lateral load to two additional joists (3 joists total).
5. See Table 3.10 for other fastener and sheathing combinations.
6. Tabulated values for 8d common and 10d box nails are applicable to carbon steel nails (bright or galvanized).
Cost Impact: The code change proposal will increase the cost of construction. The change in the spacing of the nails for the small portions of roofs will result in an increase in the cost due to a the increased number of nails and the time to install them, but that cost should be negligible when considering the overall cost for the construction of even a modest commercial building.
**2018 International Building Code**

Revise as follows:

<table>
<thead>
<tr>
<th>DESCRIPTION OF BUILDING ELEMENTS</th>
<th>NUMBER AND TYPE OF FASTENER</th>
<th>SPACING AND LOCATION</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Roof</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Blocking between ceiling joists, rafters or trusses to top plate or other framing below</td>
<td>4-8d box (2½&quot; x 0.113&quot;) or 3-8d common (2½&quot; x 0.131&quot;) or 3-10d box (3&quot; x 0.128&quot;) or 3-3° x 0.131&quot; nails or 3-3° 14 gage staples, 7/16&quot; crown</td>
<td>Each end, toenail</td>
</tr>
<tr>
<td>2. Ceiling joists to top plate</td>
<td>4-8d box (2½&quot; x 0.113&quot;) or 3-8d common (2½&quot; x 0.131&quot;) or 3-10d box (3&quot; x 0.128&quot;) or 3-3° x 0.131&quot; nails or 3-3° 14 gage staples, 7/16&quot; crown</td>
<td>Each joist, toenail</td>
</tr>
<tr>
<td>3. Ceiling joist not attached to parallel rafter, laps over partitions (no thrust) (see Section 2308.7.3.1, Table 2308.7.3.1)</td>
<td>3-16d common (3½&quot; x 0.162&quot;) or 4-10d box (3&quot; x 0.128&quot;) or 4-3° x 0.131&quot; nails or 4-3° 14 gage staples, 7/16&quot; crown</td>
<td>Face nail</td>
</tr>
<tr>
<td>4. Ceiling joist attached to parallel rafter (heal joint) (see Section 2308.7.3.1, Table 2308.7.3.1)</td>
<td>Per Table 2308.7.3.1</td>
<td>Face nail</td>
</tr>
<tr>
<td>5. Collar tie to rafter</td>
<td>3-10d common (3&quot; x 0.148&quot;) or 4-10d box (3&quot; x 0.128&quot;) or 4-3° x 0.131&quot; nails or 4-3° 14 gage staples, 7/16&quot; crown</td>
<td>Face nail</td>
</tr>
<tr>
<td>6. Rafter or roof truss to top plate (See Section 2308.7.5, Table 2308.7.5)</td>
<td>3-10d common (3&quot; x 0.148&quot;) or 3-16d box (3½&quot; x 0.135&quot;) or 4-10d box (3&quot; x 0.128&quot;) or 4-3° x 0.131&quot; nails or 4-3° 14 gage staples, 7/16&quot; crown</td>
<td>2 toenails on one side and 1 toenail on opposite side of rafter or truss ≤ 2 Toenails</td>
</tr>
<tr>
<td>7. Roof rafters to ridge valley or hip rafters; or roof rafter to 2-inch ridge beam</td>
<td>2-16d common (3½&quot; x 0.162&quot;) or 3-16d box (3½&quot; x 0.135&quot;) or 3-10d box (3&quot; x 0.128&quot;) or 3-3° x 0.131&quot; nails or 3-3° 14 gage staples, 7/16&quot; crown</td>
<td>End nail</td>
</tr>
<tr>
<td></td>
<td>3-10d common (3½&quot; x 0.148&quot;) or 4-16d box (3½&quot; x 0.135&quot;) or 4-10d box (3&quot; x 0.128&quot;) or 4-3° x 0.131&quot; nails or 4-3° 14 gage staples, 7/16&quot; crown</td>
<td>Toenail</td>
</tr>
<tr>
<td><strong>Wall</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8. Stud to stud (not at braced wall panels)</td>
<td>16d common (3½&quot; x 0.162&quot;)</td>
<td>24° o.c. face nail</td>
</tr>
<tr>
<td></td>
<td>10d box (3&quot; x 0.128&quot;) or 3° x 0.131&quot; nails or 3-3° 14 gage staples, 7/16&quot; crown</td>
<td>16° o.c. face nail</td>
</tr>
<tr>
<td>9. Stud to stud and abutting studs at intersecting wall corners (at braced wall panels)</td>
<td>16d common (3½&quot; x 0.162&quot;) or 16° o.c. face nail</td>
<td></td>
</tr>
<tr>
<td></td>
<td>16d box (3½&quot; x 0.135&quot;) or 12° o.c. face nail</td>
<td>3° x 0.131&quot; nails or 3-3° 14 gage staples, 7/16&quot; crown</td>
</tr>
</tbody>
</table>

Stainless Steel Fasteners are not applicable in this connection.
<table>
<thead>
<tr>
<th>Section</th>
<th>Fastener Details</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>10. Built-up header (2&quot; to 2&quot; header)</td>
<td>16d common (3½&quot; x 0.162’’); or 16d box (3½&quot; x 0.135’’)</td>
<td>16’ o.c. each edge, face nail</td>
</tr>
<tr>
<td></td>
<td>12’ o.c. each edge, face nail</td>
<td></td>
</tr>
<tr>
<td>11. Continuous header to stud</td>
<td>4-8d common (2½&quot; x 0.131’’); or 4-10d box (3” x 0.128’’); or 5-8d box (2½” x 0.113’’)</td>
<td>Toenail</td>
</tr>
<tr>
<td>12. Top plate to top plate</td>
<td>16d common (3½&quot; x 0.162’’); or 10d box (3” x 0.128’’); or 3” x 14 gage staples, 7/16” crown</td>
<td>16’ o.c. face nail</td>
</tr>
<tr>
<td></td>
<td>12’ o.c. face nail</td>
<td></td>
</tr>
<tr>
<td>13. Top plate to top plate, at end joints</td>
<td>8-16d common (3½&quot; x 0.162’’); or 12-16d box (3½” 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</td>
<td>Each side of end joint, face nail (minimum 24” lap splice length each side of end joint)</td>
</tr>
<tr>
<td>14. Bottom plate to joist, rim joist, band joist or blocking (not at braced wall panels)</td>
<td>16d common (3½&quot; x 0.162’’); or 16d box (3½&quot; x 0.135’’)</td>
<td>16’ o.c. face nail</td>
</tr>
<tr>
<td></td>
<td>12’ o.c. face nail</td>
<td></td>
</tr>
<tr>
<td>15. Bottom plate to joist, rim joist, band joist or blocking at braced wall panels</td>
<td>2-16d common (3½&quot; x 0.162’’); or 3-16d box (3½” x 0.135’’); or 4-3” x 0.131” nails; or 4-3” x 14 gage staples, 7/16” crown</td>
<td>16’ o.c. face nail</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16. Stud to top or bottom plate</td>
<td>3-16d box (3½” x 0.135’’); or 4-8d common (2½” x 0.131’’); or 4-10d box (3” x 0.128’’); or 4-3” x 0.131” nails; or 4-3” 14 gage staples, 7/16” crown; or 4-8d box (2½” x 0.113”); or 4-3” 14 gage staples, 7/16” crown</td>
<td>End nail</td>
</tr>
<tr>
<td>17. Top plates, laps at corners and intersections</td>
<td>2-16d common (3½” x 0.162’’); or 3-10d box (3” x 0.128’’); or 3-3” x 0.131” nails; or 3-3” x 14 gage staples, 7/16” crown</td>
<td>Face nail</td>
</tr>
<tr>
<td>18. 1” brace to each stud and plate</td>
<td>3-8d box (2½” x 0.113’’); or 2-8d common (2½” 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>Face nail</td>
</tr>
<tr>
<td></td>
<td>Stainless Steel Fasteners are not applicable in this connection</td>
<td></td>
</tr>
<tr>
<td>19. 1” x 6” sheathing to each bearing</td>
<td>3-8d box (2½” x 0.113’’); or 2-8d common (2½” x 0.131’’); or 2-10d box (3” x 0.128’’); or 2-1½” 16 gage staples, 1” crown</td>
<td>Face nail</td>
</tr>
<tr>
<td></td>
<td>Stainless Steel Fasteners are not applicable in this connection</td>
<td></td>
</tr>
<tr>
<td>20. 1” x 8” and wider sheathing to each bearing</td>
<td>3-8d common (2½” x 0.131’’); or 3-8d box (2½” x 0.113’’); or 4-2-10d box (3” x 0.128’’); or 3-1½” 16 gage staples, 1” crown</td>
<td>Wider than 1” x 8”</td>
</tr>
<tr>
<td></td>
<td>Stainless Steel Fasteners are not applicable in this connection</td>
<td></td>
</tr>
<tr>
<td>21. Joist to sill, top plate, or girder</td>
<td>4-8d box (2½” x 0.113’’); or 3-8d common (2½” x 0.131’’); or 4-10d box (3” x 0.128’’); or 3-3” x 0.131” nails; or 3-3” 14 gage staples, 7/16” crown</td>
<td>Toenail</td>
</tr>
<tr>
<td></td>
<td>12’ o.c. face nail</td>
<td></td>
</tr>
<tr>
<td>22. Rim joist, band joist, or blocking to top plate, sill or other framing below</td>
<td>4-8d box (2½” x 0.113’’); or 8d common (2½” 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>6’ o.c., toenail</td>
</tr>
</tbody>
</table>
### ICC COMMITTEE ACTION HEARINGS :: April, 2019

**23. 1” x 6” subfloor or less to each joist**

- 3-8d box (2\(\frac{1}{2}” \times 0.113\)); or 2-8d common (2\(\frac{1}{2}” \times 0.131\)); or 2-10d box (3” x 0.128); or 2-1\(\frac{1}{2}” \times 16\) 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 (3\(\frac{1}{2}” \times 0.135\)); or 2-16d common (3\(\frac{1}{2}” \times 0.162\))

**Blind and Face nail**

### 25. 2” planks (plank & beam – floor & roof)

- 3-16d box (3\(\frac{1}{2}” \times 0.135\)); or 2-16d common (3\(\frac{1}{2}” \times 0.162\))

**Each bearing, face nail**

### 26. Built-up girders and beams, 2” lumber layers

- 20d common (4” x 0.192”)

**32” O.C., face nail at top and bottom staggered on opposite sides**

- 10d box (3” x 0.128”); or 3” x 0.131” nails; or 3” 14 gage staples, \(\frac{1}{8}”\) crown

**24” O.C. face nail at top and bottom staggered on opposite sides**

**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, \(\frac{1}{8}”\) crown**

**Ends and at each splice, face nail**

### 27. Ledger strip supporting joists or rafter

- 3-16d common (3\(\frac{1}{2}” \times 0.162\)); or 4-16d box (3\(\frac{1}{2}” \times 0.135\)); or 4-10d box (3” x 0.128”); or 4-3” x 0.131” nails; or 4-3” 14 gage staples, \(\frac{1}{8}”\) crown

**Each joist or rafter, face nail**

### 28. Joist to band joist or rim joist

- 3-16d common (3\(\frac{1}{2}” \times 0.162\)); or 4-10d box (3” x 0.128”); or 4-3” x 0.131” nails; or 4-3” 14 gage staples, \(\frac{1}{8}”\) crown

**End nail**

### 29. Bridging or blocking to joist, rafter or truss

- 2-8d common (2\(\frac{1}{2}” \times 0.131\)); or 2-10d box (3” x 0.128”); or 2-3” x 0.131” nails; or 2-3” 14 gage staples, \(\frac{1}{8}”\) crown

**Each end, toenail**

### Wood structural panels (WSP), subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framing

<table>
<thead>
<tr>
<th>Thickness</th>
<th>Edges (inches)</th>
<th>Intermediate supports (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6d common or deformed (2” x 0.113”); or 2(\frac{1}{4}” \times 0.113”) nail (subfloor and wall)</td>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td>8d common or deformed (2(\frac{1}{2}” \times 0.131”) (roof) or RSRS-01 (2(\frac{1}{2}” \times 0.113”) nail (roof))</td>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td>2(\frac{1}{4}” \times 0.113”) nail (subfloor and wall)</td>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td>1(\frac{3}{8}”) 16 gage staple, (\frac{1}{8}”) crown (subfloor and wall)</td>
<td>4</td>
<td>8</td>
</tr>
<tr>
<td>2(\frac{3}{8}” \times 0.113”) nail (roof)</td>
<td>4</td>
<td>8</td>
</tr>
<tr>
<td>1(\frac{3}{8}”) 16 gage staple, (\frac{1}{8}”) crown (roof)</td>
<td>3</td>
<td>6</td>
</tr>
<tr>
<td>31. (\frac{19}{32}” - \frac{3}{4}”)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8d common (2(\frac{1}{2}” \times 0.131”))</td>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td>8d common or deformed (2(\frac{1}{2}” \times 0.131”)) (roof) or RSRS-01 (2(\frac{3}{8}” \times 0.113”)) nail (roof)</td>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td>2(\frac{3}{8}” \times 0.113”) nail; or 2” 16 gage staple, (\frac{1}{8}”) crown</td>
<td>4</td>
<td>8</td>
</tr>
<tr>
<td>32. (\frac{7}{8}” - 1\frac{1}{4}”)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10d common (3” x 0.148”); or 8d deformed (2(\frac{1}{2}” \times 0.131”))</td>
<td>6</td>
<td>12</td>
</tr>
</tbody>
</table>

### Other exterior wall sheathing

<table>
<thead>
<tr>
<th>Material</th>
<th>Edges (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1(\frac{1}{2}”) x 0.120”, galvanized roofing nail ((\frac{1}{8}”) head diameter); or 1(\frac{1}{4}”) 16 gage staple with (\frac{1}{8}”) or 1” crown</td>
<td>3</td>
</tr>
<tr>
<td>1(\frac{9}{32}”) x 0.120”, galvanized roofing nail ((\frac{1}{8}”) diameter head); or 1(\frac{1}{2}”) 16 gage staple with (\frac{1}{8}”) or 1” crown</td>
<td>3</td>
</tr>
</tbody>
</table>

### Wood structural panels, combination subfloor underlayment to framing

<table>
<thead>
<tr>
<th>Thickness</th>
<th>Edges (inches)</th>
<th>Intermediate supports (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>35. (\frac{3}{4}”) and less</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8d common (2(\frac{1}{2}” \times 0.131”)); or 6d deformed (2” x 0.113”) or deformed 2&quot; x 0.120”</td>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td>36. (\frac{7}{8}” - 1”)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8d common (2(\frac{1}{2}” \times 0.131”)); or 6d deformed (2(\frac{1}{2}” \times 0.131”)); or deformed 2(\frac{1}{2}” \times 0.120”)</td>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td>37. (1\frac{1}{4}” - 1\frac{1}{4}”)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10d common (3” x 0.148”); or 8d deformed (2(\frac{1}{2}” \times 0.131”)); or deformed 2(\frac{1}{2}” \times 0.120”)</td>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td>Connection</td>
<td>Details</td>
<td></td>
</tr>
<tr>
<td>------------</td>
<td>---------</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Added 8d box nails to match IRC R602.3(1)</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Added note regarding stainless steel fasteners</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Added 8d box nails from IRC R602.3(1)</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Added note regarding stainless steel fasteners</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Changed Fastener Spacing and Location note to match IRC R602.3(1)</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Added 16d Box nails to match IRC R602.3(1)</td>
<td></td>
</tr>
</tbody>
</table>

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 and 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.

### Nailing Schedule

#### Wood structural panels (WSP), subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framing

<table>
<thead>
<tr>
<th>Edges (inches)</th>
<th>Intermediate supports (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td>6</td>
<td>12</td>
</tr>
</tbody>
</table>

#### Interior paneling

<table>
<thead>
<tr>
<th>Edges (inches)</th>
<th>Intermediate supports (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td>6</td>
<td>12</td>
</tr>
</tbody>
</table>
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 deformed (2" x 0.113")

**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 deformed (2½" x 0.131")

**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

Proposal # 3953

S174-19
2018 International Building Code

Revise as follows:

2304.12.1 Locations requiring waterborne preservatives or naturally durable wood. Wood used above ground in the locations specified in Sections 2304.12.1.1 through 2304.12.1.5, 2304.12.3 and 2304.12.5 shall be naturally durable wood or preservative-treated wood using waterborne preservatives, in accordance with AWPA U1 for above-ground use.

2304.12.1.1 Joists, girders and subfloor. Wood joists or wood structural floors that are closer than 18 inches (457 mm) or wood girders that are closer than 12 inches (305 mm) to the exposed ground in crawl spaces or unexcavated areas located within the perimeter of the building foundation shall be of naturally durable or preservative-treated wood.

2304.12.1.2 Wood supported by exterior foundation walls. Wood framing members, including wood sheathing, that are in contact with exterior foundation walls and are less than 8 inches (203 mm) from exposed earth shall be of naturally durable or preservative-treated wood.

2304.12.1.3 Exterior walls below grade. Wood framing members and furring strips in direct contact with the interior of exterior masonry or concrete walls below grade shall be of naturally durable or preservative-treated wood.

2304.12.1.4 Sleepers and sills. Sleepers and sills on a concrete or masonry slab that is in direct contact with earth shall be of naturally durable or preservative-treated wood.

2304.12.1.5 Wood siding. Clearance between wood siding and earth on the exterior of a building shall be not less than 6 inches (152 mm) or less than 2 inches (51 mm) vertical from concrete steps, porch slabs, patio slabs and similar horizontal surfaces exposed to the weather except where siding, sheathing and wall framing are of naturally durable or preservative-treated wood.

2304.12.2 Other locations. Wood used in the locations specified in Sections 2304.12.2.1 through 2304.12.2.5 shall be naturally durable wood or preservative-treated wood in accordance with AWPA U1. Preservative-treated wood used in interior locations shall be protected with two coats of urethane, shellac, latex epoxy or varnish unless waterborne preservatives are used. Prior to application of the protective finish, the wood shall be dried in accordance with the manufacturer’s recommendations.

2304.12.2.1 Girder ends. The ends of wood girders entering exterior masonry or concrete walls shall be provided with a 1/2-inch (12.7 mm) airspace on top, sides and end, unless naturally durable or preservative-treated wood is used.

2304.12.2.2 Posts or columns. Posts or columns supporting permanent structures and supported by a concrete or masonry slab or footing that is in direct contact with the earth shall be of naturally durable or preservative-treated wood.

Exception: Posts or columns that meet all of the following:

1. Are not exposed to the weather, or are protected by a roof, eave, overhang, or other covering if exposed to the weather.
2. Are supported by concrete piers or metal pedestals projected not less than 1 inch (25 mm) above the slab or deck and are separated from the concrete pier by an impervious moisture barrier.
3. Are located not less than 8 inches (203 mm) above exposed earth.

2304.12.2.3 Supporting member for permanent appurtenances. Naturally durable or preservative-treated wood shall be utilized for those portions of wood members that form the structural supports of buildings, balconies, porches or similar permanent building appurtenances where such members are exposed to the weather without adequate protection from a roof, eave, overhang or other covering to prevent moisture or water accumulation on the surface or at joints between members.

Exception: Buildings located in a geographical region where experience has demonstrated that climatic conditions preclude the need to use durable materials where the structure is exposed to the weather.

2304.12.2.4 Laminated timbers. The portions of glued-laminated timbers that form the structural supports of a building or other structure and are exposed to weather and not fully protected from moisture by a roof, eave or similar covering shall be pressure treated with preservative or be manufactured from naturally durable or preservative-treated wood.

2304.12.2.5 Supporting members for permeable floors and roofs. Wood structural members that support moisture-permeable floors or roofs that are exposed to the weather, such as concrete or masonry slabs, shall be of naturally durable or preservative-treated wood unless separated from such floors or roofs by an impervious moisture barrier. The impervious moisture barrier system protecting the structure supporting floors shall provide positive drainage of water that infiltrates the moisture-permeable floor topping.
2304.12.2.6 Ventilation beneath balcony or elevated walking surfaces. Enclosed framing in exterior balconies and elevated walking surfaces that are exposed to rain, snow or drainage from irrigation shall be provided with openings that provide a net free cross-ventilation area not less than \( \frac{1}{150} \) of the area of each separate space.

2304.12.3 Wood in contact with the ground or fresh water. Wood used in contact with exposed earth shall be naturally durable for both decay and termite resistance or preservative treated in accordance with AWPA U1 for soil or fresh water use.

   Exception: Untreated wood is permitted where such wood is continuously and entirely below the ground-water level or submerged in fresh water.

2304.12.3.1 Posts or columns. Posts and columns that are supporting permanent structures and embedded in concrete that is exposed to the weather or in direct contact with the earth shall be of preservative-treated wood.

2304.12.4 Termite protection. In geographical areas where hazard of termite damage is known to be very heavy, wood floor framing in the locations specified in Section 2304.12.1.1 and exposed framing of exterior decks or balconies shall be of naturally durable species (termite resistant) or preservative treated in accordance with AWPA U1 for the species, product preservative and end use or provided with approved methods of termite protection.

2304.12.5 Wood used in retaining walls and cribs. Wood installed in retaining or crib walls shall be preservative treated in accordance with AWPA U1 for soil and fresh water use.

Reason: The only change is to Section 2304.12.1. The other sections are shown for context only. References to sections 2304.12.3 and 2304.12.5 for locations requiring waterborne preservatives are unnecessary. AWPA has several oil-borne preservatives approved for use in ground contact applications (UC4A and higher) that could be used in these locations. When used in interior locations, Section 2304.12.2 requires them to be protected with two coats of urethane, shellac, latex epoxy or varnish when not treated with waterborne preservative.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
Any cost differences depend on the choice of products for their application; the proposal adds flexibility and therefore may decrease cost in some circumstances.
2018 International Building Code

Revise as follows:

**2304.12.2.3 Supporting member for permanent appurtenances.** Naturally durable or preservative-treated wood shall be utilized for those portions of wood members that form the structural supports of buildings, balconies, porches or similar permanent building appurtenances where such members are exposed to the weather without adequate protection from a roof, eave, overhang or other covering to prevent moisture or water accumulation on the surface or at joints between members.

**Exception:** Buildings sawn lumber in buildings located in a geographical region where experience has demonstrated that climatic conditions preclude the need to use durable materials where the structure is exposed to the weather.

Delete without substitution:

**2304.12.2.4 Laminated timbers.** The portions of glued-laminated timbers that form the structural supports of a building or other structure and are exposed to weather and not fully protected from moisture by a roof, eave or similar covering shall be pressure treated with preservative or be manufactured from naturally durable or preservative-treated wood.

Reason: Having a separate section for laminated timbers is unnecessary since they are required to be protected as for all other wood members in the locations described in the subsections of 2304.12.1 (locations requiring waterborne preservatives or naturally durable wood) and 2304.12.2 (other locations). Currently 2304.12.2.3 and 2304.12.2.4 duplicate each other except for the exception in 2304.12.2.3, which does not apply to laminated timber and presumable should not apply to any engineered wood product using adhesives. Therefore, the proposed modification of the exception to 2304.12.2.3 to exclude engineered wood products makes the separate section on laminated timbers unnecessary. This will also solve the problem of interpreting the current code as prohibiting glued-laminated timbers from being used in the locations described in 2304.12.1. Glue-laminated timber can be used in those locations as long as they are treated with water-borne preservatives or protected in accordance with 2304.12.2 for oil-borne preservatives used in interior locations.

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

Manufacturers of engineered wood products typically recommend their protection when exposed to the weather, regardless of climate or geographic location. The current code requirements are retained for sawn lumber and glued laminated timber specifically. Therefore there is no anticipated cost increase or decrease.
**2018 International Building Code**

Revise as follows:

**2304.12.2 Other locations.** Wood used in the locations specified in Sections 2304.12.2.1 through 2304.12.2.9 shall be naturally durable wood or preservative-treated wood in accordance with AWPA U1. Preservative-treated wood used in interior locations shall be protected with two coats of urethane, shellac, latex epoxy or varnish unless waterborne preservatives are used. Prior to application of the protective finish, the wood shall be dried in accordance with the manufacturer’s recommendations.

**2304.12.2.1 Girder ends.** The ends of wood girders entering exterior masonry or concrete walls shall be provided with a 1/2-inch (12.7 mm) airspace on top, sides and end, unless naturally durable or preservative-treated wood is used.

**2304.12.2.2 Posts or columns.** Posts or columns supporting permanent structures and supported by a concrete or masonry slab or footing that is in direct contact with the earth shall be of naturally durable or preservative-treated wood.

**Exception:** Posts or columns that meet all of the following:

1. Are not exposed to the weather, or are protected by a roof, eave, overhang, or other covering if exposed to the weather.
2. Are supported by concrete piers or metal pedestals projected not less than 1 inch (25 mm) above the slab or deck and are separated from the concrete pier by an impervious moisture barrier.
3. Are located not less than 8 inches (203 mm) above exposed earth.

**2304.12.2.3 Supporting member for permanent appurtenances.** Naturally durable or preservative-treated wood shall be utilized for those portions of wood members that form the structural supports of buildings, balconies, porches or similar permanent building appurtenances where such members are exposed to the weather without adequate protection from a roof, eave, overhang or other covering to prevent moisture or water accumulation on the surface or at joints between members.

**Exception:** Buildings located in a geographical region where experience has demonstrated that climatic conditions preclude the need to use durable materials where the structure is exposed to the weather.

**2304.12.2.4 Laminated timbers.** The portions of glued-laminated timbers that form the structural supports of a building or other structure and are exposed to weather and not fully protected from moisture by a roof, eave or similar covering shall be pressure treated with preservative or be manufactured from naturally durable or preservative-treated wood.

**2304.12.2.5 Supporting members for permeable floors and roofs.** Wood structural members that support moisture-permeable floors or roofs that are exposed to the weather, such as concrete or masonry slabs, shall be of naturally durable or preservative-treated wood unless separated from such floors or roofs by an impervious moisture barrier. The impervious moisture barrier system protecting the structure supporting floors shall provide positive drainage of water that infiltrates the moisture-permeable floor topping.

**2304.12.2.6 Ventilation beneath balcony or elevated walking surfaces.** Enclosed framing in exterior balconies and elevated walking surfaces that are exposed to rain, snow or drainage from irrigation shall be provided with openings that provide a net free cross-ventilation area not less than 1/150 of the area of each separate space.

**2304.12.3 Wood in contact with the ground or fresh water.** Wood used in contact with exposed earth shall be naturally durable for both decay and termite resistance or preservative treated in accordance with AWPA U1 for soil or fresh water use.

**Exception:** Untreated wood is permitted where such wood is continuously and entirely below the ground-water level or submerged in fresh water.

**2304.12.4 Posts or columns.** Posts and columns that are supporting permanent structures and embedded in concrete that is exposed to the weather or in direct contact with the earth shall be of preservative-treated wood.

**2304.12.5 Termite protection.** In geographical areas where hazard of termite damage is known to be very heavy, wood floor framing in the locations specified in Section 2304.12.1 and exposed framing of exterior decks or balconies shall be of naturally durable species (termite resistant) or preservative treated in accordance with AWPA U1 for the species, product preservative and end use or provided with approved methods of termite protection.

**2304.12.6 Wood used in retaining walls and cribs.** Wood installed in retaining or crib walls shall be preservative treated in accordance with AWPA U1 for soil and fresh water use.
2304.12.6 Attic ventilation. For attic ventilation, see Section 1202.2.2.

2304.12.7 Under-floor ventilation (crawl space). For under-floor ventilation (crawl space), see Section 1202.4.

Reason: This is a proposed renumbering of the subsections without changing the content. There does not seem to be any reason why current subsections 2304.12.3 through 2304.12.5 (Wood in contact with the ground or fresh water, posts and columns embedded in concrete in direct contact with the earth, wood needing termite protection, and retaining walls and crib walls) are not subsections of 2304.12.2 as locations requiring protection. In addition, there seems to be no reason why oil-borne treatments used in these locations should not also be protected as prescribed in 2304.12.2 when used in interior locations. This renumbering cleans up the section and brings appropriate provisions under the charging language of 2304.12.2 for that protection.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. This is a renumbering of subsections only and does not affect the normal application of the code, since the protection provisions of 2304.12.2 are broad and not necessarily limited to only locations listed in the current subsections of that section.
Rebecca Baker, Jefferson County CO, representing the Colorado Chapter ICC (bbaker@co.jefferson.co.us)

2018 International Building Code

Revise as follows:

2304.12.2.6 Ventilation beneath balcony or elevated walking surfaces. Enclosed framing in exterior balconies and elevated walking surfaces that are exposed to rain, snow or drainage from irrigation have weather-exposed surfaces shall be provided with openings that provide a net free cross-ventilation area not less than \( \frac{1}{150} \) of the area of each separate space.

Reason: The term irrigation was added to the 2018 and goes beyond the scope of previous editions of the code. To verify compliance, landscape irrigation plans would need to become part of the construction documents. The proposed language uses a defined terms which will increase consistency and satisfy the intent.

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

This proposal will help standardize the application of the code.
2018 International Building Code

Revise as follows:

2306.1 Allowable stress design. The design and construction of wood elements in structures using allowable stress design shall be in accordance with the following applicable standards:

American Society of Agricultural and Biological Engineers.

ASABE S618 Post Frame Building System Nomenclature

(Portions of standards not shown remain unchanged)

Add new text as follows:

S618 DEC2010 (R2016): Post Frame Building System Nomenclature

Reason: Post frame construction continues to grow in popularity. Design guidance is provided through the ASABE Engineering Practices identified in Section 2306.1. A number of the terms used in these engineering practices and in post frame design overall are specific to the industry and a clear understanding of terms is critical. This standard provides that clarity of terms through a combination of text and figures for all aspects of post frame construction. This proposal adds reference to ASABE S618 in Section 2306.1 and in Chapter 35.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. This standard provides definitions and figures showing the critical elements of post frame construction. There are no procedures or additional design considerations included in this document.

Staff Analysis: A review of the standard proposed for inclusion in the code, ASABE S618 Dec2010 (R2016), 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.

Proposal # 4743

American Society of Agricultural and Biological Engineers
2950 Niles Road
St. Joseph MI 49085
S180-19

IBC®: TABLE 2306.1.4

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

2018 International Building Code

Revise as follows:

TABLE 2306.1.4

ALLOWABLE LOADS FOR LUMBER DECKING

Portions of table not shown remain unchanged.

<table>
<thead>
<tr>
<th>PATTERN</th>
<th>ALLOWABLE AREA LOAD(a, b)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Flexure</td>
</tr>
<tr>
<td>3-inch and 4-inch decking</td>
<td>(\sigma_b = \frac{8}{3} \frac{2F_{b}'}{d^2} )</td>
</tr>
</tbody>
</table>

For SI: 1 inch = 25.4 mm.

- a. \(\sigma_b\) = Allowable total uniform load limited by bending.
- b. \(\sigma_d\) = Allowable total uniform load limited by deflection.
- \(d\) = Actual decking thickness.
- \(l\) = Span of decking.

\(F_{b}'\) = Allowable bending stress adjusted by applicable factors.

\(E\) = Modulus of elasticity adjusted by applicable factors.

**Reason:** Correct the flexure equation for 3-inch and 4-inch decking to be consistent with lumber decking design documents WCD2 and AITC 112. The equation in its current form was introduced through code change S170-04/05. This equation erroneously incorporated a 2/3 factor applied to the moment of inertia used for controlled random layup patterns of 2-inch and mechanically laminated decking, rather than the 0.8 factor specified in WCD2 and AITC 112 for controlled random layup patterns of 3-inch and 4-inch decking. The revised equation incorporates a 0.8 factor in the flexure equation, consistent with use of the 0.8 factor implemented in the existing deflection equation. The equations used for flexure and deflection for controlled random layup decking assumes a 3 equal-span uniformly loaded baseline condition modified by either the 2/3 or 0.8 adjustment factors described above.

**Bibliography:**

**Cost Impact:** The code change proposal will decrease the cost of construction. This correction to the flexure-based allowable load limit for 3-inch and 4-inch controlled random layup decking will decrease the cost of construction because it will result in higher allowable loads than the incorrect equation that is currently provided within this particular cell of Table 2306.1.4.
**2018 International Building Code**

Revise as follows:

**2306.1.4 Lumber decking.** The capacity of lumber decking arranged according to the patterns described in Section 2304.9.2 shall be the lesser of the capacities determined for flexure moment and deflection according to the formulas in Table 2306.1.4.

**TABLE 2306.1.4**

**ALLOWABLE LOADS FOR LUMBER DECKING**

<table>
<thead>
<tr>
<th>PATTERN</th>
<th>ALLOWABLE AREA LOAD&lt;sup&gt;*&lt;/sup&gt;</th>
<th>Moment Flexure</th>
<th>Deflection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Simple span</td>
<td>$\sigma_b \cdot \frac{20F_p d^2}{3I^2}$</td>
<td>$\sigma_b \cdot \frac{384AE'd^4}{5I^4}$</td>
<td>$\frac{185AE'd^4}{I^4}$</td>
</tr>
<tr>
<td>Two-span continuous</td>
<td>$\sigma_b \cdot \frac{20F_p d^2}{3I^2}$</td>
<td>$\sigma_b \cdot \frac{131AE'd^4}{I^4}$</td>
<td>$\frac{105AE'd^4}{I^4}$</td>
</tr>
<tr>
<td>Combination simple- and two-span continuous</td>
<td>$\sigma_b \cdot \frac{20F_p d^2}{3I^2}$</td>
<td>$\sigma_b \cdot \frac{100AE'd^4}{I^4}$</td>
<td>$\frac{116AE'd^4}{I^4}$</td>
</tr>
<tr>
<td>Cantilevered pieces intermixed</td>
<td>$\sigma_b \cdot \frac{20F_p d^2}{3I^2}$</td>
<td>$\sigma_b \cdot \frac{100AE'd^4}{I^4}$</td>
<td>$\frac{116AE'd^4}{I^4}$</td>
</tr>
<tr>
<td>Controlled random layup</td>
<td>$\sigma_b \cdot \frac{20F_p d^2}{3I^2}$</td>
<td>$\sigma_b \cdot \frac{100AE'd^4}{I^4}$</td>
<td>$\frac{116AE'd^4}{I^4}$</td>
</tr>
<tr>
<td>Mechanically laminated decking</td>
<td>$\sigma_b \cdot \frac{20F_p d^2}{3I^2}$</td>
<td>$\sigma_b \cdot \frac{100AE'd^4}{I^4}$</td>
<td>$\frac{116AE'd^4}{I^4}$</td>
</tr>
<tr>
<td>2-inch decking</td>
<td>$\sigma_b \cdot \frac{20F_p d^2}{3I^2}$</td>
<td>$\sigma_b \cdot \frac{100AE'd^4}{I^4}$</td>
<td>$\frac{116AE'd^4}{I^4}$</td>
</tr>
<tr>
<td>3-inch and 4-inch decking</td>
<td>$\sigma_b \cdot \frac{20F_p d^2}{3I^2}$</td>
<td>$\sigma_b \cdot \frac{100AE'd^4}{I^4}$</td>
<td>$\frac{116AE'd^4}{I^4}$</td>
</tr>
</tbody>
</table>

For SI: 1 inch = 25.4 mm.

a. $\sigma_w = $ Allowable total uniform load limited by bending moment.

b. $\sigma_w = $ Allowable total uniform load limited by deflection.

d = Actual decking thickness.

I = Span of decking.

$F_p = $ Allowable bending stress adjusted by applicable factors.

$E = $ Modulus of elasticity adjusted by applicable factors.

**Reason:** Notation for allowable uniform load is changed from the Greek letter sigma to "w" in order to match notation more commonly used to express uniform loads. Also, the term "flexure" is changed to "moment" to more accurately express the moment capacity basis of this limit and distinguish it from the deflection-based limit. The term moment also more clearly addresses the basis of the equations in lieu of terms bending and flexure which might be incorrectly construed as also addressing shear in members subject to bending and flexure. Definitions for all notation are combined under Footnote a as there does not appear to be a need to divide the listing of notations into two separate footnotes.

**Cost Impact:** The code change proposal will not increase or decrease the cost of construction. These revisions to the notation and terminology provided in Section 2306.1.4 are for clarification only, and do not affect the cost of construction.
**Proponent:** 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>
<thead>
<tr>
<th>TYPE OF MATERIAL</th>
<th>THICKNESS OF MATERIAL</th>
<th>WALL CONSTRUCTION</th>
<th>STAPLE SPACING</th>
<th>SHEAR VALUE</th>
<th>MINIMUM STAPLE SIZE</th>
</tr>
</thead>
<tbody>
<tr>
<td>4. Gypsum board, gypsum veneer base or water-resistant gypsum backing board</td>
<td>5/8&quot;</td>
<td>Unblocked</td>
<td>7</td>
<td>75</td>
<td>No. 16 gage galv. staple, 1 1/2&quot; long</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Unblocked</td>
<td>4</td>
<td>110</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Unblocked</td>
<td>7</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Unblocked</td>
<td>4</td>
<td>125</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Blocked</td>
<td>7</td>
<td>125</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Blocked</td>
<td>4</td>
<td>150</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5/8&quot;</td>
<td>Unblocked</td>
<td>7</td>
<td>115</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Blocked</td>
<td>4</td>
<td>145</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Blocked</td>
<td>7</td>
<td>145</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Blocked Two-ply</td>
<td>Base ply: 9 Face ply: 7</td>
<td>250</td>
<td></td>
</tr>
</tbody>
</table>

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per foot = 14.5939 N/m.

- a. These shear walls shall not be used to resist loads imposed by masonry or concrete walls (see AWC SDPWS). Values shown are for short-term loading due to wind or seismic loading. Walls resisting seismic loads shall be subject to the limitations in Section 12.2.1 of ASCE 7. Values shown shall be reduced 25 percent for normal loading.
- b. Applies to fastening at studs, top and bottom plates and blocking.
- c. Except as noted, shear values are based on a maximum framing spacing of 16 inches on center.
- d. Maximum framing spacing of 24 inches on center.
- e. All edges are blocked, and edge fastening is provided at all supports and all panel edges.
- f. Staples shall have a minimum crown width of 7/16 inch, measured outside the legs, and shall be installed with their crowns parallel to the long dimension of the framing members.
- g. Staples for the attachment of gypsum lath and woven-wire lath shall have a minimum crown width of 5/4 inch, measured outside the legs.

**Reason:** Material Type 4 Gypsum board, gypsum veneer base or water-resistant gypsum backing board
5/8" thick Unblocked and Blocked

There are 2 staple lengths shown. The 1 5/8" will be the most conservative length.

**Cost Impact:** The code change proposal will not increase or decrease the cost of construction
There is no cost impact. There were two fastener lengths shown and the shorter one was removed.
2018 International Building Code

Revise as follows:

2308.5.6 Cripple walls. Foundation cripple walls shall be framed of studs that are not less than the size of the studding above and . Exterior cripple wall studs shall be not less than 14 inches (356 mm) in length, or shall be framed of solid blocking. Where exceeding 4 feet (1219 mm) in height, such walls shall be framed of studs having the size required for an additional story. See Section 2308.6.6 for cripple wall bracing.

Proposal # 4089
S183-19 Part II
IRC®: R602.9

Proponent: Robert Rice C.B.O., representing Southern Oregon Chapter-ICC (RobertR@nwcodepros.com)

2018 International Residential Code
Revise as follows:

R602.9 Cripple walls. Foundation cripple walls shall be framed of studs not smaller than the studding above. Where exceeding 4 feet (1219 mm) in height, such walls shall be framed of studs having the size required for an additional story. Cripple walls shall be supported on continuous foundations.

Exterior cripple walls with a stud height less than 14 inches (356 mm) shall be continuously sheathed on one side with wood structural panels fastened to both the top and bottom plates in accordance with Table R602.3(1), or the cripple walls shall be constructed of solid blocking.

Cripple walls shall be supported on continuous foundations.

Reason: Both the IRC and the IBC require foundation cripple wall studs less than 14 inches to be "continuously-sheathed". (Note: This requirement is not related to wall bracing which is covered elsewhere in the codes.) Per the Code Commentaries, the requirement for continuous sheathing on cripple wall studs 14 inches and less (or solid-blocking) is intended to ensure the integrity of the studs when nails are end-nailed into the studs (face-nailed through the top and bottom plates). The IRC Commentary states;

"The minimum length of 14 inches (356 mm) for cripple wall studs provides sufficient clear space for required nailing of the framing."

In regions with shallow frost-depth it is common to have shallow crawlspaces. With an 18 inch minimum crawlspace, an interior cripple wall on a continuous 6 inch thick concrete footing is as short as 12 inches. With a top and bottom plate, the studs would be as short as 9 inches.

Field observations have shown that these short studs typically do not present a problem when installed in this situation. Continuously-sheathing these short walls in a crawlspace is costly and very labor-intensive without justifiable benefit. It can also present additional adverse effects with ventilation, under-floor mechanical systems, plumbing and access. There are other effective methods of connecting the studs to the top and bottom plates such as toe-nailing, mechanical fasteners and plywood gussets. The fact is, poorly installed/damaged studs could, and should, be replaced whether in a full-height wall or in a cripple wall and the inspector is qualified to make that determination and has the authority to require correction of defective materials.

Also, cripple wall studs are used in other places in the code for load-bearing application such as above/below window and door headers (e.g. See IRC Figures R602.3(2) and R602.7.1). This existing limitation of 14 inches does not apply to those conditions even though they are also subject to gravity loads which in some cases may be significantly greater (e.g. long-span pre-engineered trusses) than interior cripple walls under the floor. In those cases, an inspector would be competent enough, and authorized, to require replacement of any split or damaged cripple studs if they did exist. It makes sense that the same would apply to cripple walls in the floor system.

In addition, it needs to be clarified that the continuous sheathing mentioned in this section is not for "wall bracing" even though it is often misunderstood to be so. The additional sentence that is added to the IRC makes this clear by referencing section R602.10.10 for the cripple wall bracing requirements. The IBC section already has a sentence that references the wall bracing section.

These pictures are indicative of typical crawl-space construction in areas of minimal (e.g. 12 inch) frost depth.
Cost Impact: The code change proposal will decrease the cost of construction.

This code proposal eliminates the requirement to apply sheathing (e.g. OSB, Plywood, etc.) to interior foundation cripple walls which commonly occur in shallow crawlspaces. Therefore, there is a savings on the material and labor costs to install the sheathing that is currently required in these code sections.
S184-19 Part I

PART I — IBC®: 2308.5.9, 2308.5.10; IFGC®: [BS] 302.3.3; IPC®: [BS] C101.2, [BS] C101.3

PART II — IRC®: R602.6

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

2018 International Building Code

Revise as follows:

2308.5.9 Cutting and notching. In exterior walls and bearing partitions, a wood stud shall not be cut or notched to a depth not exceeding 25 percent of the width of the stud. Cutting or notching of studs to a depth not greater than 40 percent of the width of the stud is permitted in nonbearing partitions not supporting in excess of 25 percent of its depth. In nonbearing partitions that do not support loads other than the weight of the partition, a stud shall not be cut or notched in excess of 40 percent of its depth.

2308.5.10 Bored holes. Bored holes not greater than 40 percent of the stud width are permitted to be bored in any wood stud. Bored holes not greater than 60 percent of the stud width are permitted in nonbearing partitions or depth in nonbearing partitions. The diameter of bored holes in wood studs shall not exceed 60 percent of the stud depth in any wall where each bored stud is doubled, provided that not more than two such successive doubled studs are so bored. The edge of the bored hole shall not be nearer than 5/8 inch (15.9 mm) to the edge of the stud. Bored holes shall not be located at the same section of stud as a cut or notch.

2018 International Fuel Gas Code

[BS] 302.3.3 Stud cutting and notching. In exterior walls and bearing partitions, any wood stud is permitted to be cut or notched to a depth not exceeding 25 percent of the stud width. Cutting or notching of studs to a depth not greater than 40 percent of the width of the stud is permitted in nonbearing partitions supporting no loads other than the weight of the partition. In nonbearing partitions that do not support loads other than the weight of the partition, a stud shall not be cut or notched in excess of 40 percent of its depth.

2018 International Plumbing Code

[BS] C101.2 Stud cutting and notching. In exterior walls and bearing partitions, any wood stud is permitted to be cut or notched to a depth not exceeding 25 percent of the stud width. Cutting or notching of studs to a depth not greater than 40 percent of the width of the stud is permitted in nonbearing partitions supporting no loads other than the weight of the partition. In nonbearing partitions that do not support loads other than the weight of the partition, a stud shall not be cut or notched in excess of 40 percent of its depth.

[BS] C101.3 Bored holes. The diameter of bored holes in wood studs shall not exceed 40 percent of the stud depth. The diameter of bored holes in wood studs shall not exceed 60 percent of the stud depth in any wall where each stud is doubled, provided that not more than two such successive doubled studs are so bored. The edge of the bored hole shall be not be closer than 5/8 inch (15.9 mm) to the edge of the stud. Bored holes shall not be located at the same section of stud as a cut or notch.

Proposal # 4065
R602.6 Drilling and notching of studs. Drilling and notching of studs shall be in accordance with the following:

1. **Notching.** Any stud in an exterior wall or bearing partition shall be permitted to not be cut or notched to a depth not exceeding 25 percent of its **width**. Studs in nonbearing partitions shall be permitted to not be notched to a depth not exceeding 40 percent of a single stud **width**. Studs in nonbearing partitions shall be permitted to not be notched to a depth not exceeding 40 percent of a single stud **depth**.

2. **Drilling.** Any stud shall be permitted to be bored or drilled, provided that the diameter of the resulting hole is not more than 60 percent of the stud **width**. The diameter of bored holes in studs shall not exceed 60 percent of the stud **width**. The edge of the hole is shall not more be less than 5/8 inch (16 mm) to from the edge of the stud, and the hole is shall not be located in the same section as a cut or notch. Studs Where the diameter of a bored hole in a stud located in exterior walls or bearing partitions drilled is over 40 percent and up to 60 percent such stud shall be doubled with and not more than two successive doubled studs shall be so bored. See Figures R602.6(1) and R602.6(2).

**Exception:** Use of approved stud shoes is permitted where they are installed in accordance with the manufacturer's recommendations.

**Reason:** The current text uses the word width, when actually it is the depth that is meant. The depth of a stud is the plane in which a hole is bored. Holes are not bored in the width (1 1/2 inches) of a stud. This revision also gets rid of unenforceable permissive language. The current text says that any stud is permitted to be notched to a depth not exceeding 25%. This is stating a permitted limit; not a mandatory limit. A highway speed limit is not permitted to be 55 miles per hour, rather it is an absolute limit of 55. If the stud is permitted to be notched to not exceed 25%, then it also permitted to be notched to not exceed other percentages. Lastly, this proposal corrects a flaw where the text said that the edge of the hole cannot be more than 5/8 inch to the edge of the stud. The intent is exactly the opposite. The edge of the hole must not be less than 5/8 inch to the edge of the stud. This language currently exists in the IMC. Similar changes are being proposed in the IBC, IFGC and IPC to coordinate the terms between I-codes. 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/.

**Cost Impact:** The code change proposal will not increase or decrease the cost of construction. This proposal will not increase the cost of construction as it is a clarification of terms and current requirements only.
S185-19
IBC®: 2308.6.6.2

**Proponent:** 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); Julie Furr, Rimkus Consulting Group, representing Federal Emergency Management Agency and Applied Technology Council Seismic Code Support Committee (FEMA/ATC SCSC) (jfurr@rimkus.com); Michael Mahoney, representing Federal Emergency Management Agency (mike.mahoney@fema.dhs.gov)

**2018 International Building Code**

Revise as follows:

2308.6.6.2 Cripple wall bracing in Seismic Design Categories D and E. For the purposes of this section, cripple walls in Seismic Design Categories D and E having shall not have a stud height exceeding 14 inches (356 mm) shall be considered to be a story and, and studs shall be braced solid blocked in accordance with Table 2308.6.1. Where interior braced wall lines occur without a continuous foundation below, the length of parallel exterior cripple wall bracing shall be one and one half times the lengths required by Table 2308.6.1. Where the cripple wall sheathing type used is Method WSP or DWB and this additional length of bracing cannot be provided, the capacity of WSP or DWB sheathing shall be increased by reducing the spacing of fasteners along the perimeter of each piece of sheathing to 4 inches (102 mm) on center. Section 2308.5.6 for the full dwelling perimeter and for the full length of interior braced walls lines supported on foundations, excepting ventilation and access openings.

**Reason:** This change proposal makes clear the restrictions already imposed by Section 2308.6.6.2 and Table 2308.6.1 by deleting unnecessary and contradictory language. Permitted in SDC D and E are one-story buildings with slab on grade construction and one-story buildings over solid blocked cripple walls, with studs 14 inches or less in height. IBC Section 2308 provisions do not allow for prescriptive bracing of cripple walls in Seismic Design Categories D and E. This is because Table 2308.6.1 is limited to one-story buildings and cripple walls over 14 inches in height are considered an additional story, turning a one-story building over cripple walls into a prohibited two-story building. Language is added clarifying the extent of solid blocking of the studs as the solid blocking will be providing seismic and wind bracing.

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

This proposal is an editorial clarification of current provisions and does not have any cost impact.

Proposal # 4576
Proponent: Rick Allen, representing International Staple, Nail and Tool Association (rallen@santa.org)

2018 International Building Code
Revise as follows:

TABLE 2308.6.3(1)
BRACING METHODS

Portions of table not shown remain unchanged.

<table>
<thead>
<tr>
<th>METHODS, MATERIAL</th>
<th>MINIMUM THICKNESS</th>
<th>FIGURE</th>
<th>CONNECTION CRITERIA(^a)</th>
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</thead>
<tbody>
<tr>
<td>PCP Portland cement plaster</td>
<td>Section 2510 to studs at maximum of 16” o.c.</td>
<td>1½&quot; long, 11 gage, 0.120&quot; dia., 7/16&quot; dia. head nails or 7/8&quot; long, 16 gage staples</td>
<td>6&quot; o.c. on all framing members</td>
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</tbody>
</table>

For SI: 1 foot = 304.8 mm, 1 degree = 0.01745 rad.

a. Method LIB shall have gypsum board fastened to one or more side(s) with nails or screws

Reason: ASTM F1667-18 requires that when gage is used as a diameter for nails, a decimal equivalent must also be shown. This requirement was put in place because of the multiple and conflicting wire gage tables that are used in the manufacturing of nails.

Cost Impact: The code change proposal will not increase or decrease the cost of construction. This proposal will not change the cost of production. It only provides clarification required by ASTM F1667-18

Proposal # 4084
TABLE 2308.7.3.1
RAFTER TIE CONNECTIONS

<table>
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<tr>
<th>RAFTER SLOPE</th>
<th>TIE SPACING (inches)</th>
<th>NO SNOW LOAD</th>
<th>GROUND SNOW LOAD (pound per square foot)</th>
<th>30-pounds-per-square-foot</th>
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Required number of 16d common (3\times0.162\textsuperscript{a}) nails\textsuperscript{b} per connection\textsuperscript{c}.
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For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound per square foot = 47.8 N/m².

a. 40d box (5\(\frac{1}{4}\) x 0.162\(\frac{\text{in.}}{}\)) or 16d sinker (3\(\frac{1}{4}\) x 0.148\(\text{in.}\)) nails are permitted to be substituted for 16d common (3\(\text{in.}\) x 0.16\(\text{in.}\)) 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 clinched.
Rafter tie heel joint connections are not required where the ridge is supported by a load-bearing wall, header or ridge beam.

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.

Equivalent nailing patterns are required for ceiling joist to ceiling joist lap splices.

Connected members shall be of sufficient size to prevent splitting due to nailing.

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.

Applies to roof live load of 20 psf or less.

Tabulated heel joint connection requirements 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:

<table>
<thead>
<tr>
<th>$H_C/H_R$</th>
<th>Heel Joint Connection Adjustment Factor</th>
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<tbody>
<tr>
<td>$1/3$</td>
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<td>$1/10$ or less</td>
<td>$1.11$</td>
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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.

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 $(Z \times 0.77 \times 1.25)/(Z \times 1.0 \times 1.15) = 0.84$ 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:**

**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.

Proposal # 5101

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ICC COMMITTEE ACTION HEARINGS :: April, 2019

S187-19
S188-19

IBC®: 2401.1 (New)

Proponent: Mike Fischer, Kellen Company, representing The Plastic Glazing Coalition of the American Chemistry Council (mfischer@kellencompany.com)

2018 International Building Code

Revise as follows:

2401.1 Scope. The provisions of this chapter shall govern the materials, design, construction and quality of glass, light-transmitting ceramic and light-transmitting plastic panels for exterior and interior use in both vertical and sloped applications in buildings and structures. Light-transmitting plastic glazing shall also meet the applicable requirements of Chapter 26.

Reason: While Chapter 24 is titled Glass and Glazing, it specifically includes provisions for plastics in skylights and sloped glazing as well as other uses of light-transmitting plastics. Chapter 24 includes no “materials” section as some other IBC chapters do. Adding a reference to the additional requirements of Chapter 26 connects these chapters to ensure all the provisions are recognized.

Cost Impact: The code change proposal will not increase or decrease the cost of construction
The proposal is editorial and makes no technical changes to the code.

Proposal # 2043
S189-19

IBC®: 2403.3

Proponent: Tom Zaremba, representing Glazing Industry Code Committee (GICC), a section of the National Glass Association (NGA) (tzaremba@ralaw.com)

2018 International Building Code

Revise as follows:

2403.3 Glass Framing. To be considered firmly supported, the framing members for each individual pane of glass shall be designed so that the deflection of the edge of the glass perpendicular to the glass pane shall not exceed $\frac{1}{175}$ of the glass edge length or $\frac{1}{240} + \frac{1}{4}$ inch (19.1 mm), whichever is less when the glass edge length is not more than 13 feet 6 inches (4115 mm), or 1/240 of the edge length + 1/4 inch (19.1 mm) whenever the glass edge length is greater than 13 feet 6 inches (4115 mm), when subjected to the larger of the positive or negative load where loads are combined as specified in Section 1605.

Reason: Permissible framing deflection is addressed in sections 1604.3.7 and 2403.3 of the IBC. Section 1604.3.7 was modified in the last code development cycle and now provides:

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

1. 1/175 of the length of span of the framing member, for framing members having a length not more than 13 feet 6 inches (4115 mm).

2. 1/240 of the length of span of the framing member + 1/4 inch (6.4 mm), for framing members having a length greater than 13 feet six inches (4115 mm)."

This proposal updates section 2403.3 to make it consistent with section 1604.3.7. It also deletes the reference to a 3/4 inch framing deflection from section 2403.3 since it is technically correct only for shorter glass spans and leaving it in would make section 2403.3 inconsistent with section 1604.3.7.

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

This proposed change makes section 2403.3 of the IBC consistent with section 1604.3.7. Making these IBC sections consistent will not increase or decrease the cost of construction.
Proponent: 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.

Exception: In monolithic and multiple-layer sloped glazing systems, the following apply:

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 m²) 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 \(\frac{3}{16}\) 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 m²) 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.

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:

4.1. Each pane of the glass is 16 square feet (1.5 m²) 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. 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:

5.1. Each pane of glass is 16 square feet (1.5 m²) 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.

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.
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.
2018 International Building Code

Revise as follows:

2407.1 Materials. Glass used in a handrail or a guard shall be laminated glass constructed of fully tempered or heat-strengthened glass and shall comply with Category II of CPSC 16 CFR Part 1201 or Class A of ANSI Z97.1. Glazing in in-fill panels a handrail or a guard shall be of an approved safety glazing material that conforms to the provisions of Section 2406.1.1. For all glazing types, the minimum nominal thickness shall be 1/4 inch (6.4 mm).

Exception: Single fully tempered glass complying with Category II of CPSC 16 CFR Part 1201 or Class A of ANSI Z97.1 shall be permitted to be used in handrails and guardrails where there is no walking surface beneath them or the walking surface is permanently protected from the risk of falling glass.

2407.1.1 Loads. The panels, handrails, and guards and their support system shall be designed to withstand the loads specified in Section 1607.8. Glass guard elements, handrails, and guards shall be designed using a factor of safety of four.

2407.1.2 Structural Guards with structural glass baluster panels. Guards with structural glass baluster panels shall be installed with an attached top rail or handrail. The top rail or handrail shall be supported by not fewer than three glass baluster panels, or shall be otherwise supported to remain in place should one glass baluster panel fail.

Exception: An attached top rail or handrail is not required where the glass baluster panels are laminated glass with two or more glass plies of equal thickness and of the same glass type. The panels shall be tested to remain in place as a barrier following impact or glass breakage in accordance with ASTM E2353.

2407.1.3 Parking garages. Glazing materials shall not be installed in handrails or guards in parking garages except for pedestrian areas not exposed to impact from vehicles.

2407.1.4 Glazing in windborne debris regions. Glazing installed in in-fill panels exterior handrails or balusters guards in windborne debris regions shall comply with the following: be laminated glass complying with Category II of CPSC 16 CFR 1201 or Class A of ANSI Z97.1. Where the top rail is supported by glass, the assembly shall be tested according to the impact requirements of Section 1609.2 and the top rail shall remain in place after impact.

Delete without substitution:

2407.1.4.1 Balusters and in-fill panels. Glass installed in exterior railing in-fill panels or balusters shall be laminated glass complying with Category II of CPSC 16 CFR Part 1201 or Class A of ANSI Z97.1.

2407.1.4.2 Glass supporting top rail. Where the top rail is supported by glass, the assembly shall be tested according to the impact requirements of Section 1609.2. The top rail shall remain in place after impact.

Reason: Changes made in this proposal are not intended to alter any substantive requirements of section 2407. Instead, it is intended to simplify, clarify and make the language of section 2407 consistent with IBC's defined terms.

As written, section 2407 uses a number of undefined terms, such as "panels," "in-fill panels," and "guardrails" that are actually comprehended within, and are replaced in this proposal with, the defined term "guard." A "guard" is defined in IBC section 202 as follows:

"GUARD. A building component or a system of building components located at or near the open sides of elevated walking surfaces that minimizes the possibility of a fall from the walking surface to a lower level."

It is also important to note that the term "handrail" is defined in section 202 of the IBC as follows:

"HANDRAIL. A horizontal or sloping rail intended for grasping by the hand for guidance or support."

Some terms used in section 2407, such as "panels" and "railing in fill panels" are used incorrectly as they should actually refer to both "handrails" and "guards." Where multiple defined terms are intended, they are replaced using those defined terms.

Finally, modifications to section 2407.1.4, including the deletion of subsections 2407.1.4.1 and 2407.1.4.2, are a simplification. All substantive requirements of the deleted subsections are incorporated into 2407.1.4, the main provision that addresses glazing used in windborne debris regions.

Cost Impact: The code change proposal will not increase or decrease the cost of construction.
The proposed changes do not change any substantive requirement of section 2407. They are intended merely to simplify, clarify and make the terminology used in section 2407 consistent with defined terms in the code. These changes will not increase or decrease the cost of construction.
S193-19

IBC®: 2407.1.1 (New)

Proponent: 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 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).

Proposal # 1864
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 and is separated from the stucco by a 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 area 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 of 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.
Add new definition as follows:

**BOND BREAK**: A substantially nonwater-absorbing layer placed directly behind stucco to prevent adhesion of the stucco to the surface of the water-resistive barrier, to serve as a protective layer over the water-resistive barrier, to provide a capillary break, and to promote drainage as required.

**DRAINAGE SPACE**: A separation between cladding and the surface of a water-resistive barrier created by a furred gap, channels, a porous material or matrix, or by other means to provide drainage of water downward to an outlet.

**VENTILATED DRAINAGE SPACE**: A drainage space that further incorporates the capability to allow outdoor air flow into and back out of the space behind cladding, usually by way of high and low vent inlets and outlets or by way of an air permeable (vented) cladding.

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 at least equivalent to two layers of water-resistive barrier complying with ASTM E2556, Type I. The individual installation shall comply with Table 2510.6. The individual water-resistive barrier and bond break material 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:

**TABLE 2510.6**

**WATER-RESISTIVE BARRIER, BOND BREAK, DRAINAGE, AND VENTILATION REQUIREMENTS FOR EXTERIOR PLASTER (STUCCO)**

<table>
<thead>
<tr>
<th>CLIMATE ZONE</th>
<th>WATER-RESISTIVE BARRIER</th>
<th>BOND BREAK LAYER</th>
<th>DRAINAGE SPACE</th>
<th>VENTILATED DRAINAGE SPACE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Application: Stucco over any substrate other than wood-based sheathing</td>
<td>Required</td>
<td>Not Required</td>
<td>Not Required</td>
<td>Not Required</td>
</tr>
<tr>
<td>All Climate Zones</td>
<td>Required</td>
<td>Required</td>
<td>Not Required</td>
<td>Not Required</td>
</tr>
<tr>
<td>Application: Stucco over wood-based sheathing</td>
<td>Required</td>
<td>Required</td>
<td>Required</td>
<td>Not Required</td>
</tr>
<tr>
<td>Dry (B)</td>
<td>Required</td>
<td>Required</td>
<td>Not Required</td>
<td>Not Required</td>
</tr>
<tr>
<td>Moist (A) and Marine (C), except Warm-Humid</td>
<td>Required</td>
<td>Required</td>
<td>Required</td>
<td>Not Required</td>
</tr>
<tr>
<td>Warm-Humid</td>
<td>Required</td>
<td>Required</td>
<td>Required</td>
<td>Not Required</td>
</tr>
</tbody>
</table>

a. Water-resistive barrier complying with Section 1403.2 shall be 10-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.

b. Water-resistive barrier complying with Section 1403.2 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.

c. Drainage space shall be minimum 1/8-inch (3.2 mm) depth or have a minimum drainage efficiency of 90% as measured in accordance with ASTM E2273 or Annex A2 of ASTM E2925.

d. Ventilated drainage space shall be minimum 3/16-inch (4.8 mm) depth and, where not a clear airspace, have a minimum drainage efficiency of 90% as measured in accordance with ASTM E2273 or Annex A2 of ASTM E2925.
e. Where foam plastic insulating sheathing complying with ASTM C578 or ASTM C1289 is located between the stucco and wood-based sheathing with a drainage space in accordance with footnote 'c', a ventilated drainage space is not required.

E2925-17: Standard Specification for Manufactured Polymeric Drainage and Ventilation Materials Used to Provide a Rainscreen Function

Reason: The current minimum requirements for installation of stucco over wood-based sheathing are confusing and also problematic in that they are predominantly aimed at practices that have been successful mainly in drying climates. In more moist climates, these minimum stucco installation requirements, particularly in regard to the WRB layer and lack of sufficient drainage or ventilation or hygric redistribution, has resulted in or contributed to numerous moisture-related problems.

Given the above concerns, this proposal achieves the following:

1) First, it **re-formats** the provisions into an easy-to-use tabulated (“look up” table) format as shown in proposed Table 2510.6. This will make it much easier to identify the various installation practices (including those also currently permitted in the code).

2) Second, it **clarifies** much of the confusion or ambiguity in this section of code. This is done through definitions and terminology that reflect the primary purpose of various features or materials that are important to an overall stucco installation and proper functioning of the WRB layer. This has also allowed the exceptions to be deleted since they are now incorporated more appropriately within the requirements of Table 2510.6 and the added definition of a “bond break” (replacing current use of “nonwater absorbing layer” is consistent with the intent of the existing exception #1 as explained in the reason statement to proposal S93-03/04 which brought the exception into the 2006 code).

3) Third, it provides **enhanced moisture control practices only where needed** for the moist (rainy) and hot/humid climates where rainwater management (drainage) and also ventilation (drying) or hygric redistribution become more important, particularly when used over wood-based sheathing. Thus, these provisions add the enhancements only where needed and only where stucco is used over wood sheathing which is susceptible to moisture (following the current approach to single-out special requirements for application over wood-based sheathing). It does not change requirements where stucco has been performing successfully for decades.

Finally, this proposal provides for **flexibility** in meeting the requirements, including both prescriptive and performance requirements for drainage and ventilation in the footnotes. And, these requirements are consistent with a wide selection of suitable materials currently being used and relies on available (and widely used) consensus standards for measuring performance of those materials or alternatives.

**Cost Impact:** The code change proposal will increase the cost of construction
This 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 very successful performance record. Even where enhanced practices (drainage or ventilation) are required, this will impact cost only where they are not already being used to control risk of moisture damage. For those installations not already using these enhanced provisions in moist/rainy/humid climates, this proposal will likely reduce long term costs to builders, designers, and building owners because it will reduce risk of moisture problems and improve durability.

**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.
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 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*.

_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 sheathing) 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

_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.
Proponent: Benjamin Meyer, representing self (2ben.meyer@gmail.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 at least equivalent to two layers of water-resistive barrier complying with ASTM E2556, Type I. The individual layers of the innermost water-resistive barrier shall be installed independently complying with Section 1402.2, 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, integrated with this layer. The outermost water-resistive barrier shall be installed as an intervening layer and shall not impede drainage to the exterior.

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 and 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.

Reason: Clarifies language regarding the purpose and installation methodology of a two layer WRB system for an adhered cladding system.

Cost Impact: The code change proposal will not increase or decrease the cost of construction Clarification language
S198-19

IBC: 2512.1.3 (New)

Proponent: Michael Gardner, M Gardner Services, LLC, representing Wall and Ceiling Alliance (michael@mgardnerservices.com)

2018 International Building Code

Add new text as follows:

2512.1.3. Control Joints Control joints shall be installed in accordance with ASTM C1063.

Exception: Lath shall be permitted to be installed continuous through control joints.

Reason: Proposal seeks to remedy two common exterior plaster application issues.

The proposal clarifies that control joints must be installed in exterior plaster to mitigate the stresses that cause plaster to crack. While the application requirements for control joints are identified in the ASTM C1063 standard that is referenced in Chapter 25 of the IBC, no specific code requirement mandating control joints exists. Lacking such language, the installation of control joints in exterior plaster is often overlooked.

The proposal also modifies an ASTM C1063 requirement that lath must be discontinuous at each control joint by permitting, but not requiring, the lath to run continuous through the control joints.

The placement of control joints is dictated by the maximum spacing requirements contained in the ASTM C1063 standard coupled with aesthetic design considerations. As a result, the final placement of control joints is often not established until after the wall or ceiling is erected and sheathed and the lath has been installed. In such a circumstance, the framed and lathed surfaces often have to be breached and reworked to accommodate control joints, because the C1063 standard requires the lath to be discontinuous. This creates costly re-work and weakens the exterior skin of the building.

The need to discontinue the lath at control joints is historically justified by the belief that the practice reduces plaster cracks; however, no evidence exists to support the theory. To the contrary, a multi-year testing program sponsored by the Wall & Ceiling Conference (WCC) has determined no difference in plaster cracking between continuous and discontinuous lath applications.

Significant practical evidence, created through many decades of successful applications, also indicates that a continuous lath installation does not cause plaster cracking. Continuous lath installation was permitted by the Uniform Building Code, yet there are no indications that jurisdictions enforcing the UBC experienced related plaster cracking issues.

The identified ASTM C1063 requirement should not be mandatory. This proposal seeks to permit the use of an established alternate method for control joint installation.

Cost Impact: The code change proposal will decrease the cost of construction

The proposal will reduce the cost of construction by allowing an alternate method of installation which can eliminate the need for costly re-working of previously installed lath and framing.
S199-19

IBC: TABLE 2509.2

Proponent: Patrick Vandergriff, Patrick Vandergriff Code Consulting Services, representing Patrick Vandergriff Code Consulting (pvandergriff@codeconsult.org)

2018 International Building Code

Revise as follows:

### TABLE 2509.2

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>STANDARD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Glass mat gypsum backing panel</td>
<td>ASTM C 1178</td>
</tr>
<tr>
<td>Fiber reinforced gypsum panels</td>
<td>ASTM C 1278</td>
</tr>
<tr>
<td>Nonasbestos fiber-cement backer board</td>
<td>ASTM or ISO 8336, Category C</td>
</tr>
<tr>
<td>Nonasbestos fiber-mat reinforced cementitious backer unit</td>
<td>ASTM C 1325</td>
</tr>
</tbody>
</table>

Reason: ASTM C 1278 is the Standard Specification for Fiber-Reinforced Gypsum panels. C 1278 has been an allowed standard for tile backer boards since the 2009 edition of International Residential Code. It has not, to date, been included in the International Building Code. When compliant with the standard and the manufacturer's instruction there has not been any issues with the use of this backerboard within residential construction. The serviceability of the products after installation has been of consistently similar quality of those backerboards complying with ASTM C 1178, which has been listed as an acceptable standard for backerboard in both the IRC and IBC.

In addition, a space has been added between the C and the ASTM designation number to match the actual title of the standards listed within this table.

This proposal correlates the two codes and insures more available options for materials to be used by designers and installers.

Cost Impact: The code change proposal will decrease the cost of construction. It is believed that this proposal will have no negative impact to construction costs. If anything, the addition of other competitive products into the mix will help to lower prices for new commercial construction.
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.
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.

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, \( V_{\text{allow}} \) 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, \( S_{sb} \) 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.

37-14: Design Loads on Structures during Construction

Add new standard(s) as follows:

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 temporary 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 application 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 accept structural designs for concert stages and structures used in sporting events based on load reductions permitted in the two new referenced standards. As a consequence of theis code change it is expected that ballast materials used to provide overturning 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.