S4-12
202 (NEW), 1507.16, 1507.16.1, 1607.12.3, 1607.12.3.1

Proposed Change as Submitted

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

Add new text as follows:

SECTION 202
DEFINITIONS

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

Revise as follows:

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

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

Revise as follows:

1607.12.3 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 1607.10.

1607.12.3.1 Vegetative and landscaped roofs. The uniform design live load in unoccupied landscaped areas on roofs shall be 20 psf (0.958 kN/m2). The weight of all landscaping materials shall be considered as dead load and shall be computed on the basis of saturation of the soil.

Reason: This code change proposal is intended to use terminology in the IBC that is consistent with that of the *International Green Construction Code* (IgCC). IgCC uses the terminology "vegetative roof" for what is referred to in the IBC as a "roof garden" or "landscaped roof".

This code change proposal adds a definition for the term "vegetative roof" in Section 202. The definition is identical to that in the IgCC and ASTM D1079, "Standard Terminology Relating to Roofing and Waterproofing." The term "vegetative roof" is also added where appropriate in Section 1507.16 and Section 1607.12.3.

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

1507.16-S-GRAHAM

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: The proposed definition of "vegetative roof" coordinates the IBC with the IGCC, providing a needed link.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Craig Conner, Building Quality, representing self, requests Disapproval.

Commenter's Reason: The definition is confusing – What are "interacting components"? what does "normally insulate" mean? The definition includes text that is commentary. The IBC and IECC use the terms "roof gardens" and "landscape roofs". The terms "vegetative roofs", "roof gardens" and "landscaped roofs" are overlapping. Are there vegetative roofs that are not "roof gardens and landscaped roofs?" In a code section referring to one should the other terms be used too? Best to stick with one set of terms, the terms already used in the IBC and IECC.

S4-12

Final Action: AS AM AMPC____ D

S9-12
1504.3.1.1 (New), Chapter 35 (New)

Proposed Change as Submitted

Proponent: Mike Ennis, Single Ply Roofing Industry (SPRI) (m.ennis@mac.com)

Add new text as follows:

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

Add new standard to Chapter 35 as follows:

ANSI/SPRI

WD-1-XX Wind Design Standard Practice for Roofing Assemblies

Reason: There are two primary reasons that ANSI/SPRI WD-1 should be included as a reference standard in the IBC.

1. The International Building Code provides specific requirements for calculating the wind uplift load pressure on the roof assembly. However it does not currently provide a prescriptive method to enhance the perimeter and corner attachment due to the higher wind loads in these regions. ANSI/SPRI WD-1 is a national consensus standard that has been reviewed by testing laboratories, membrane manufacturers, roofing system component suppliers, contractors and consultants. This standard provides prescriptive requirements for corner and perimeter enhancement. The user first identifies a suitable roof assembly that will resist the calculated wind uplift pressure for the field of the roof, then enhances the fastening pattern to meet the calculated corner and perimeter wind uplift load pressure. Designing the roof system to resist the higher wind loads at the perimeter and corner regions is accomplished by either adding additional fasteners or increasing the amount of adhesive used, depending upon the specific roof system chosen. This approach allows the user to work from one base assembly and enhance the attachment of the base assembly for perimeter and corner regions instead of trying to locate tested assemblies for each of these areas.

The ANSI/SPRI standard also requires that a 2.0 safety factor be applied to tested wind uplift values, unless another value is specified. So, for example, if a roof system passes a wind uplift test at 120 lbs/ft2, this value is divided by 2 before determining if the system will resist the calculated wind uplift pressure loads for the building. This safety factor has historically been used by the roofing industry to account for variables between tested loads and performance in the field. These variables include deviations in installation and the fact that the wind load test procedures used incorporate static applied loads while dynamic, cyclic loads occur in the field. The IBC does not currently contain this requirement.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] 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, 2012.

1504.3.1.1 (NEW)-S-ENNIS

Public Hearing Results

Note: For staff analysis of the content of SPRI WD-1 relative to CP#28, Section 3.6, please visit: http://www.iccsafe.org:8888/cs/codes/Documents/2012-13cycle/Proposed-A/00a_updates.pdf

Committee Action: **Disapproved**

Committee Reason: There are concerns about wind loading requirements in the proposed reference standard and opposing testimony suggests it could circumvent ASCE 7. Also the committee reviewed the 2008 edition of the standard, while a proposed modification would have adopted a different edition.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Mike Ennis, Single Ply Roofing Industry Inc. (SPRI), requests Approval as Submitted.

Commenter's Reason: The Code Committee recommended this proposal for disapproval because there were concerns about wind loading requirements in the proposed reference standard and opposing testimony suggesting it could circumvent ASCE 7. Also the committee reviewed the 2008 edition of the standard, while a proposed modification would have adopted a different edition.

The proposed reference standard does not circumvent ASCE 7. The formulas developed for the extrapolation methods are based on an empirical analysis of wind resistance test results. The extrapolation methods can be used to enhance perimeter and corner attachment to meet the higher wind loads in these areas. The increased fastening, as determined by the extrapolation method, in these locations assures that the perimeter and corner regions can resist the wind loads as calculated in accordance with ASCE 7.

Both the version of ANSI/SPRIWD-1 that was in force at the time the code change proposal was submitted, and the draft version that was being updated to ASCE 7-10 requirements were submitted with the code change proposal. The new version of ANSI/SPRI WD-1 was approved by the ANSI Board of Standards review on July 10, 2012.

S9-12

Final Action: AS AM AMPC____ D

S11-12
1504.3.1

Proposed Change as Submitted

Proponent: Bonnie Manley, P.E., American Iron and Steel Institute (bmanley@steel.org)

Revise as follows:

1504.3.1 Other roof systems. ~~Roof systems with built~~ Built-up, modified bitumen, fully adhered or mechanically attached single-ply ~~through fastened metal panel~~ roof systems, and other types of membrane roof coverings shall also be tested in accordance with FM 4474, UL 580 or UL 1897.

Reason: The first change is purely editorial – the sentence doesn't need to reference "roof systems" twice. Also, this section should not include reference to through fastened metal panel roof systems, since they are covered in Section 1504.3.2.

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

1504.3.1-S-MANLEY

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

1504.3.1 Other roof systems. Built-up, modified bitumen, fully adhered or mechanically attached single-ply roof systems, metal panel roof systems applied to a solid or closely fitted deck, and other types of membrane roof coverings shall also be tested in accordance with FM 4474, UL 580 or UL 1897.

Committee Reason: This proposal is editorial in nature, deleting redundant wording. The modification assures that metal panel roof systems that are installed over solid decking are covered.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

John C. Harrington, representing FM Global, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1504.3.2 Metal panel roof systems. This section applies to structural metal panel roof systems where the roof panel deck acts as the roof deck and roof covering and provides both weather protection and support for structural loads. Structural standing seam metal panel roof systems shall be tested in accordance with ASTM E 1592 or FM 4474. Structural through-fastened metal panel roof systems shall be tested in accordance with FM 4474, UL 580 or ASTM E 1592.

(Portions of proposal not shown remain unchanged)

Commenter's Reason: The existing language in 1504.3.1 included FM 4474 as a means of testing metal panel roof systems. Code proposal S11-12 (Approved as Modified) changed the language in this section of the code and narrowed the scope of what type of metal panel roof systems that FM 4474 could be used for. We were fine with the existing 1504.3.1 but after this scope change was made, we need to provide this comment for the broader category of metal panel roof systems in 1504.3.2 to include FM 4474 as a means of testing on any type of metal panel roof system in accordance with the scope of this testing standard. The scope of FM 4474 includes both standing seam and lap seam (through-fastened) metal roof systems. There are numerous roof manufacturers who already have certified their metal panel roofing systems to FM 4474 and many other systems where the roofs are in the process of this testing certification and it is critical to the roofing industry that this alternate means of roofing certification be maintained. Note that this modification to Section 1504.3.2 uses the updated wording based on S13-12 (AM).

S11-12

Final Action:	AS	AM	AMPC____	D

S13-12
1504.3.2

Proposed Change as Submitted

Proponent: Bonnie Manley, P.E., American Iron and Steel Institute (bmanley@steel.org) and Lee Shoemaker, Metal Building Manufacturer's Association

Revise as follows:

1504.3.2 Metal panel roof systems. ~~Metal~~ Standing seam metal panel roof systems ~~through fastened or standing seam~~ shall be tested in accordance with ~~UL 580 or~~ ASTM E 1592. Through-fastened metal panel roof systems shall be tested in accordance with UL 580 or ASTM E1592.

> **Exception:** Metal roofs constructed of cold-formed steel, where the roof deck acts as the roof covering and provides both weather protection and support for structural loads, shall be permitted to be designed and tested in accordance with the applicable referenced structural design standard in Section 2210.1.

Reason: The recommended language provides consistency with the uplift test requirements for standing seam roofs systems as specified in AISI S100, Section D6.2.1. AISI S100 requires that standing seam roofs be tested in accordance with ASTM E1592 to determine panel strength and UL580 is not an optional test for this type of roof system. Panel strengths for through fastened roofs, on the other hand, as specified in AISI S100, can be developed either analytically or through testing in accordance with either UL 580 or ASTM E1592.

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

1504.3.2-S-MANLEY

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

1504.3.2 Metal panel roof systems. This section applies to structural metal panel roof systems where the roof panel deck acts as the roof deck and roof covering and provides both weather protection and support for structural loads. Structural standing seam metal panel roof systems shall be tested in accordance with ASTM E 1592. Structural through-fastened metal panel roof systems shall be tested in accordance with UL 580 or ASTM E1592.

> **Exception:** Metal roofs constructed of cold-formed steel~~, where the roof deck acts as the roof covering and provides both weather protection and support for structural loads,~~ shall be permitted to be designed and tested in accordance with the applicable referenced structural design standard in Section 2210.1.

Committee Reason: This proposal clarifies the application of this section to different types of structural metal panel roof systems and better coordinates these requirements with other code provisions. The modification provides clarity by stating that this section applies to metal panel roof systems that are structural.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Jonathan Humble, AIA, NCARB, LEED BD&C, representing American Iron and Steel Institute, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1504.3.2 Structural metal panel roof systems. ~~This section applies to structural metal panel roof systems~~ Where the roof panel ~~deck acts~~ functions as the roof deck and roof covering and provides both weather protection and support for ~~structural~~ loads, the structural metal panel roof system shall comply with this section. Structural standing seam metal panel roof systems shall be tested in accordance with ASTM E 1592. Structural through-fastened metal panel roof systems shall be tested in accordance with UL 580 or ASTM E1592.

> **Exception:** Metal roofs constructed of cold-formed steel shall be permitted to be designed and tested in accordance with the applicable referenced structural design standard in Section 2210.1.

Commenter's Reason: The public comment proposes to further modify the committee recommendation to effectively overcome some grammatical and ICC manual of style issue. We propose to:

- Change the first sentence in order to read as a mandatory introduction.
- Use a more appropriate word "functions" in place of "deck acts".
- Change the title to reflect the content of the section.

S13-12

Final Action: AS AM AMPC____ D

S14-12
1504.4

Proposed Change as Submitted

Proponent: Mike Ennis, Single Ply Roofing Industry (SPRI) (m.ennis@mac.com)

Revise as follows:

1504.4 Ballasted low-slope roof systems. Ballasted lowslope (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. <u>Ballasted roof systems shall be subject to the special inspection requirements of Section 1705.10 to verify conformance to ANSI/SPRI RP-4 standard.</u>

Reason: During the 2005/2006-code change cycle a proposal was submitted to prohibit gravel or stone used as ballast on the roof of a building located in a hurricane-prone regions or on any other building with a mean roof height exceeding prescribed limits based on the building height, exposure category and basic wind speed at the site. These requirements are contained in Section 1504.8. These restrictions were imposed due to damage that occurred reportedly due to wind borne roof aggregate during high wind events. The building height restrictions were imposed due to calculated values.

Prior to this code change proposal the design of ballasted roofs were required to meet ANSI/SPRI RP-4 Wind Design Standard For Ballasted Single-ply Roofing Systems. While this is still a requirement, the code change that occurred due to this proposal now requires that both requirements be met, i.e. the requirements included in the proposal and the requirements of RP-4. This leads to conflicting requirements.

The issue with gravel blow-off that was raised by the NCSEA is that non-code compliant ballasted roof systems are being installed, which is particularly problematic in areas with the potential for high wind events. If these roof systems were installed in accordance with ANSI/SPRI RP-4, then this would not be an issue since this standard is specifically designed to prevent gravel blow-off. This statement is based on the fact that the roof systems that were reported by the NCSEA were investigated and found that they did not conform to the design requirements of the code-referenced standard, ANSI/SPRI RP-4.

To address the issue of gravel blow-off, this code change proposal requires special inspection of ballasted roof assemblies to verify conformance with ANSI/SPRI RP-4 if they are being installed in high wind regions as defined in Section 1705.10 Special inspections for wind resistance.

The ANSI/SPRI RP-4 standard was first included in the building code in 1988. It has demonstrated excellent performance, with no reports of gravel or roof blow-off on systems designed in accordance with the standard. Over 6 billion square feet of ballasted single ply roofing applications have been installed over the last two decades The vast majority of these systems have performed very well with respect to their resistance to wind pressure loads. However some damage has been observed due to aggregate blowing off non-code compliant roofs during high wind events, as noted in the NCSEA proposal.

The ANSI/SPRI Ballast Design Guide is based on over 200 wind tunnel tests conducted at the National Research Council of Canada (NRCC). This is the largest commercially available wind tunnel in North America. The tunnel and the experts at the NRCC have used this tunnel to design some of the largest suspension bridges in the world. In addition, over 40 years of field experience and observations from hurricane investigation teams from RICOWI and FEMA have been used in the development of the design criteria.

ANSI/SPRI RP-4 was revised and re-approved in 2008 and is currently being balloted for re-approval. The ballot currently out for re-approval updates the standard to ASCE7-10 requirements. One of the design objectives of ANSI/SPRI RP-4 is to prevent gravel blow-off. The above-mentioned wind tunnel testing evaluated conventional stone ballasted and stone and paver ballasted protected membrane roofs. For the systems containing stone ballasting the primary objective was to determine 4 critical wind speeds:

1. U_{c1} – the wind speed at which one or more stones were first observed to move an appreciable distance (i.e. several inches)
2. U_{c2} – the wind speed above which scouring of stones would continue more or less indefinitely as long as the wind speed is maintained.
3. U_{c3} – the wind speed at which stones were first observed to leave the roof by going over the upstream parapet (this was the parapet adjacent to the wind direction)
4. U_{c4} – the wind speed at which stones were first observed to leave the roof by going over the downstream parapet (opposite side from the wind)

In these experiments three nominal stone sizes were used. Each nominal stone size represented a mixture of stone sizes (larger and smaller) similar to the gradation, which would be obtained from a stone quarry. These experiments evaluated the impact of the following variables on the critical wind speeds defined above:

- Stone size
- Parapet height
- Building height
- Building geometry
- Direction of wind impacting the building
- Rooftop wind speed, rooftop gust wind speed, and the shape of the approaching wind velocity profile

The basic approach taken in the ANSI/SPRI RP-4 standard is that as the anticipated wind load on the roof increases due to variables such as design wind speed, building height, exposure category and parapet height, the ballast design requirements get more robust by using larger stone, or substituting pavers for stone, and ultimately not allowing for the use of a ballasted roof system.

The ballast designs contained in the national consensus standard provide restrictions on the use of ballasted single ply roof systems that will allow for the responsible use of aggregate surfacing. There is often the potential for building envelope materials, and many other materials, to become windborne debris in hurricane force wind exposures. In these situations, the approach is to learn how to properly use these materials in high wind areas, not ban their use. The ANSI/SPRI RP-4 standard allows for the continued use of ballasted roofing systems, which are a cost effective method to keep the roof system in place and to improve the energy performance of the building. (Reference the SPRI/DOE/ORNL report on energy effectiveness of ballasted roof systems by going to the following web link, http://www.spri.org/publications/policy.htm under Technical Reports. Select the research report entitled: Evaluating the Energy Performance of Ballasted Roof Systems.

Two of the most critical controlling factors identified through this extensive test program on the various critical wind speeds were stone size and parapet height. A brief summary of the wind tunnel test program, and reports written as part of this program follows. The reports can be viewed in the entirety at the same web link provided above for the energy study report. The wind tunnel reports are located at the bottom of that page under Miscellaneous.

LTR-LA-142 Estimation of Critical Wind Speeds for Scouring of Gravel or Crushed Stone on Rooftops January 1974

Objectives:

- Determine the critical wind speeds and corresponding surface shear stress that cause movement of various stone sizes and shapes by taking direct measurements of these values via wind tunnel testing.
- Use this data to determine constants that can be used in equations to calculate critical surface shear stress
- Obtain guidance about the effects of parapets and obstacles, which cause strong three-dimensional effects, notably vortices.

Conclusions:

- The surface shear stress required to cause stone motion is directly proportional to nominal stone diameter.
- The constant of proportionality appears to be essentially independent of stone size and shape and of the detailed shape of the velocity profile near the gravel surface.
- Critical wind speeds to initiate stone motion can therefore be easily predicted if the relationship between surface shear stress and wind speed is known for the situation of interest.
- The dead air region behind a parapet extended downstream about 15 parapet heights. The turbulence of natural wind will tend to reduce the dead air zone.

LTR-LA-162 Wind Tunnel Tests on Some Building Models to Measure Wind Speeds at Which Gravel is Blown Off Rooftops June 1974

Objectives:

- This series of tests was conducted to build upon the data obtained in the January 1974 test series. Specifically to provide data for some typical building geometries and to investigate the effects of building form, building height, parapet height, wind direction, and gravel size on the critical wind speeds required to cause scouring and blow-off of roofing gravel.
- In this series 1/10 scale models were evaluated in a 30' x 30' wind tunnel.

Conclusions

- The critical wind speeds at which scouring of nominal 0.9", 1.5" and 2.8" diameter gravel (scaled to 1/10 size) occurs and begins to blow-off rooftops were investigated. The nominal sizes represent the average size of a typical mixture.
- The critical wind speeds are lowest when the wind direction is at or about 45° to the walls of the building.
- For a given building configuration the critical wind speeds are proportional to the square root of the gravel size.
- The critical wind speeds increase with increasing parapet height and decrease with increasing building height.
- The length:width ratio of the building is unimportant as long as the width and length are large compared to the parapet height.

NRC No. 15544 Design of Rooftops Against Gravel Blow-Off September 1976

Objectives:

- This report describes a procedure that can be used to estimate the wind speeds at which gravel of a given nominal size will be blown off rooftops.
- The report also describes a procedure for determining design wind speeds at rooftop level.
- The gravel blow-off procedure is based on data obtained from previous wind tunnel tests described above.

Conclusions

- The results of wind tunnel tests conducted to determine critical wind speeds for scour or blow-off of roofing gravel for a specific low-rise building shape can be generalized to apply to any low-rise rectangular building having a flat rooftop.
- Similar generalization is possible for high-rise shapes of any particular length: width ratio.
- This permits development of a general, easy to use procedure for estimating critical wind speeds required to cause scour or blow-off of roofing gravel from various building configurations.

LTR-LA-189 Further Wind Tunnel Tests on Building Models to Measure Wind Speeds at Which Gravel is Blown Off Rooftops August 1977

Objectives:

- Obtain additional data to permit previously obtained results to be generalized so as to be applicable to any rectangular flat-roofed low-rise building.
- Provide data on the effects of substituting solid paving blocks for loose gravel in the most wind sensitive areas of the rooftop.

Conclusions:

- The wind speed at rooftop level appears to be the dominant factor in controlling gravel scour and blow-off as opposed to the wind velocity profile.
- The measured wind speeds at rooftop level were used to reinterpret the data from previous wind tunnel tests.
- Within the boundaries of experimental scatter the critical wind speeds are independent of the rooftop level in the wind

boundary layer, allowing for generalization of results to various building heights and geometries.

LTR-LA-234 Model Studies of the Wind Resistance of Two Loose-Laid Roof-Insulation Systems May 1979

Objectives:

- Investigate the resistance of protected membrane roof systems to damage from high winds.
- Identify wind speeds and failure mechanisms for protected membrane roof systems.

Conclusions:

- The results show that wind flows induce pressure distributions underneath the roof-insulation systems as well as on their exterior surfaces.
- These pressure differences cause uplift and are responsible for system failure.
- The wind speed to cause failure for the 2 ft. x 2 ft. paver slabs was found to be proportional to the square root of the system weight per unit area. This relationship should also be true for different geometries.

LTR-LA-269 Further Model Studies of the Wind Resistance of Two Loose-Laid Roof-Insulation Systems (High Rise Buildings) April 1984

Objectives:

- This study is an extension of the May 1979 study, to investigate the resistance of various protected membrane roof systems to damage from high winds when they are installed on high-rise buildings.

Conclusions:

- The mechanisms for wind damage are the same as those identified in earlier tests, namely gravel scour and uplifting of boards by pressure forces.
- The static pressure underneath boards or pavers tend to become equal to the exterior surface because of airflow through the joints between boards or pavers. Complete equalization cannot occur, however, in regions where the exterior pressure distribution is highly non-linear and uplifting pressure differences occur in those regions. System failure therefore tends to occur in these regions.
- High parapets are very effective in increasing resistance to wind damage.
- Mechanical interconnection of boards or pavers by use of strapping, tongue & groove, etc. is an effective method for increasing wind resistance.
- For any particular system configuration, the wind speed to cause failure is proportional to the square root of the system weight per unit area.
- Gust speed at rooftop level is the pertinent speed for use in assessing the resistance of the roofing system to wind damage.

LTR-LA-294 Further Wind Tunnel Tests of Loose-Laid Roofing Systems April 1987

Objectives:

- Conduct extensive wind tunnel work to further assess the resistance to wind damage of protected membrane roofing system using paver slabs, or similar elements.
- Low, intermediate and high-rise buildings were tested, each with several parapet heights.

Conclusions:

- When a membrane is loose-laid on a leaky roof deck, ballooning will occur due to air flowing through holes in the deck from the interior of the building. This will normally result in failure at wind speeds well below those required to product failure by other mechanisms.
- In the case of immobile membranes, failure results from pressure differences, which develop across elements in some regions of the roof.
- Increased parapet height generally resulted in more favorable pressure distributions. That is, maximum suctions were reduced and suction peaks were broadened, so that pressure was less non-uniform and therefore increased failure speeds could be expected.
- Element size has a noticeable effect on failure speed, i.e. failure speeds were higher for larger elements.
- Pressure non-uniformity is reduced by vortex generators mounted on the parapets near the upwind corner of the roof, thus increasing failure wind speeds.

LTR-LA-295 Pressure Distribution Data Measured During the September 1986 Wind Tunnel Tests on Loose-Laid Roofing Systems September 1987

Objectives:

- This report supplements LTR-LA-294 by including contour plots of mean and peak roof surface pressure coefficients and mean and peak coefficients for pressure differential between the upper surface and the underside of the roofing system.

Cost Impact: This proposal will increase the cost of construction. The cost increase will be due to the cost of doing a special inspection if the system is being installed in a region described in Section 1705.10 Special inspections for wind resistance.

1504.4-S-ENNIS

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: It is unclear what special inspections requirements would apply to ballasted roof systems with the proposed reference to Chapter 17 – the section in question covers inspections of lateral force-resisting systems. Disapproval of this code change is consistent with past committee actions.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Mike Ennis, representing Single Ply Roofing Industry Inc., requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1504.4 Ballasted low-slope roof systems. Ballasted low-slope (roof slope < 2:12) single-ply roof system coverings installed in accordance with Sections 1507.12 and 1507.13 shall be designed in accordance with ANSI/SPRI RP-4. ~~Ballasted roof systems shall be subject to the special inspection requirements of Section 1705.10 to verify conformance to ANSI/SPRI RP-4 standard.~~

1504.4.1 Special inspection. Special inspection of ballasted low-slope (roof slope < 2:12) single-ply roof system coverings shall be provided in accordance with Section 1705.18.

1705.18 Ballasted low-slope roof systems. Ballasted low-slope (roof slope < 2:12) single-ply roof system coverings installed in hurricane-prone regions as defined in Section 202 shall be subject to periodic special inspection to verify that the assembly has been installed in accordance with ANSI/SPRI RP-4.

Commenter's Reason: The Code Committee recommended the original code change proposal for disapproval because it was unclear what special inspections requirements would apply to ballasted roof systems with the proposed reference to Chapter 17. The modification clarifies the special inspection requirements.

The ANSI/SPRI RP4 standard is based on hundreds of wind tunnel tests, field studies and post hurricane field inspections. In 1988 it was included in the building code as the design guide to be used for ballasted single ply roofs. It has been revised five times to include current information and recommendations. The link to Section 1504.8 was added in the 2006 version of the IBC due to a concern with gravel blow-off. Upon investigating the situations where gravel blow-off occurred, two conclusions were drawn:
1) The blow-off occurred during exposure to very high wind events.
2) The roofs where blow-off occurred were not installed per the ANSI/SPRI RP4 standard.

The solution to the blow-off problem is to verify that the roof has been installed per the standard via special inspection. SPRI believes in the use of national consensus standards, which have been developed and reviewed by subject matter experts as compared to imposing requirements that conflict with the requirements of the consensus standard. ANSI/SPRI RP4 should be a stand-alone design standard for ballasted single ply roofs as no blow-off problems have been reported for roofs installed per the requirements of this standard.

S14-12
Final Action: AS AM AMPC____ D

S15-12

1504.5.1 (New), Chapter 35 (New)

Proposed Change as Submitted

Proponent: Mike Ennis, Single Ply Roofing Industry (SPRI) (m.ennis@mac.com)

Add new text as follows:

1504.5.1 Gutter securement for low-slope roofs. Low-slope (roof slope < 2:12) roof system gutter securement shall be designed and installed for wind loads in accordance with Chapter 16 and tested for resistance in accordance with ANSI/SPRI GD-1, except V_{ult} wind speed shall be determined from Figure 1609A, 1609B, or 1609C as applicable.

Add new standard to Chapter 35 as follows:

SPRI

ANSI/SPRI GD-1-2010 Structural Design Standard for Gutter Systems Use with Low-Slope Roofs

Reason: Currently the IBC contains no requirement that gutters be designed and installed to resist wind and static loads. Studies of the aftermaths of hurricanes revealed a need for better gutter system design. Examples of these observations are shown below. SPRI developed this Standard in response to those studies.

The wind resistance tests contained 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 the building. The standard also measures the resistance of the gutter system to static forces of water and ice acting downwardly.

Following are observations of results of gutter failures during high wind events. These observations were made during post hurricane investigations conducted by RICOWI (Roofing Industry Committee on Weather Issues).

Figure 1

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

Figure 2

Figure 3 is a photo of a building located in Dickinson, TX after Hurricane Ike. This building experienced wind speeds of 100 mph.

Figure 3

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

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

Figure 4

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

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] 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, 2012.

1504.5.1 (NEW)-S-ENNIS

Public Hearing Results

Note: For staff analysis of the content of SPRI GD-1 relative to CP#28, Section 3.6, please visit:
http://www.iccsafe.org:8888/cs/codes/Documents/2012-13cycle/Proposed-A/00a_updates.pdf

Committee Action: Disapproved

Committee Reason: There's no industry consensus on the adoption of the proposed standard for gutter systems. It uses ASCE 7-05 and would mix those requirements with 2010 edition referenced by the IBC, making the outcome of its adoption unclear and enforcement a moving target.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Mike Ennis, representing Single Ply Roofing Industry Inc. (SPRI), requests Approval as Submitted.

Commenter's Reason: The Code Committee recommended this proposal for disapproval because they concluded that there's no industry consensus on the adoption of the proposed standard for gutter systems and it uses ASCE 7- 05 and would mix those requirements with 2010 edition referenced by the IBC, making the outcome of its adoption unclear and enforcement a moving target.

It is very important that the test requirements contained in this standard be adopted into the International Building Code. Failure of the edge securement in low slope roof systems has been found to be the primary cause for damage when these systems are exposed to high wind events. A study of 145 FM Global losses involving built-up (BUR) systems showed 85 losses (59%) occurred because the roof perimeter failed.

The Committee is correct that the standard references load calculations per ASCE7-05, however the code change proposal states that the load shall be calculated per the requirements of Chapter 16. Once these loads are determined per Chapter 16, the test procedures contained in ANSI/SPRI GD-1 are to be used to evaluate the strength of the attachment. This is then compared to the calculated loads to verify that the gutter is attached in a manner to resist the calculated wind loads.

S15-12

Final Action: AS AM AMPC____ D

S18-12
1504.9 (New), Chapter 35 (New)

Proposed Change as Submitted

Proponent: Mike Ennis, Single Ply Roofing Industry (SPRI) (m.ennis@mac.com)

Add new text as follows:

1504.9 Roof gardens and landscaped roofs. Roof gardens and landscaped roofs shall comply with Section 1507.16 and shall be installed in accordance with ANSI/SPRI RP14. Garden and landscaped roof systems shall be subject to the special inspection requirements of Section 1705.10 to verify conformance to ANSI/SPRI RP-14.

Add new standard to Chapter 35 as follows:

SPRI
ANSI/SPRI RP-14-2010 Wind Design Standard for Vegetative Roofing Systems

Reason: Section 1507.16 requires that roof gardens and landscaped roofs comply with the requirements of Chapter 15. Section 1504.1 provides requirements for wind resistance of various roofing assemblies, however no guidance is provided for designing roof gardens and landscaped roofs to withstand wind loads. Roof gardens and landscaped roofs perform in the same manner as ballasted single ply roof assemblies when exposed to wind loads. ANSI/SPRI RP14 is a national consensus standard that has been developed in cooperation with Green Roofs for Healthy Cities with input from roof membrane manufacturers, component suppliers, contractors, green roofing professionals, testing organizations, and consultants. This design standard is much like the ballast design guide for single-ply roofs currently recognized by the IBC (ANSI/SPRI RP4). It provides the user with a series of tables that define requirements based on design wind speed, building height, parapet height and wind exposure. Three design options are provided. These design options vary in their ability to resist wind loads. Design option 1 uses a 10 lbs/ft2 minimum required load of growth media or trays, Design option 2 also requires minimum 10 lbs/ft2 of growth media or trays in the field of the roof and 13 lbs/ft2 of growth media or interlocking trays or 22 lbs/ft2 of individual trays in the corner and perimeter regions. Design option 3, which is designed for high wind load areas, requires 13 lbs/ft2 of growth media or interlocking trays, or 22 lbs/ft2 of individual trays in the field of the roof and does not allow any loose growth media or trays in the perimeter and corner regions. The perimeter of the building is defined as 40% of the building height. Adjustments are provided to increase the wind resistance of the design based on specific building conditions such as the buildings importance factor, large openings in adjacent walls and rooftop projections to name a few. The standard also provides requirements for newly planted garden roofs that do not have fully developed root systems. Fully developed root systems allow the garden roof assembly to perform very well when exposed to high wind situations, however prior to development of the root system special precautions must be taken.

This proposal includes a requirement for special inspection to verify conformance to the ANSI/SPRI RP14 design standard when the system is installed in a high wind region as described in Section 1705.10.

The basis for the standard includes wind tunnel data generated in support of the ballasted single ply design guide. This wind tunnel testing helped develop an understanding of the impact of particle size and parapet height on the performance of ballasted assemblies. It also provided information regarding the weight of ballast required to keep the roof systems in place at various wind speeds. This data, along with 50-years of garden roof performance data from both the US and Europe were used in the development of this standard.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] 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, 2012.

1504.9 (NEW) #2-S-ENNIS

Public Hearing Results

Note: For staff analysis of the content of SPRI RP-14 relative to CP#28, Section 3.6, please visit:
http://www.iccsafe.org:8888/cs/codes/Documents/2012-13cycle/Proposed-A/00a_updates.pdf

Committee Action: **Disapproved**

Committee Reason: The proposal does not appear to address all variations of vegetative roof systems. The proposed referenced standard is not based on current wind load requirements of the code and the committee does not see a consensus regarding the adoption of this new standard. Furthermore, the proposed special inspection for conformance with a design standard does not work, since the special inspection should be for the installation.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Mike Ennis, representing Single Ply Roofing Industry Inc., (SPRI) requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1504.9 Roof gardens and landscaped roofs. Roof gardens and landscaped roofs shall comply with Section 1507.16 and shall be installed in accordance with ANSI/SPRI RP14. ~~Garden and landscaped roof systems shall be subject to the special inspection requirements of Section 1705.10 to verify conformance to ANSI/SPRI RP-14.~~

1504.9.1 Special inspection. Special inspection of roof gardens and landscaped roofs shall be provided in accordance with Section 1705.18.

1705.18 Roof gardens and landscaped roofs. Roof gardens and landscaped roofs installed in hurricane-prone regions as defined in Section 202 shall be subject to periodic special inspection as defined in Section 202 to verify that the assembly has been installed in accordance with ANSI/SPRI RP-14.

(Portions of code change proposal not shown remain unchanged)

Commenter's Reason: Section 1507.16 of the IBC requires that Roof gardens and landscaped roofs meet the requirements of Chapter 15, Sections 1607.12.3 and 1607.12.3.1 and the *International Fire Code*. However, no guidance is provided regarding how to meet the requirements of Section 1504.1 Wind resistance of roofs.

Roof gardens and landscaped roofs are not new. They have been used in Europe and North America for over 70 years. Methods for keeping the roof system in place when they are exposed to high wind conditions are well established.

The ANSI/SPRI RP14 standard provides design guidelines for vegetative roofs to meet wind resistance requirements. It is based on wind tunnel data, European design guides and FM Loss Prevention Guide 1-35.

Following are the reasons provided by the Code Committee for recommending this proposal for disapproval, and our response.

1) The Standard does not address all variations of vegetative roof systems - The Standard provides design requirements based on variables such as design wind speed, exposure category, building height and parapet height. It also provides specific requirements for special building conditions such as positive pressure in buildings, and rooftop projections to name a couple. These requirements can be applied to any type of vegetative roofing system.

2) The Standard is not based on proposed current wind load requirements of the code. The standard is based on nominal design wind speeds, not wind loads. The wind speed maps referenced in the code are based on ultimate wind speeds. Table 1609.3.1 provides conversions from ultimate wind speed to nominal wind speed, which can then be used with the Standard.

3) The committee did not see consensus regarding the adoption of this new standard. - It is an ANSI National Consensus standard. This does not mean that there is unanimous support, but the majority of the canvass body supports the Standard.

4) The proposed special inspection for conformance with a design standard does not work, since the special inspection should be for the installation. - The proposed modification addresses this issue.

S18-12

Final Action: AS AM AMPC____ D

S24-12
1505.9 (New), Chapter 35 (New)

Proposed Change as Submitted

Proponent: Mike Ennis, Single Ply Roofing Industry (SPRI) (m.ennis@mac.com)

THIS CODE CHANGE PROPOSAL IS ON THE AGENDA OF THE IBC FIRE SAFETY CODE DEVELOPMENT COMMITTEE. SEE THE HEARING ORDER FOR THE IBC FIRE SAFETY CODE DEVELOPMENT COMMITTTEE

Add new text as follows:

1505.9 Roof gardens and landscaped roofs. Roof gardens and landscaped roofs shall comply with Section 1507.16 and shall be installed in accordance with ANSI/SPRI VF-1.

Add new standard to Chapter 35 as follows:

SPRI
VF-1-2010 External Fire Design Standard for Vegetative Roofs

Reason: Section 1507.16 requires that roof gardens and landscaped roofs comply with the requirements of Chapter 15. Section 1505 requires that roofing assemblies be fire classified. The current test procedures used to provide this fire classification are not applicable to garden and landscape roofs due to the many variables (plant types, moisture content, etc.) that exist for these types of systems. ANSI/SPRI VF-1 is a national consensus standard that has been developed in conjunction with Green Roofs for Healthy Cities with input from roof membrane manufacturers, component suppliers, contractors, green roofing professionals, testing organizations, and consultants. This standard provides a design method to assure an acceptable level of performance of roof gardens and landscaped roofs when exposed to exterior fire sources. The general approach used in this standard is to design in fire breaks for large roof areas, around rooftop equipment and penetrations, and next to adjacent walls. Some of the specific requirements are:

- Exposed membrane areas must conform to the designed fire resistance requirements as determined by the authority having jurisdiction.
- For all vegetated roofing systems abutting combustible vertical surfaces, a Class A (per ASTM E108 or UL790) rated assembly must be achieved for a minimum 6 ft (1.83 m) wide continuous border placed around rooftop structures and all rooftop equipment.

For large roof areas: Partition the roof area into sections not exceeding 15,625 ft2 (1,450 m^2), with each section having no dimension greater than 125 ft (39 m) by installing a a minimum of 3ft. (0.9 m) wide, Class A rated assembly barrier zones.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] 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, 2012.

1505.9-S-ENNIS.doc

Public Hearing Results

This code change was heard by the IBC Fire Safety code development committee.

Note: For staff analysis of the content of SPRI VF-1 relative to CP#28, Section 3.6, please visit:
http://www.iccsafe.org:8888/cs/codes/Documents/2012-13cycle/Proposed-A/00a_updates.pdf

Committee Action: Approved as Submitted

Committee Reason: The committee agreed that fire design contained within the SPRI VF-1 standard was appropriate for roof gardens and landscaped roofs rather than the traditional test methods used to determine fire classification. Further, the committee felt that the standard was compliant with ICC Council Policy 28 (CP28).

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Julie Ruth, JRuth Code Consulting representing American Architectural Manufacturers Association, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1505.9 Roof gardens and landscaped roofs. Roof gardens and landscaped roofs shall comply with Section 1507.16 and shall be installed in accordance with ANSI/SPRI VF-1.

> **Exception:** Skylights shall comply with Section 711.4, Chapters 15, 17, 24 and 26 of the IBC, and shall not be considered as roof penetrations.

Commenter's Reason: This Public Comment addresses an error in a newly proposed IBC referenced standard. Specifically, the standard, ANSI/SPRI VF-1, classifies skylights as roof penetrations. Skylights are fenestration products and should not be classified as roof penetrations.

The International Building Code distinguishes between penetrations of an assembly, and openings such as fenestrations. Penetrations, such as ductwork or piping, pass through an assembly and extend beyond the plane of the assembly extensively on either side of it.

Openings, on the other hand, occur primarily within the plane of the assembly. Typically the only projection of products installed in those openings may be pieces of trim or other finishing type materials.

More significantly, products intended for installation into openings, such as fenestration products, are designed and developed to maintain the integrity of the assembly into which they are inserted. Fenestration must be designed and installed to preserve the integrity of the building envelope. Specifically, all fenestration products, including skylights, must provide resistance to the applicable structural loads, water penetration resistance, resistance to air leakage, reduced thermal transmittance and solar heat gain while providing appropriate transmittance of visible light to the building interior.

Skylights are included within the definition of fenestration in the *International Energy Conservation Code* and the *International Residential Code*, They are dealt with as fenestration throughout the International Codes.

The fire resistance characteristics of skylights as a component of the building envelope are already addressed in Chapters 7, 15 and 26 of the International Building Code.

Penetrations of the building envelope are dealt with differently than fenestration throughout the IBC, IRC and IgCC. Classifying skylights as penetrations of the roof assembly would not be appropriate.

S24-12

Final Action: AS AM AMPC____ D

S25-12
1506.1

Proposed Change as Submitted

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

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 <u>printed</u> installation instructions. Installation of roof coverings shall comply with the applicable provisions of Section 1507.

Reason: This code change proposal clarifies the intent of the code by specifically stipulating manufacturers' installation instructions need to be in print. Other forms of instructions, such as verbal statements, are not appropriate for code compliance purposes.

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

1506.1-S-GRAHAM

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal 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 ~~printed~~ <u>*approved*</u> installation instructions. Installation of roof coverings shall comply with the applicable provisions of Section 1507.

Committee Reason: The proposal does clarify which installation instructions are applicable to roof covering installations. The modification substitutes the term "approved" which is preferred because it will allow the jurisdiction to verify the roof covering installation.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment 1:

Jonathan Siu, representing City of Seattle Department of Planning & Development, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

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

Commenter's Reason: The original proposal had the right idea, in that it prevented verbal statements from overriding the code. However, as the Structural Committee modified this section, the building official now has to approve all manufacturers' installation instructions. This is not something most building officials have the time or expertise to do—on what basis will he/she approve the instructions? Will he/she have to review the test reports for each and every roofing products being installed in the jurisdiction? The text approved by the Committee seems to indicate so. Will the jurisdiction take on the liability for failed roofs if the building official's "approved" installation instructions contradict the manufacturers' instructions? In addition, from the roofing contractors' side, the modified text appears to introduce a lot of subjectivity and uncertainty into what should be a simple and straightforward process.

The reason statement published in the Report of Hearings indicates the Committee felt it was appropriate for the jurisdiction to verify the roofing installation. We agree with the statement, but feel this is why the installation is inspected by the jurisdiction.

During the discussions in Dallas, the issue was raised that not all manufacturers' instructions are actually printed (which was the term added in the original proposal)—many are now available electronically. This public comment accomplishes the intent of the original proposal by requiring the instructions be "documented" in some fashion, but leaves flexibility as to the media used.

Public Comment 2:

Steven P. Regoli, Ohio Board of Building Standards, requests Disapproval.

Commenter's Reason: Both the original code change proposal to Chapter 15, Roof Assemblies and Rooftop Structures, Section 1506 Materials, 1506.1 Scope, and the subsequent committee action to approve as modified have created an odd internal inconsistency within the language in the codes.

The original change proposal suggested that this language needed modification to clarify the intent of the code because manufacturers' installation instructions need to be *in print* and other forms of instructions, such as verbal statements, are not appropriate for code compliance purposes.

The proposal, after adjusting the language on-the-fly during the committee hearing, was approved as modified. The committee did not accept the code change as submitted because digital versions of installation instructions exist which may not be reflected by the use of the introduced word "printed." Instead, the committee modified the code change to replace the word "printed" with the word "approved". Unfortunately rather than deny the change and maintain consistency within the codes, the committee modified it in an unrelated way and the language now presents two problems.

First, the IBC definition of the term "approved" reads, "Acceptable to the code official or authority having jurisdiction" and, as the IBC Commentary explains, "Whenever this term is used, it intends that only the enforcing agency can accept a specific installation or component as complying with the code."

The implication is that the code official would now have to approve roof material manufacturer's installation instructions (with no criteria provided with which to make that determination).

Additionally, the term "manufacturer's installation instructions" is used 181 times (refer to attached table) in the Public Hearing (Group A) codes heard in Dallas. The committee inadvertently created a condition in which only the roof material manufacturer's installation instructions must be approved while all others incidences of the term will not need this clarification. This changes the way in which manufacturer's installation instruction are used, implies that perhaps all manufacturer's installation instructions should be approved by the code official, or suggests that roof material manufacturer's installation instructions are more critical than others.

Given the frequency of the use of this term and the fact that the original code change only intended to address of the form of the installation instructions and not the approval of them, this modification adds a unique material manufacturer's installation instruction approval to a code official's duties with no approval criteria provided for the approval and no explanation off why these instructions should be addressed in the codes differently than the hundreds of others referenced in code language.

INSTANCES OF "*MANUFACTURER'S INSTALLATION INSTRUCTONS*" IN HEARING GROUP A CODES

2012 IBC - 38 Instances

IBC Section	Title
704.13.2	Manufacturer's installation instructions
704.13.4	Temperature
717.2	Installation
717.6.2.1	Ceiling radiation dampers
906.7	Hangars and brackets
1404.11	Polypropylene siding
1405.18	Polypropylene siding
1407.6	Weather resistance
1409.6	Weather resistance
1503.5	Roof ventilation
1506.1	Scope
1507.1	Scope
1507.3.8	Application
1507.3.9	Flashing
1507.4.2	Deck slope
T1507.4.3(1)	Metal Roof Coverings
1507.4.5	Underlayment and high wind - 2X
1507.5.3.1	Underlayment and high wind
1507.6.3.1	Underlayment and high wind - 2X
1507.7.3.1	Underlayment and high wind
1507.8.3.1	Underlayment and high wind - 2X
1507.8.8	Flashing
1507.9.3.1	Underlayment and high wind - 2X
1507.9.9	Flashing
1507.17.2	Attachment
1509.7.4	Photovoltaic panels and modules
1510.6	Flashings
1805.2.1	Floors
1805.3	Floors
2111.11	Fireplace clearance, Exception 1
2112.2	Installation
2405.4	Framing, Exception
2610.2	Mounting
K105.2	Design criteria

2012 IPC - 9 Instances

IPC Section	Title
301.7	Conflicts
303.2	Installation of materials
306.2	Trenching and bedding
417.5.2	PVC sheets
417.5.2.2	Chlorinated polyethylene (CPE) sheets
421.2	Installation
421.5	Access to pump
502.1	General
604.3	Water distribution system design criteria

2012 IPSDC - 3 Instances

IPSDC Section	Title
505.6.3	Mechanical joint coupling
505.10.1	Mechanical joints
505.11.1	Mechanical joints

2012 IFGC - 67 Instances

2013 IMC - 64 Instances

IMC Section	Title
304.1	General - 2X
304.2	Conflicts - 3X
304.1	Clearances from grade
306.1.1	Central furnaces - Exception
307.1	Fuel-burning appliances
502.11.1	Projectors with an exhaust discharge
504.6.4.2	Manufacturer's instructions
504.7	Commercial clothes dryers
506.3.11.2	Field-applied grease duct enclosures
506.3.11.3	Factory-built grease duct assemblies
506.4.2	Type II terminations
603.6.4	Flexible air duct and air connector clearance
603.9	Joints, seams and connections
603.11	Furnace connections
603.18	Registers, grilles and diffusers
607.2	Installation
607.6.2	Ceiling radiation dampers
801.10.2	Connection to factory-built fireplace flue
801.14	Connections to exhauster
801.16	Flue lining - 2X
801.2	Plastic vent joints
802.3	Installation
802.4	Vent termination caps required
802.6	Minimum vent heights - Exceptions 2 and 3
802.8	Insulation shield
804.1	Direct-vent terminations
804.2	Appliances with integral vents
804.3.7	Exhauster sizing
805.1	Listing
905.1	General
906.1	General
907.1	General
908.1	General
909.1	General
910.1	General
911.1	General
912.3	Clearances
913.1	General
914.2	Installation
915.2	Powered equipment and appliances
916.1	General
917.1	Cooking appliances
918.2	Minimum duct sizes - 3X
920.1	General
923.1.1	Installation
924.1	General
1002.1	General
1002.2	Water heaters utilized for space heating
1002.3	Supplemental water-heating devices
1003.1	General
1004.4	Mounting
1006.3	Pressure relief for pressure vessels
1006.7	Boiler safety devices
1104.2	Machinery room - Exception 1
1203.3.7	Grooved and shouldered mechanical joints
1209.4	Not embedded related piping
1401.4	Solar energy equipment and appliances

TOTAL = 181 INSTANCES

S25-12

Final Action: AS AM AMPC____ D

S28-12
1507.10.3 (New)

Proposed Change as Submitted

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

Add new text as follows:

1507.10.3 Mopping asphalt. Asphalt used in the field application of hot-applied built-up roofs shall comply with ASTM D312 and have a minimum 125°F (69.4°C) temperature differential between the asphalt's equiviscous temperature and its flash point temperature. Asphalt shall not be heated to or above its flash point temperature.

Reason: This code change proposal is intended to add requirements to the Code to provide for the safe and proper installation of hot-applied built-up roofs.

The application of most built-up roofs involves heating asphalt at the jobsite, typically in either an asphalt kettle or asphalt tanker located at ground level, to temperatures in excess of 500 °F (260°C) in order to dispense the asphalt at the point of application (rooftop) at an adequate temperature for proper application. The material standard for roofing asphalt--ASTM D312, which is already referenced in the Code--provides for the testing and labeling of asphalt's maximum heating temperature (flash point temperature) and proper application temperature (equiviscous temperature).

In order to minimize the risks of fires associated with jobsite heating of asphalt, an asphalt should not be heated to its flash point temperature. To allow for the proper application of mopping asphalt, a temperature differential between the asphalt's heating temperature and its equiviscous temperature is necessary to account for the asphalt's cooling during transportation from the heating location (e.g., ground level) and the point of applcaition (rooftop). *The NRCA Roofing Manual* suggests a minimum 125°F (69.4°C) differential between an asphalt's equiviscous temperature and it's flash point temperature for this purpose.

This code change proposal establishes a minimum temperature differential between and asphalt's equiviscous temperature and it's flash point temperature, and stipulates asphalt shall not be heated to or above its flash point temperature.

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

1507.10.3 (NEW)-S-GRAHAM

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: The committee agreed that adding the minimum temperature differential for asphalt to the code is a good idea that will provide direction to installers/contractors.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Steven P. Regoli, Ohio Board of Building Standards, requests Disapproval.

Commenter's Reason: This National Roofing Contractors Association code change proposal adds requirements to the Code to provide for the safe and proper installation of hot-applied built-up roofs by introducing an asphalt kettle or asphalt tanker temperature requirements. This is being proposed, apparently , to assure that the dispensing of asphalt at the point of application (rooftop) is at an adequate temperature for proper application.

This was done, as explained by the proponent, "*to minimize the risks of fires associated with jobsite heating of asphalt by stipulation that asphalt should not be heated to its flash point temperature."* No data was provided to indicate the scope or frequency of these fires and the need to bring the requirement into the code. By adding this language, the proposal thereby makes this an item of inspection by and potentially the responsibility of the local building department.

The proponent explained that this language would "*assure, for the proper application of mopping asphalt, a temperature differential between the asphalt's heating temperature and its equiviscous temperature necessary to account for the asphalt's* *cooling during transportation from the heating location (e.g., ground level) and the point of application (rooftop). This code change proposal establishes a minimum temperature differential between and asphalt's equiviscous temperature and its flash point temperature, an dstipulates asphalt shall not be heated to or above its flash point temperature."* No explanation was provided as to why this type of roofing systems needed this additional requirement when other systems do not. The IBC lists several roofing systems that may be temperature sensitive in their application – 1507.12 Thermoset single-ply roofing, 1507.15 Liquid-applied coatings – yet installation procedures are not specified in the way this proposal does. The consensus standards referenced in these sections are material standards not installation guidelines.

ASTM D312, to which this new language refers, is itself titled, "A Standard *Specification* for Asphalt Used in Roofing" and, as a specification, states within the document that it is intended for general asphalt classification purposes only. It is not an installation guideline or safety standard. The document even includes the statement, in section 1.2 Scope that, "*This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitation prior to use."*

A result of this proposed change one could now expect, in the event of a fire associated with jobsite heating of asphalt, that a contractor could argue that the building department failed to make the code required inspection on the temperature differential of 125 degrees between the asphalt's heating temperature and its equiviscous temperature. While the committee felt this requirement was "a good idea that will provide direction to installers/contractors," by inserting these installation requirements into the code without specifying contractor responsibility, they make them an inspection responsibility of the building department. As the proponent indicated in the original supporting statement, an installation manual by National Roofing Contractors Association (NRCA) Roofing Manual is already being used that directs the installer to maintain a minimum 125°F differential between an asphalt's equiviscous temperature and its flash point temperature for this purpose.

The proponent and the committee have uniquely, whether inadvertently or intentionally, made the preparation and temperature of the asphalt for built-up roofs an item requiring that the temperatures to be evaluated by the building department whenever asphalt is heated for this roof type. These and other means and method of construction are traditionally the responsibility of the contractor and mechanic doing the installation. This could be seen as an example of scope creep as more and more of the means and methods of the construction process are finding their way into the code. If there does exist a hazard, although no data was provided by the proponents indicating the scope or magnitude of any problem, perhaps these inspections checking the temperature differential between the asphalt equiviscous temperature and its flash point temperature should be made a part of special inspections requirements in IBC Chapter 17.

S28-12

Final Action: AS AM AMPC____ D

S29-12

1507.2 (New), 1507.2.1 (New), 1507.2.2 (New), 1507.2.3 (NEW), 1507.2.8.1, 1507.3.3.3, 1507.4.5, 1507.5.3.1, 1507.6.3.1, 1507.7.3.1, 1507.8.3.1, 1507.9.3.1, Chapter 35

Proposed Change as Submitted

Proponent: T. Eric Stafford, representing Insurance Institute for Business and Home Safety (IBHS)

Revise as follows:

1507.2. Sealed roof decks. When required, a sealed roof deck shall be installed in accordance with Section 1507.2.1, 1507.2.2 or 1507.2.3.

1507.2.1 Self-adhering cap sheet. The entire roof deck shall be covered with a self adhering polymer modified bitumen membrane complying with ASTM D 1970. An approved underlayment for the applicable roof covering shall be applied over the cap sheet, unless the top surface of the membrane provides a bond break between the membrane and the roof covering.

1507.2.2 Self-adhering strips. A minimum 4 inch wide strip of self adhering polymer modified bitumen membrane complying with ASTM D 1970 shall be applied over all joints in the roof decking. An approved underlayment for the applicable roof covering shall be applied.

1507.2.3 Synthetic underlayment. The roof deck shall be covered with a reinforced synthetic roof underlayment approved as an alternate to ASTM D 226 Type I or II. The synthetic underlayment shall have a minimum tear strength of 20 lbs in accordance with ASTM D 1970 or ASTM D 4533. This underlayment shall be attached using annular ring or deformed shank roofing fasteners with minimum 1 inch diameter caps at 6 inches on center spacing along all laps and at 12" on center in the field or a more stringent fastener schedule if required by the manufacturer for high wind installations. Metal caps are required for areas where the V_{asd}, in accordance with Section 1609.3.1, equals or exceeds 110 mph. Side laps shall be a minimum of 2 inches and end laps shall be a minimum of 6 inches. All seams shall be sealed with a compatible adhesive or a compatible 4 inch wide tape. For roofs with slopes of 45 degrees and higher, seams are not required to be sealed provided laps are a minimum of 18 inches. No additional underlayment is required.

1507.2.8.1 High wind attachment. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's instructions. Fasteners are to be applied along the overlap at a maximum spacing of 36 inches (914 mm) on center.

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

> **Exception:** As an alternative, ~~adhered underlayment complying with ASTM D 1970~~ a sealed roof deck installed in accordance with Section 1507.2 shall be permitted.

1507.3.3.3 High wind attachment. Underlayment applied in areas subject to high wind [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

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

> **Exception:** As an alternative, ~~adhered underlayment complying with ASTM D 1970~~ <u>a sealed roof deck installed in accordance with Section 1507.2</u> shall be permitted.

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

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

> **Exception:** As an alternative, ~~adhered underlayment complying with ASTM D 1970~~ <u>a sealed roof deck installed in accordance with Section 1507.2</u> shall be permitted.

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

Underlayment installed where V_{asd}, in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch spacing (152 mm) at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.

> **Exception:** As an alternative, ~~adhered underlayment complying with ASTM D 1970~~ <u>a sealed roof deck installed in accordance with Section 1507.2</u> shall be permitted.

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

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

> **Exception:** As an alternative, ~~adhered underlayment complying with ASTM D 1970~~ <u>a sealed roof deck installed in accordance with Section 1507.2</u> shall be permitted.

1507.7.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion resistant fasteners in accordance with the manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

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

> **Exception:** As an alternative, ~~adhered underlayment complying with ASTM D 1970~~ <u>a sealed roof deck installed in accordance with Section 1507.2</u> shall be permitted.

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

Underlayment installed where V_{asd}, in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.

> **Exception:** As an alternative, ~~adhered underlayment complying with ASTM D 1970~~ <u>a sealed roof deck installed in accordance with Section 1507.2</u> shall be permitted.

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

Underlayment installed where V_{asd}, in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.

> **Exception:** As an alternative, ~~adhered underlayment complying with ASTM D 1970~~ a sealed rood deck installed in accordance with Section 1507.2 shall be permitted.

Add new standard to Chapter 35 as follows:

ASTM

D 4533-11 Standard Test Method for Trapezoid Tearing Strength of Geotextiles

Reason: This code change proposal simply seeks to expand and provide additional specification for using self-adhering polymer modified bitumen membrane to prevent water intrusion. The commonly used term "secondary water barrier" is no longer used, since some have argued that underlayment itself is a secondary water barrier. Secondary water barrier has been replaced by the term "sealed roof deck." Regardless of the terminology, the purpose of these provisions is provide an additional level of protection to the roof decking in the event that the primary roof covering is blown off due to high winds. It's important to note that this code change proposal does not require a sealed roof deck. Rather, it provides specific criteria for creating a sealed roof deck as an alternative to the requirements for underlayment in high winds (e.g., Section 1507.2.8.1). While providing specific installation criteria for the bitumen membrane, this code change proposal also incorporates the use of reinforced synthetic underlayment for creating a sealed roof deck. The criteria specified are consistent with the IBHS Fortified program requirements for creating a sealed roof deck.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] 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, 2012.

1507.2 (NEW)-S-STAFFORD

Public Hearing Results

Note: For staff analysis of the content of ASTM D 4533 relative to CP#28, Section 3.6, please visit:
http://www.iccsafe.org:8888/cs/codes/Documents/2012-13cycle/Proposed-A/00a_updates.pdf

Committee Action: Disapproved

Committee Reason: There is confusion over when and where these provisions for self-adhering polymer are required. Since the reports provided to the committee were nonpersuasive, there's a lack of technical date to substantiate this change.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

T. Eric Stafford, T. Eric Stafford & Associates, LLC, representing Insurance Institute for Business and Home Safety (IBHS), requests Approval as Submitted.

Commenter's Reason: We are seeking Approval as Submitted for S29-12. During the code development hearings on this proposal, there was a good bit of confusion amongst committee members regarding where and when the provisions for self-adhering polymer modified bitumen membrane was required. Much of the confusion was due, in part, to some incorrect statements from a few of the opponents to this code change. This code change does not require the use of the self-adhering polymer modified bitumen membrane. It simply provides clarification on the proper installation if that option is chosen. Sections 1507.2.8.1, 1507.3.3.3, 1507.4.5, 1507.5.3.1, 1507.6.3.1, 1507.7.3.1, 1507.8.3.1, and 1507.9.3.1 currently require “enhanced” underlayment methods (thicker felt and tighter fastening) where the V_{asd} equals of exceeds 120 mph. An exception to each of these sections permits the use of an adhered underlayment complying with ASTM D 1970 in lieu of the enhanced underlayment methods. This exception was added during the last code development cycle and is contained in the 2012 IRC and 2012 IBC, at the request of the IBC Structural Committee. This code change does not change either of those requirements that are currently in the 2012 IBC and 2012 IRC. It simply clarifies how to properly apply the self-adhering underlayment – 1) apply the membrane over the entire roof (proposed Section 1507.2.1); or 2) apply minimum 4 in. wide strips over all the joints in the roof decking (proposed Section 1507.2.2).

Additionally, this proposal provides one other alternative to the enhanced underlayment methods. Synthetic underlayment installed in accordance with proposed Section 1507.2.3 is a recognized option for creating a sealed roof deck in the IBHS Fortified program. This code change does not require the use of a synthetic underlayment. It simply provides clarification on proper installation of the synthetic underlayment to provide an additional level of protection from water penetration that is consistent with the enhanced underlayment methods currently required in the 2012 IBC and IRC. Several manufacturers of synthetic underlayment have ICC ES reports and this underlayment is currently in use. During the hearings, one of the opponents suggested that there was some research indicating that there were issues with synthetic underlayments properly shedding water. We repeated requested that information from the opponent and to this point have not received any information to support his claim. In fact, in subsequent conversations, the opponent has backed off his claim to a degree. We are not aware of any data or research that suggests synthetic underlaymnets do not properly shed water.

S29-12

Final Action: AS AM AMPC____ D

S31-12

1507.2.6.1 (New), 1507.2.8.1, 1507.3.3.3, 1507.3.6.1 (New), 1507.4.5, 1507.5.3.1, 1507.6.3.1, 1507.7.3.1, 1507.8.3.1, 1507.9.3.1

Proposed Change as Submitted

Proponent: T. Eric Stafford, representing Insurance Institute for Business and Home Safety (IBHS)

Revise as follows:

1507.2.6.1 Fasteners and high winds. In areas where the ultimate design wind speed, V_{ult} equals or exceeds 130 mph, fasteners for asphalt shingles shall be annular ring shank nails having not less than 20 rings per inch in addition to the requirements of Section 1507.2.6.

1507.2.8.1 High wind attachment. Underlayment applied in areas subject to high winds ~~[V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1]~~ [V_{ult} equals to or greater than 130 mph, shall be applied with corrosion-resistant fasteners complying with Section 1507.2.6.1 in accordance with the manufacturer's instructions. Fasteners are to be applied along the overlap at a maximum spacing of 36 inches (914 mm) on center.

Underlayment installed where ~~V_{asd}, in accordance with Section 1609.3.1,~~ the ultimate design wind speed, V_{ult} equals or exceeds ~~120~~ 140 mph (54 m/s) shall comply with ASTM D 226 Type II, ASTM D 4869 Type IV, or ASTM D 6757. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with Section 1507.2.8 except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a ~~head~~ cap diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall comply with Section 1507.2.6.1 and shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of 3/4 inch (19.1 mm) into the roof sheathing.

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

1507.3.3.3 High wind attachment. Underlayment applied in areas subject to high wind ~~[V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1]~~ [V_{ult} equal or greater than 130 mph] shall be applied with minimum 12 gage [0.105 inch (2.67 mm) corrosion-resistant ~~fasteners in accordance with the manufacturer's installation instructions~~ annular ring shank nails having not less than 20 rings per inch, with a minimum 3/8 inch-diameter (9.5 mm) head, of a length to penetrate through the roofing sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where ~~V_{asd}, in accordance with Section 1609.3.1,~~ the ultimate design wind speed, V_{ult} equals or exceeds ~~120~~ 140 mph (54 m/s) shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with Sections 1507.3.3.1 and 1507.3.3.2 except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. ~~The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of 3/4 inch (19.1 mm) into the roof sheathing.~~

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

1507.3.6.1 Fasteners and high winds. In areas where the ultimate design wind speed, V_{ult} equals or exceeds 130 mph, fasteners for tile shall be a minimum 11 gage [0.105 inch (2.67 mm)] annular ring shank nails having not less than 20 rings per inch shank, with a minimum 5/16 inch-diameter (9.5 mm) head, of a length to penetrate through the roofing sheathing or a minimum of 3/4 inch (19.1 mm) into the roof sheathing.

1507.4.5 Underlayment and high wind. Underlayment applied in areas subject to high winds ~~[V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1]~~ [V_{ult} equal to or greater than 130 mph] shall be applied with minimum 12 gage [0.105 inch (2.67 mm) corrosion-resistant ~~fasteners in accordance with the manufacturer's installation instructions.~~ annular ring shank nails having not less than 20 rings per inch, with a minimum 3/8 inch-diameter (9.5 mm) head, of a length to penetrate through the roofing sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where ~~V_{asd}, in accordance with Section 1609.3.1,~~ the ultimate design wind speed, V_{ult} equals or exceeds ~~120~~ 140 mph (54 m/s) shall comply with ASTM D 226 Type II, ASTM D 4869 Type IV, or ASTM D 1970. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. ~~The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of 3/4 inch (19.1 mm) into the roof sheathing.~~

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

1507.5.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds ~~[V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1]~~ [V_{ult} equal to or greater than 130 mph shall be applied with minimum 12 page [0.105 inch (2.67 mm) corrosion-resistant ~~fasteners in accordance with the manufacturer's installation instructions~~ annular ring shank nails having not less than 20 rings per inch, with a minimum 3/8 inch-diameter (9.5 mm) head, of a length to penetrate through the roofing sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where ~~V_{asd}, in accordance with Section 1609.3.1~~ the ultimate design wind speed, V_{ult}, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch spacing (152 mm) at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a ~~head~~ cap diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. ~~The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.~~

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

1507.6.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds ~~[V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1]~~ [V_{ult} equal to or greater than 130 mph] shall be applied with minimum 12 gage [0.105 inch (2.67 mm) corrosion resistant ~~fasteners in accordance with the manufacturer's installation instructions.~~ annular ring shank nails having not less than 20 rings per inch, with a minimum 3./8 inch-diameter (9.5 mm) head, of a length to penetrate through the roofing sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where ~~V_{asd}, in accordance with Section 1609.3.1,~~ the ultimate design winds peed, V_{ult} equals or exceeds ~~120~~ 140 mph (54 m/s) shall comply with ASTM D 226 Type II. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a ~~head~~ cap diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. ~~The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of 3/4 inch (19.1 mm) into the roof sheathing.~~

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

1507.7.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds ~~[V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1]~~ [V_{ult} equal to or greater than 130 mph] shall be applied with minimum 12 gage [0.105 inch (2.67 mm) corrosion resistant ~~fasteners in accordance with the manufacturer's installation instructions.~~ annular ring shank nails having not less than 20 rings per inch, with a minimum 3/8 inch-diameter (9.5 mm) head, of a length to penetrate through the roofing sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where ~~V_{asd}, in accordance with Section 1609.3.1,~~ the ultimate design wind speed, V_{ult} equals or exceeds ~~120~~ 140 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a ~~head~~ cap diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. ~~The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of 3/4 inch (19.1 mm) into the roof sheathing.~~

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

1507.8.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds ~~[V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1]~~ [V_{ult} equal to or greater than 130 mph shall be applied with minimum 12 gage [0.105 inch (2.67 mm) corrosion resistant fasteners ~~in accordance with the manufacturer's installation instructions.~~ annular ring shank nails having not less than 20 rings per inch, with a minimum 3/8 inch-diameter (9.5 mm) head, of a length to penetrate through the roofing sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where ~~V_{asd}, in accordance with Section 1609.3.1,~~ the ultimate design wind speed, V_{ult} equals or exceeds ~~120~~ 140 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a ~~head~~ cap diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. ~~The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.~~

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

1507.9.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds ~~[V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1]~~ [V_{ult} equal to or greater than 130 mph] shall be applied with minimum 12 gage [0.105 inch (2.67 mm) corrosion resistant ~~fasteners in accordance with the manufacturer's installation instructions.~~ annular ring shank nails having not less than 20 rings per inch, with a minimum 3/8 inch-diameter (9.5 mm) head, of a length to penetrate through the roofing sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where ~~V_{asd}, in accordance with Section 1609.3.1,~~ the ultimate design wind speed, V_{ult} equals or exceeds ~~120~~ 140 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a ~~head~~ cap diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. ~~The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.~~

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

Reason: Water intrusion continues to be an issue with hurricanes and high wind events. Significant improvements have been made recently to the codes and other voluntary methods that help prevent water intrusion through the roof decking when the primary roof covering has been blown off or damaged. These include the underlayment and high wind requirements in the 2012 IBC and the 2012 IRC in addition to the Sealed Roof Deck provisions recommended by the IBHS Fortified program and FEMA hurricane retrofit program guidance. However, recent tests on sealed roof decks at the IBHS Research Center indicate that water intrusion through nail holes left in the roof decking when the primary roof covering has been lost is still an issue. In the areas specified, this code change proposal requires the roof underlayment to be attached with ring shank nails. Where nails are specified for the roof covering attachment, this code change proposal requires the use of ring shank nails. Ring shank nails have a significantly higher withdrawal capacity to similar sized smooth shank nails (up to 131% higher). The use of ring shank nails will help keep the nails in place when the roof covering is blow off and reduce the chance that unfilled nail holes will allow water intrusion.

This code change proposal also changes the wind speed trigger for when the improved underlayment and fastening methods are required. The wind speed is changed to a V_{ult} value consistent with the wind speeds represented in Figures 1609A, 1609B, and 1609C. Additionally, the wind speed threshold that triggers the improved underlayment and fastening methods has been slightly reduced. The proposed 130 mph and 140 mph V_{ult} wind speed triggers are more comparable geographically to the 110 mph and 120 mph wind speeds in the 2009 IBC. The triggers are also consistent with the wind speed limitations on conventional construction and the prescriptive non-high wind provisions of the 2012 IRC (The Wind Design Required Region in the 2012 IRC is tied to the 130 mph V_{ult} wind speed). Post-storm investigations also show that water intrusion is an issue in inland areas when the primary roof covering has been blown off.

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

1507.2.6.1 (NEW)-S-STAFFORD

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The proposed change to the wind speed threshold for underlayment in high wind regions was more than a conversion from nominal to ultimate design wind speeds. The more restrictive threshold that was proposed seemed arbitrary in that insufficient technical justification was given for this change.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

T. Eric Stafford, T. Eric Stafford & Associates, LLC, representing Insurance Institute for Business and Home Safety (IBHS), requests Approval as Submitted.

Commenter's Reason: We are seeking Approval as Submitted for S31-12. There were two primary opposition points to this proposal. The first was that the change to the wind speed threshold for underlayment in high wind regions was more than just a conversion from nominal to ultimate design wind speeds. This is true, and the reason statement for S31-12 clearly states this. When this code change was adopted, a separate proposal was approved that updated the wind speed maps in the IBC to be consistent with the strength-design level maps in ASCE 7-10. The original 120 mph threshold was chosen, largely based on engineering judgment, to apply to areas that had the highest risk of an impact from a Category III or higher hurricane. The proposed V_{ult} equal to or exceeding 140 mph threshold is approximately consistent geographically with the 120 mph contour on the wind speed maps in the 2009 IBC and ASCE 7-05.

The second point of opposition was primarily to specifying the use of ring shank nails for attaching the roof covering and the underlayment in areas where V_{ult} equal to or exceeding 130 mph. The opposition was not due to cost, as the cost of using ring shank nails over smooth shank nails is negligible. The debated centered on the supposed lack of specification for the nail and whether or not this nail was covered by ASTM F 1667. Deformed shank nails are specifically covered by ASTM F 1667. Section 10.3 in ASTM F 1667, *Altered Shapes and Dimensions*, specifically addresses mechanically formed or deformed nail shanks. In fact, deformed shank shingle and underlayment nails are specifically addressed in other sections of ASTM F 1667. Ring shank nails have a significantly higher withdrawal capacity to similar sized smooth shank nails. The use of ring shank nails will help keep the nails in place when the roof covering is blown off and reduce the chance that unfilled nail holes will allow water intrusion into the building.

S31-12

Final Action: AS AM AMPC____ D

S33-12
1507.2.8.2

Proposed Change as Submitted

Proponent: Bill McHugh, Chicago Roofing Contractors Association (bill@crca.org)

Revise as follows:

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

Exceptions:

<u>1.</u> Detached accessory structures that contain no conditioned floor area.
2. <u>Roofs with slope equal to or greater than 8/12, the ice barrier shall be applied to a point 36 inches (914 mm) past the outside part of the inside wall line of the building up the slope of the roof deck.</u>

Reason: The Chicago Roofing Contractors Association (CRCA) and other steep slope roofing contractors work in all climates from hot summer to the dead of cold, snowy winters. We have enough snow most years to get much experience in ice dam situations.

In steep slope applications in climates where ice forms at the eave edge of roofs. Ice melts due to heat from below melting snow, then freezes where the water meets roof surfaces that are over unheated areas, making a buildup of ice. This buildup becomes a 'dam' that backs water up under the roof covering and underlayment leaking into the building.

The purpose of this proposal is to bring to the Code into alignment with the practical application of the ice barrier underlayment products in the field. Since gravity stops water from backing up very far on super steep slopes greater than 8" in 12" there needs to be a limit to the amount of ice barrier underlayment applied.

On very steep sloped roofs, the ice dams will still occur. However, buildup of ice cannot build far beyond the ball that forms at the gutter edge on slopes greater than 8" in 12". Secondly, the water will not defy gravity and move very far upward, when the physics of the application are that the water will drip over the dam due to gravity first.

The way the current code is written, ice barrier material may be needed on the complete roof deck rather than to protect just the eave edges and 3' up slope. Through clarifying this requirement with the exception, the intent of the code is met while reducing costs to builders and building owners and managers.

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

1507.2.8.2 #1-S-MCHUGH

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee felt the proponent may have a good idea and perhaps it should be added to the base requirements for ice barriers rather than formatted as a new exception. The actual overhang length is not addressed and there is a problem with the 8:12 slope or greater.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Bill McHugh, Chicago Roofing Contractors Association, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

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

Exceptions:

1. Detached accessory structures that contain no conditioned floor area.
2. On ~~roofs with~~ roof slopes ~~equal to or greater than 8/12~~, not less than 8 units vertical in 12 units horizontal the ice barrier shall be applied from the eave to a point 36 inches (914 mm) ~~past the outside part of the inside wall line of the building~~ measured up the slope of the roof deck.

Commenter's Reason: The Chicago Roofing Contractors Association (CRCA) and other steep slope roofing contractors work in all climates from hot summer to the dead of cold, snowy winters. We have enough snow most years to be familiar with ice dam situations.

In steep slope roofs in climates where ice forms at the eave edge of roofs due to heat from below contacting snow on roofs. The Ice that melts due to heat from below melting snow then freezes where the water meets roof surfaces that are over unheated areas, creating a buildup of ice. This buildup becomes a 'dam' that backs water up under the roof covering and underlayment leaking into the building.

The purpose of this proposal is to bring to the Code into alignment with the practical application of the ice barrier underlayment products in the field on 'super steep' slope roofs. Since gravity stops water from backing up very far on slopes greater than 8" in 12" there needs to be a limit to the amount of ice barrier underlayment required by the code.

The way the current code is written, on a 'mansard roof'' the slope may require full coverage of the mansard to comply. Therefore, more ice barrier material may be needed on the complete roof deck rather than to protect just the eave edges and 3' up slope.

Through clarifying this requirement with the exception, the intent of the code is met while reducing extraneous costs to developers, building owners and managers and construction firms. On roofs sloped 8" in 12" and greater, ice dams may occur. However, the resulting ice formation cannot extend vertically upslope far.

Secondly, the resulting ice dam cannot defy gravity and traverse vertically upslope due to the physics of the application. Water will drip over the ball shaped dam due to gravity rather than keep backing upslope. When calculating the distance up the slope the ice barrier membrane applied seems to equate to 36" up the slope. Rolls of ice barrier material are supplied in 36" wide rolls.

S33-12

Final Action: AS AM AMPC____ D

S34-12
1507.2.8.2

Proposed Change as Submitted

Proponent: Bill McHugh, Chicago Roofing Contractors Association (bill@crca.org)

Revise as follows:

1507.2.8.2 Ice barrier. In areas where there has been a history of ice forming along the eaves causing a backup of water, an ice barrier that consists of at least two layers of underlayment cemented together or of a self adhering polymer modified bitumen sheet shall be used in lieu of normal underlayment and extend <u>2 inches (51 mm) down the fascia and under the drip edge,</u> from the lowest edges of all roof surfaces to a point at least 24 inches (610 mm) inside the *exterior wall* line of the building.

Exceptions:

<u>1.</u> Detached accessory structures that contain no conditioned floor area.
<u>2. Roof recover applications where no new metal drip edges or gutters are incorporated.</u>

Reason: The Chicago Roofing Contractors Association (CRCA) and other steep slope roofing contractors work in all climates from hot summer to the dead of cold, snowy winters. We have enough snow most years to get much experience in ice dam situations.

In steep slope applications in climates where ice forms at the eave edge of roofs. Ice melts due to heat from below melting snow, then freezes where the water meets roof surfaces that are over unheated areas, making a buildup of ice. This buildup becomes a 'dam' that backs water up under the underlayment and roof covering.

Studies show that roof recover applications typically fail at flashings on all roof slopes. The roof edge flashings are most susceptible to leaks from water backing up under the underlayment and roof covering because it freezes at the eave edge first driving water up-slope.

According to CRCA roofing contractors, if the code required ice barrier is applied improperly to the top of the metal drip edge, the water will leak into the structure. The leak(s) may be difficult to detect in the concealed space location.

In new construction, tear off and roof replacement situations the roofing underlayment construction is easily phased to be installed before the drip edges at the eave edge.

In roof recover applications where metal is not removed, surfaces may be dirty, uneven, and very difficult even for the best contractors to provide a water tight seal.

To provide the building owner the best application and give the code requirement the best chance at working as intended, this proposal from the Chicago Roofing Contractors Association is presented.

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

1507.2.8.2 #2-S-MCHUGH

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The wording of this proposal needs work. The requirement to extend underlayment 2 inches down the fascia should be separated from the current phrase "from the lowest edges". Placing the recover application in an exception could appear to eliminate the ice barrier.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Bill McHugh, Chicago Roofing Contractors Association, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1507.2.8.2 Ice barrier. In areas where there has been a history of ice forming along the eaves causing a backup of water, an ice barrier that consists of at least two layers of underlayment cemented together or of a self adhering polymer modified bitumen sheet shall be used in lieu of normal underlayment and extend 2 inches (51 mm) down the fascia and under the drip edge, from the lowest edges of all roof surfaces to a point at least 24 inches (610 mm) inside the *exterior wall* line of the building.

Exception:

1. Detached accessory structures that contain no conditioned floor area
2. ~~Roof recover applications where no new metal drip edges or gutters are incorporated.~~

Commenter's Reason: The Chicago Roofing Contractors Association (CRCA) and other steep slope roofing contractors work in all climates from hot summer to the dead of cold, snowy winters. We have enough snow most years to get much experience in ice dam situations.

In steep slope applications in climates where ice forms at the eave edge of roofs. Ice melts due to heat from below melting snow, then freezes where the water meets roof surfaces that are over unheated areas, making a buildup of ice. This buildup becomes a 'dam' that backs water up under the underlayment and roof covering.

Studies show that roof recover applications typically fail at flashings on all roof slopes. The roof edge flashings are most susceptible to leaks from water backing up under the underlayment and roof covering because it freezes at the eave edge first driving water up-slope.

According to CRCA roofing contractors, if the code required ice barrier is applied improperly to the top of the metal drip edge, the water will leak into the structure. The leak(s) may be difficult to detect in the concealed space location.

In new construction, tear off and roof replacement situations the roofing underlayment construction is easily phased to be installed before the drip edges at the eave edge.

In roof recover applications where metal is not removed, surfaces may be dirty, uneven, and very difficult even for the best contractors to provide a water tight seal, hence removing the exception we proposed in May in Dallas as was pointed out by the committee. We believe this clarifies the proposal as the committee recommended.

To provide the building owner the best application and give the code requirement the best chance at working as intended, this proposal from the Chicago Roofing Contractors Association is presented.

The proposed exception is removed by this modification in response to the committee reason.

S34-12

Final Action: AS AM AMPC____ D

S35-12
1507.2.8.2

Proposed Change as Submitted

Proponent: Bill McHugh, Chicago Roofing Contractors Association (bill@crca.org)

Revise as follows:

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

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

Reason: In a survey of CRCA Steep & Shingle Committee Members it appears this method for ice barrier protection is no longer used due to labor intensive and messy application.

At the time the ice barrier materials were introduced to the code, this was an application used because the ice barrier materials were not in the code. After years of use, it seems the two layers of underlayment cemented together method is not used as it is much more costly than the self adhering polymer modified bitumen sheet materials.

Therefore, we propose to remove this option from the code.

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

1507.2.8.2 #3-S-MCHUGH

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: There is no need to eliminate the option of two layers of underlayment cemented together. It is still a valid application and retaining it keeps the minimum code requirements.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Bill McHugh, Chicago Roofing Contractors Association, requests Approval as Submitted.

Commenter's Reason: The code currently allows an option of either 'at least two layers of (felt) underlayment cemented together' or a 'self adhering polymer modified bitumen' sheet instead. The ice barrier sheets were developed in the late 1970's and mastic layers used prior to that time widely.

In a survey of CRCA Steep & Shingle Committee Members and others currently in the roofing contractor industry, it appears this 'two layers of felt underlayment with roof cement method for ice barrier protection used very infrequently and seems to provide a risky application as well. The method of using wet mastics to felt in layers is no longer used due to safety concerns, labor intensive costs, and displacement when stepping on the material before cure of the mastics that can cause falls on or from the roof.

There is an alternative to the 'mastic and felt underlayment' method of underlayment. The alternative is an ice barrier sheet. These products are widely available with several manufacturers of this product providing competition and alternatives. The products are also available worldwide through wholesale distributors and retail outlets in all 50 states and internationally as well.

Secondly, the mastic and underlayment method has technical limitations. In order to apply shingles over the mastic and felt, a worker must walk on the application. If the worker walks before the material is fully cured, the worker's foot may displace the mastic forming an undetectable void under the 1st layer of underlayment and also under shingles. A workers hammer may also displace the material leaving a void.

Third, if the mastic is not fully cured, the worker, even tied off, is more likely to fall due to a slippery mass of material under the felt which may move under his or her feet. This can cause slips and possibly falls on the roof or off the roof to the ground.

Fourth, the labor intensive method that the material is applied could be better used more efficiently.

This was an application allowed by the code prior to the ice barrier materials being invented and available to allow in the code. After years of use, it seems the 'two layers of underlayment cemented together' method is not used as it is much less efficient than the self adhering polymer modified bitumen sheet materials.

For safety and practical application we propose to remove this option from the code.

S35-12

Final Action: AS AM AMPC____ D

S37-12

1507.2.8.1, Table 1507.2.8.1 (New), 1507.3.3.3, 1507.4.5, 1507.5.3.1, 1507.6.3.1, 1507.7.3.1, 1507.8.3.1, 1507.9.3.1

Proposed Change as Submitted

Proponent: John Kurtz, International Staple, Nail & Tool Association (isanta@ameritech.net)

Revise as follows:

1507.2.8.1 High wind attachment. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's instructions. Fasteners are to be applied along the overlap at a maximum spacing of 36 inches (914 mm) on center.

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

Exceptions:

1. As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.
2. As an alternative, cap nails and cap staples complying with requirements of ASTM F1667 and fastened in accordance with Table 1507.2.8.1 shall be permitted.

TABLE 1507.2.8.1
ROOF COVERING UNDERLAYMENT ATTACHMENT

Alternate Fastener [a]	Maximum center-to-center spacing of alternate fasteners and grid lines if required center-to-center spacing of code fastener is	
	6” (152 mm) o.c.	12” (305 mm) o.c.
5/8” leg, 21 gage staple	3” (76 mm)	6” (152 mm)
21 gage staple	3” (76 mm)	7” (178 mm)
20 gage staple	4” (102 mm)	8” (203 mm)
0.080 -.083 diam. nail	4” (102 mm)	9” (229 mm)
0.090 diam. Nail 18 gage staple	5” (127 mm)	10” (254 mm)
0.105 diam. Nail (12 gage) 17 gage staple 0.120 diam. nail (11 gage)	6” (152 mm)	12” (305 mm)

a. Minimum nail shank length or staple leg length is 3/4” (19 mm) unless otherwise stated.

1507.3.3.3 High wind attachment. Underlayment applied in areas subject to high wind [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the manufacturer’s installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

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

Exceptions:

1. As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.
2. As an alternative, cap nails and cap staples complying with requirements of ASTM F1667 and fastened in accordance with Table 1507.2.8.1 shall be permitted.

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

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

Exceptions:

1. As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.
2. As an alternative, cap nails and cap staples complying with requirements of ASTM F1667 and fastened in accordance with Table 1507.2.8.1 shall be permitted.

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

Underlayment installed where V_{asd}, in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch spacing (152 mm) at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.

Exceptions:

1. As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.
2. As an alternative, cap nails and cap staples complying with requirements of ASTM F1667 and fastened in accordance with Table 1507.2.8.1 shall be permitted.

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

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

Exceptions:

1. As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.
2. As an alternative, cap nails and cap staples complying with requirements of ASTM F1667 and fastened in accordance with Table 1507.2.8.1 shall be permitted.

1507.7.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

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

Exceptions:

1. As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.
2. As an alternative, cap nails and cap staples complying with requirements of ASTM F1667 and fastened in accordance with Table 1507.2.8.1 shall be permitted.

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

Underlayment installed where V_{asd}, in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.

Exceptions:

1. As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.
2. As an alternative, cap nails and cap staples complying with requirements of ASTM F1667 and fastened in accordance with Table 1507.2.8.1 shall be permitted.

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

Underlayment installed where $V_{asd,}$ in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.

Exceptions:

1. As an alternative, adhered underlayment complying with ASTM D 1970 shall be permitted.
2. As an alternative, cap nails and cap staples complying with requirements of ASTM F1667 and fastened in accordance with Table 1507.2.8.1 shall be permitted.

Reason: The fastener listed for attachment of roof covering underlayment in high-wind areas does not reflect commercially available fasteners successfully used in roofing material application. The code presently lists only one nail shank diameter, 0.105". This proposal addresses both commercially available hand-driven and power-driven cap-fasteners.

Tighter spacing of fasteners specified in the proposed table ensures that spacing of fasteners with diameters not currently specified in the Code would achieve equal (or greater) withdrawal strength than the currently listed nail diameter. Sufficient fastener withdrawal ensures that fastener shanks remain in roof deck while cap transfers uplift forces to the deck. This is a conservative approach because developing data indicates that the relevant failure mode is cap pulling through underlayment, rather than fastener shank withdrawal.

ASTM F1667-11a controls fastener nominal dimensions and tolerances as well as relevant fastener features.
Structure of proposal minimizes complexity of code requirements. An "Exception" is added to each roof covering's section. One table presents fastener spacing for all roof coverings.

Cost Impact: The code change proposal will not increase the cost of construction. The numerous options would allow contractors to select options which provide equivalent protection with minimized material and labor costs.

T1507.2.8.1(NEW)-S-KURTZ.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee believes the proposal to have merit but some corrections are needed. There are some questions as to the minimum size of the alternative cap nails. Test data should be examined and provided to the committee.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

John Kurtz, Executive Vice President, International Staple, Nail & Tool Association, requests Approval as Modified by this Public Comment.

Replace the proposal as follows:

1507.2.8.1 High wind attachment. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's instructions. Fasteners are to be applied along the overlap at a maximum spacing of 36 inches (914 mm) on center.

Underlayment installed where V_{asd}, in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II, ASTM D 4869 Type IV, or ASTM D 6757. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with Section 1507.2.8 except all laps shall be a minimum of 4 inches (102 mm). ~~Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of 3/4 inch (19.1 mm) into the roof sheathing.~~ Underlayment shall be attached using metal or plastic cap nails or staples with a nominal cap diameter of not less than 1 inch (25 mm.) Hand-driven metal caps shall have a minimum thickness of 0.030 inch (0.76 mm). Power-driven metal caps shall have a minimum thickness of 0.010 inch (0.25 mm). Minimum thickness of the outside edge of plastic caps shall be 0.035 inch (0.89 mm). Cap nail ring shank diameter shall be a minimum of 0.083 inch (2.11 mm). Cap nail smooth shank diameter shall be a minimum of 0.091 inch (2.31 mm). Staple gage shall be a minimum 21 gage. Cap fasteners shall have a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.

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

1507.3.3.3 High wind attachment. Underlayment applied in areas subject to high wind [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where $V_{asd,}$ in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with Sections 1507.3.3.1 and 1507.3.3.2 except all laps shall be a minimum of 4 inches (102 mm). ~~Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of 3/4 inch (19.1 mm) into the roof sheathing.~~ Underlayment shall be attached using metal or plastic cap nails or staples with a nominal cap diameter of not less than 1 inch (25 mm.) Hand-driven metal caps shall have a minimum thickness of 0.030 inch (0.76 mm). Power-driven metal caps shall have a minimum thickness of 0.010 inch (0.25 mm). Minimum thickness of the outside edge of plastic caps shall be 0.035 inch (0.89 mm). Cap nail ring shank diameter shall be a minimum of 0.083 inch (2.11 mm). Cap nail smooth shank diameter shall be a minimum of 0.091 inch (2.31 mm). Staple gage shall be a minimum 21 gage. Cap fasteners shall have a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.

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

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

Underlayment installed where V_{asd}, in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II, ASTM D 4869 Type IV, or ASTM D 1970. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). ~~Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of 3/4 inch (19.1 mm) into the roof sheathing.~~ Underlayment shall be attached using metal or plastic cap nails or staples with a nominal cap diameter of not less than 1 inch (25 mm.) Hand-driven metal caps shall have a minimum thickness of 0.030 inch (0.76

mm). Power-driven metal caps shall have a minimum thickness of 0.010 inch (0.25 mm). Minimum thickness of the outside edge of plastic caps shall be 0.035 inch (0.89 mm). Cap nail ring shank diameter shall be a minimum of 0.083 inch (2.11 mm). Cap nail smooth shank diameter shall be a minimum of 0.091 inch (2.31 mm). Staple gage shall be a minimum 21 gage. Cap fasteners shall have a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.

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

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

Underlayment installed where V_{asd}, in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch spacing (152 mm) at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). ~~Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.~~ Underlayment shall be attached using metal or plastic cap nails or staples with a nominal cap diameter of not less than 1 inch (25 mm.) Hand-driven metal caps shall have a minimum thickness of 0.030 inch (0.76 mm). Power-driven metal caps shall have a minimum thickness of 0.010 inch (0.25 mm). Minimum thickness of the outside edge of plastic caps shall be 0.035 inch (0.89 mm). Cap nail ring shank diameter shall be a minimum of 0.083 inch (2.11 mm). Cap nail smooth shank diameter shall be a minimum of 0.091 inch (2.31 mm). Staple gage shall be a minimum 21 gage. Cap fasteners shall have a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.

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

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

Underlayment installed where V_{asd}, in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. ~~Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32- gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of 3/4 inch (19.1 mm) into the roof sheathing.~~ Underlayment shall be attached using metal or plastic cap nails or staples with a nominal cap diameter of not less than 1 inch (25 mm.) Hand-driven metal caps shall have a minimum thickness of 0.030 inch (0.76 mm). Power-driven metal caps shall have a minimum thickness of 0.010 inch (0.25 mm). Minimum thickness of the outside edge of plastic caps shall be 0.035 inch (0.89 mm). Cap nail ring shank diameter shall be a minimum of 0.083 inch (2.11 mm). Cap nail smooth shank diameter shall be a minimum of 0.091 inch (2.31 mm). Staple gage shall be a minimum 21 gage. Cap fasteners shall have a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.

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

1507.7.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [V_{asd} greater than 110 mph (49 m/s) as determined in accordance with Section 1609.3.1] shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where V_{asd}, in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). ~~Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of 3/4 inch (19.1 mm) into the roof sheathing.~~ Underlayment shall be attached using metal or plastic cap nails or staples with a nominal cap diameter of not less than 1 inch (25 mm.) Hand-driven metal caps shall have a minimum thickness of 0.030 inch (0.76 mm). Power-driven metal caps shall have a minimum thickness of 0.010 inch (0.25 mm). Minimum thickness of the outside edge of plastic caps shall be 0.035 inch (0.89 mm). Cap nail ring shank diameter shall be a minimum of 0.083 inch (2.11 mm). Cap nail smooth shank diameter shall be a minimum of 0.091 inch (2.31 mm). Staple gage shall be a minimum 21 gage. Cap fasteners shall have a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.

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

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

Underlayment installed where V_{asd}, in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). ~~Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.~~ Underlayment shall be attached using metal or plastic cap nails or staples with a nominal cap diameter of not less than 1 inch (25 mm.) Hand-driven metal caps shall have a minimum thickness of 0.030 inch (0.76 mm). Power-driven metal caps shall have a minimum thickness of 0.010 inch (0.25 mm). Minimum thickness of the outside edge of plastic caps shall be 0.035 inch (0.89 mm). Cap nail ring shank diameter shall be a minimum of 0.083 inch (2.11 mm). Cap nail smooth shank diameter shall be a minimum of 0.091 inch (2.31 mm). Staple gage shall be a minimum 21 gage. Cap fasteners shall have a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.

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

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

Underlayment installed where $V_{asd,}$ in accordance with Section 1609.3.1, equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6-inch (152 mm) spacing at the side laps. Underlayment shall be applied in accordance with the manufacturer's installation instructions except all laps shall be a minimum of 4 inches (102 mm). ~~Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 inch (25 mm) with a thickness of at least 32-gauge [0.0134 inch (0.34 mm)] sheet metal. The cap nail shank shall be a minimum of 12 gauge [0.105 inch (2.67 mm)] with a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.~~ Underlayment shall be attached using metal or plastic cap nails or staples with a nominal cap diameter of not less than 1 inch (25 mm.) Hand-driven metal caps shall have a minimum thickness of 0.030 inch (0.76 mm). Power-driven metal caps shall have a minimum thickness of 0.010 inch (0.25 mm). Minimum thickness of the outside edge of plastic caps shall be 0.035 inch (0.89 mm). Cap nail ring shank diameter shall be a minimum of 0.083 inch (2.11 mm). Cap nail smooth shank diameter shall be a minimum of 0.091 inch (2.31 mm). Staple gage shall be a minimum 21 gage. Cap fasteners shall have a length to penetrate through the roof sheathing or a minimum of ¾ inch (19.1 mm) into the roof sheathing.

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

Commenter's Reason: Purpose of Public Comment is to broaden IBC to include cap fasteners established in building construction. This means (1) cap nails with smaller nail diameters than the IBC's 0.105" nail shank diameter (down to 0.083"), and (2) cap staples (21 gage and larger.) All proposed cap fasteners have the same 1" diameter cap.

Initial S37 suggested tighter spacing of expanded cap fasteners with lower withdrawal strength than the 0.105" nail. (At the time we feared fastener shank withdrawal failures.)

A Floor Amendment proposed same spacing for all cap fasteners because testing with ASTM D 226, Type I ("15 pound felt") showed that underlayment tore before cap fasteners failed.

Subsequently, we did further testing with ASTM D 4869 Type IV underlayment ("30 pound"). That underlayment is at high end of the thickness and toughness range of code required underlayment - a "worst-case test" for the fastener.

Test results indicate that cap nails of minimum diameter 0.083" and cap staples of minimum 21 gage may be used in place of the cap nail required by the IBC. Average failure force of every additional fastener exceeded IBC fastener with D 4869 Type IV underlayment. Failure forces approximately doubled with heavier underlayment.

Based on testing, S37 has been simplified to broaden the description of cap fasteners in "Underlayment" sections for each roof covering.

Cap fastener descriptions are based on the relevant ASTM specification, ASTM F1667.

Test procedure and results accompany this proposal.

Report on Testing
July 2012

Testing was performed by Stanley Black & Decker at the request of International Staple, Nail and Tool Association (ISANTA.)

Reference Standards

State of Florida

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

ASTM Standards

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

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

Materials

- Roofing paper, 30# (ASTM D 4869, Type IV)
- Sheathing material – 4-ply, 15/32-in. Southern Pine Plywood, cut in 2 by 2 in. squares
- Fasteners – Ring shank cap nails with nail shank diameters before threading of 0.083 inch and 0.105 inch. Cap staples, 18 gage and 21 gage.
- Caps – 1 inch diameter plastic caps

Method

The test method was designed to facilitate one of three potential failure modes: cap failure, fastener withdrawal, or cap pulling through underlayment. A 14x14-in. sheet of underlayment was cut from the roll. The cap-fastener was driven through the center of the underlayment sheet into a 2x2-in. block of sheathing material. The assembled test specimen was turned over so that the sheathing block was visible and the fastener head was down. The assembled specimen was secured in the test fixture base with the fastener centered below sheathing block clamping fixture. The sheathing block was clamped by the fixture attached to the traversing head of the test machine. The test specimen was loaded at constant displacement of 1 in./min. until failure. Load and displacement were monitored continuously during the test. Failure mode was observed and peak force was recorded as the failure load. Photographs provided.

Discussion

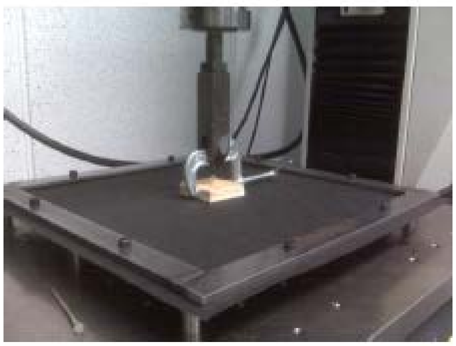
The test is intended to evaluate the functionality of the ISANTA proposal for adding additional commercially available cap fasteners for use on same spacing as IBC's 0.105" cap nail with a plastic or metal 1" diameter cap (as specified.) The underlayment is not wind qualified. However, AC188 evaluation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

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

Conclusions

From the testing and review of test standards and acceptance criteria, we can conclude that the underlayment minimum tensile strength is the controlling strength property of the system and it can be used as a reasonable approximation of the potential holding capacity of the cap-fasteners based on the cap diameter. Engineering analysis of the negative pressures on roof surfaces should provide reasonable estimates of expected forces that will be resisted by fasteners and can be used to establish fastening schedules that reflect the fastener holding capacity (pull-through or withdrawal) and tensile strength of the underlayment when loaded as a membrane between fasteners.

Test machine fixtures for the pull-through test.



Pull-through test in progress; (left) early in test; (right) nearing failure.

Metal cap with roofing nail fastener after the pull-through test. Observe the permanent deformation of the metal cap and the pull-through tears in the underlayment.

Results of Cap Fastener Testing with ASTM D 4869, Type IV Underlayment

Cap Fastener [1]	Failure Load (pounds)	Number of Failures, by Failure Mode		
		Fastener Withdrawal	Cap Failure	Under-layment Tear
<u>“Code” Nail</u> 2012 IBC Cap Nail 0.105” nail diameter ring shank nail	31.8	1	7	8
0.083” nail diameter ring shank nail	32.4	0	4	2
21 Gage staple	36.2	0	0	5
18 Gage staple	32.1	0	2	9

[1] All cap fasteners had plastic caps meeting IBC requirements.

S37-12

Final Action: AS AM AMPC____ D

S51-12

202 (New), 1509 (New), 1509.1 (New), 1509.2 (New), 1509.3 (New), Chapter 35 (New)

Proposed Change as Submitted

Proponent: Ken Sagan, NRG Code Advocates, representing Reflective Insulation Mfg. Assoc. International (ken@nrgcodeadvocates.com)

THIS CODE CHANGE PROPOSAL IS ON THE AGENDA OF THE IBC FIRE SAFETY CODE DEVELOPMENT COMMITTEE. SEE THE HEARING ORDER FOR THE IBC FIRE SAFETY CODE DEVELOMPENT COMMITTEE.

Add new text as follows:

SECTION 202
DEFINITIONS

RADIANT BARRIER. A material having a low emittance surface (0.1 or less) and where installed in building assemblies, the low emittance surface shall face a ventilated or unventilated air space.

Add new text as follows:

SECTION 1509
RADIANT BARRIER-ABOVE DECK

1509.1 General. The use of above-deck radiant barriers shall be permitted provided that the radiant barrier is covered with an *approved* roof covering and passes the tests of FM 4450 or UL 1256 when tested as an assembly.

1509.2 Radiant barrier. Installed above-deck shall have a continuous 0.5 inch (minimum) air space on the low emittance side of the product.

1509.3 Material standards, Above-deck radiant barrier shall comply with ASTM C1313/1313M

Add new standard to Chapter 35 as follows:

ASTM

C1313/C1313M-10 Standard Specification for Sheet Radiant Barriers for Building Construction Applications

Reason: There is a common misunderstanding in the market that some radiant barrier products installed above-deck, typically between the deck and the felt, provide some level of thermal benefit. This is not the case and this proposal intends to clarify the air gap requirements for above-deck radiant barriers.

References:
ASTM C1313/C1313M-10 Standard Specification for Sheet Radiant Barriers for Building Construction Applications

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] 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, 2012.

1509-S-SAGAN

Public Hearing Results

This code change was heard by the IBC Fire Safety code development committee.

Note: For staff analysis of the content of ASTM C 1313 relative to CP#28, Section 3.6, please visit:
http://www.iccsafe.org:8888/cs/codes/Documents/2012-13cycle/Proposed-A/00a_updates.pdf

Committee Action: Disapproved

Committee Reason: The committee felt that the proposal needed too many modifications; the proponent wants to substitute an updated version of the standard, modification of the definition of "radiant barrier" is suggested to be consistent with industry standards and clarification of the radiant barrier airspace as being minimum or maximum in necessary.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment 1:

Marcelo M Hirschler, GBH International, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

Section 202 - Definitions

RADIANT BARRIER. A material having a low emittance surface of ~~(~~0.1 or less~~)~~ ~~and when~~ installed in building assemblies~~, the low emittance surface shall face a ventilated or unventilated air space~~.

SECTION 1509
RADIANT BARRIER-ABOVE DECK

1509.1 General. ~~The use of above-deck radiant barriers shall be permitted provided that the radiant barrier is covered with an *approved* roof covering and passes the tests of FM 4450 or UL 1256 when tested as an assembly.~~ <u>A radiant barrier installed above a deck shall comply with Sections 1509.2 through 1509.4.</u>

~~**1509.2 Radiant barrier.** Installed above-deck shall have a continuous 0.5 inch (minimum) air space on the low emittance side of the product.~~

<u>**1509.2 Fire Testing.** Radiant barriers shall be permitted for use above decks where the radiant barrier is covered with an *approved* roof covering and the system consisting of the radiant barrier and the roof covering complies with the requirements of either FM 4550 or UL 1256.</u>

~~**1509.3 Material standards,** Above-deck radiant barrier shall comply with ASTM C1313/1313M.~~

<u>**1509.3 Installation.** The low emittance surface of the radiant barrier shall face the continuous air space between the radiant barrier and the roof covering.</u>

<u>**1509.4 Material standards.** A radiant barrier installed above a deck shall comply with ASTM C1313/C1313M.</u>

Add new standard to Chapter 35 as follows:

ASTM

C1313/C1313M-~~10~~ <u>12</u> Standard Specification for Sheet Radiant Barriers for Building Construction Applications

Commenter's Reason: A key issue that needs to be addressed in the new proposed section 1509, and that was unclear in the original proposal, was how the fire testing of the system is to be done. The comment clarifies that the testing must be done using the combination of the radiant barrier <u>**and**</u> the approved roof covering and that the system needs to pass the fire test.

The new text is necessary because there are differences between a reflective insulation and a radiant barrier, even if there are many similarities and the fire testing is similar. For example, one difference is that a radiant barrier often does not provide thermal insulation. ASTM has issued separate specifications for radiant barriers used in buildings (ASTM C1313, Standard Specification for

Sheet Radiant Barriers for Building Construction Applications) and for reflective insulations used in buildings (ASTM C1224, Standard Specification for Reflective Insulation for Building Applications).

The original proposal also contained a definition that was incorrect in that it did not just explain what a radiant barrier is but it also told users how to install products, which it should not do.

The public comment also includes the reference standard specification and includes the updated edition, without the non-mandatory language identified by the ICC standards committee. The abstract of the ASTM C1313 specification reads as follows. "This specification covers the general physical property requirements of radiant barrier materials for use in building construction. The scope is specifically limited to requirements for radiant barrier sheet materials that consist of at least one surface, such as metallic foils or metallic deposits mounted or unmounted on substrates. Sheet radiant barrier materials shall consist of low emittance surface(s) that may be in combination with any substrates and adhesives required to meet the specified physical material properties. The following test methods shall be performed: surface emittance; water vapor transmission; surface burning characteristics; corrosivity; tear resistance; and adhesive performance."

There is a companion proposal, FS199, dealing with a radiant barrier section in Chapter 26, and it proposes the same definition as this one. The proposals can be handled independently and are not a function of each other.

Public Comment 2:

Vickie Lovell, InterCode Incorporated, representing Reflective Insulation Manufacturers Association International, requests Approval as Modified by this Public Comment.

Replace proposal as follows:

SECTION 202
DEFINITIONS

RADIANT BARRIER. A material having a low emittance surface of 0.1 or less installed in building assemblies.

SECTION 1509 RADIANT BARRIERS INSTALLED ABOVE DECK

1509.1 General. A Radiant barrier installed above a deck shall comply with Sections 1509.2 through 1509.4.

1509.2 Fire Testing. Radiant barriers shall be permitted for use above decks where the radiant barrier is covered with an *approved* roof covering and the system consisting of the radiant barrier and the roof covering complies with the requirements of either FM 4550 or UL 1256.

1509.3 Installation. The low emittance surface of the radiant barrier shall face the continuous air space between the radiant barrier and the roof covering.

1509.4 Material standards. A Radiant barrier installed above a deck shall comply with ASTM C1313/1313M.

Add new standard to Chapter 35 as follows:

ASTM

C1313/C1313M-12 Standard Specification for Sheet Radiant Barriers for Building Construction Applications

Reason: Both the original proposal and this public comment intend to codify the correct fire testing requirements, proper installation, and the appropriate ASTM material standard for a radiant barrier installed above a roof deck. The proposed definition is derived from the definition for radiant barrier in ASTM C1313.

The new section as proposed in this public comment is necessary. Although, there are many inherent similarities including similar fire testing, there are significant differences between reflective insulation and radiant barriers that warrant this additional language to the code. A key issue that was addressed in the new proposed section 1509, and that was unclear in the original proposal, was how the fire testing of the system was to be done. This public comment clarifies that the testing must be done using the combination of the radiant barrier **and** the approved roof covering and that the system needs to pass the fire test.

There is a common assumption that some radiant barrier products installed above-deck, typically between the deck and the felt, provide some level of thermal benefit. This is not the case. ASTM has issued separate specifications for radiant barriers used in buildings (ASTM C1313, Standard Specification for Sheet Radiant Barriers for Building Construction Applications) and for reflective insulations used in buildings (ASTM C1224, Standard Specification for Reflective Insulation for Building Applications).

The abstract of the ASTM C1313 specification for radiant barriers reads as follows. "This specification covers the general physical property requirements of radiant barrier materials for use in building construction. The scope is specifically limited to requirements for radiant barrier sheet materials that consist of at least one surface, such as metallic foils or metallic deposits mounted or unmounted on substrates. Sheet radiant barrier materials shall consist of low emittance surface(s) that may be in combination with any substrates and adhesives required to meet the specified physical material properties. The following test methods shall be performed: surface emittance; water vapor transmission; surface burning characteristics; corrosivity; tear resistance; and adhesive performance."

At the time the originals proposals were due, the most recent edition of ASTM C1313 was not yet published. It is available now, and a live link to the read-only file has been provided by ASTM. The link is www.astm.org/

C1313. The 2012 revisions to the 2010 edition were to remove the permissive language with no other significant technical changes.

References:
ASTM C1313/C1313M "Standard Specification for Sheet Radiant Barriers for Building Construction Applications"

S51-12
Final Action: AS AM AMPC____ D

S53-12
1509.7.1

Proposed Change as Submitted

Proponent: Christine Covington, Solar Energy Industries Association

Revise as follows:

1509.7.1 ~~Wind resistance~~ Structural loads. Rooftop mounted photovoltaic systems shall be ~~designed for wind loads for component and cladding~~ capable of resisting applicable structural loads in accordance with Chapter 16 ~~using an effective wind area based on the dimensions of a single unit frame~~.

Reason: Rooftop PV systems may be subjected to structural loads other than wind. Seismic and snow loads may also be applicable and should be evaluated as part of the design.

IBC Chapter 16 addresses design loads with reference to ASCE 7. Chapter 16 and ASCE 7 include requirements for combinations of loads. Wind requirements are the subject of Chapters 26-31 of ASCE 7-10, which include multiple methods of determining wind loads. Components and cladding methods are appropriate for some rooftop PV systems, but not all. For example, some tall rooftop systems experience wind behavior appropriate to the Main Wind Force Resisting System, and some systems held close to the roof surface have been studied using Wind Tunnel testing. These approved wind load evaluation methods appear to be prohibited by the current language without justification.

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

1509.7.1-S-COVINGTON

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee felt the current wording is necessary, while the proposed revision would remove the specific reference to wind load requirements.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Joseph H. Cain, P.E. SolarCity Corporation, representing self, and John Smirnow, representing Solar Energy Industries Association (SEIA), request Approval as Modified by this Public Comment.

Modify the proposal as follows:

1509.7.1 Structural loads. Rooftop mounted photovoltaic panel systems shall be ~~capable of resisting applicable structural loads~~ designed for wind loads for components and cladding in accordance with Chapter 16 using an effective wind area in accordance with Chapter 16 and ASCE 7 Section 26.2.

Commenter's Reason: The term "photovoltaic panel systems" is used, consistent with the new definition approved in S5-12.

The term "components and cladding" is used, consistent with usage in ASCE 7.

The proposed public comment modification to Section 1509.7.1 is intended to correct a significant error in the 2012 IBC. The requirement "using an effective wind area based on the dimensions of a single unit frame" is in conflict with the definition of Effective Wind Area in ASCE 7-10.

Effective Wind Area (EWA) is defined in ASCE 7-10 Section 26.2:

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

The Structural Engineers Association of California (SEAOC) Solar Photovoltaic Systems Committee recently published a white paper titled *Wind Loads on Low-Profile Solar Photovoltaic Systems on Flat Roofs.* This paper will be presented at the SEAOC Annual Convention on September 14, 2012. The Solar Photovoltaic Systems Committee carefully considered Effective Wind Area for solar photovoltaic systems, with specific consideration of 2012 IBC Section 1509.7.1. The Committee chose to publish extensive commentary on Effective Wind Area in Section 5 of the white paper. Full text of Section 5 is provided below. It is important to note the final paragraph of the Commentary.

"The requirements and commentary above differ from the provision of IBC 2012 (14) Section 1509.7.1 that states, 'Rooftop mounted photovoltaic systems shall be designed … using an effective wind area based on the dimensions of a single unit frame.' It is the consensus opinion of the SEAOC Solar Photovoltaic Systems Committee that this provision is not appropriate for many types of systems and parts of solar arrays. The provision can be un-conservative for a fastener with tributary area less than a 'single unit frame' and is overly conservative for elements of a solar array, such as main supports or members that have a tributary area of several solar modules. The provision may also be overly conservative if applied to a framing member of a building supporting multiple attachments from a solar array."

5. Effective Wind Area

The following is proposed code language to amend ASCE 7-10 Section 26.2 (ASCE 7-05 Section 6.2) by adding the definition of effective wind area for roof mounted solar arrays.

EFFECTIVE WIND AREA, A for solar arrays: The area used to determine GC_m per Figure 29.9-1 is equal to the tributary area for the structural element being considered, except that the width of the effective wind area need not be less than one-third its length. For a fastener attaching solar modules, the effective wind area shall not be greater than the area tributary to the individual fastener.

The SEAOC Solar Photovoltaic Systems Committee chose to include the following commentary. In the last paragraph of the commentary, the Committee specifically mentioned consensus opinion that differs from 2012 IBC Section 1509.7.1.

Commentary:

The definition of effective wind area for solar arrays is similar to that for components and cladding. As with components and cladding, the width of the effective wind area need not be less than one-third its length (which is typically equal to the span of the framing element being considered). The induced wind pressure is calculated per Figure 29.9-1 using this effective wind area, and the wind pressure is then applied over the actual area tributary to the element.

Effective wind area is equal to tributary area except in cases where the exception is invoked that the width of the effective wind area need not be less than one-third its length. In such cases the effective wind area will be larger than the tributary area.

The use of effective wind area in wind design is based on the phenomenon that the highest wind pressures come from instantaneous gust effects that are concentrated on small areas. Larger areas have lower design pressure because wind pressures over the entire area do not peak at the same time (13). The concentrated pressures from gusts tend to be circular or elliptical in shape and are very unlikely to occur in an elongated shape directly over the span of a long framing member. Thus if the tributary area of a member is more elongated than a 3:1 ratio of length to width, the effective wind area can be increased to that corresponding to a width equal to 1/3 the length of the effective wind area. Further discussion is provided in Section 9.2.3 of (13).

Tributary area for a spanning structural member of a solar array depends on the span length of that member times the perpendicular distances to adjacent parallel members. For a support point or fastener, tributary area depends on the span of members framing into that support point.

Tributary area (and effective wind area) can depend on the characteristics of the solar array support system and the load path. For a roof bearing system having different load paths for upward, downward, and lateral forces, the appropriate effective wind area for each direction of forces is used.

If the support system for the solar array has adequate strength and interconnectedness to span across a support or ballast point that is subject to yielding or uplift, the tributary area (and effective wind area) can be correspondingly increased, provided that strengths are not governed by brittle failure and that the deformation of the array is evaluated and does not result in adverse performance.

The requirements and commentary above differ from the provision of IBC 2012 (14) Section 1509.7.1 that states, "Rooftop mounted photovoltaic systems shall be designed … using an effective wind area based on the dimensions of a single unit frame." It is the consensus opinion of the SEAOC Solar Photovoltaic Systems Committee that this provision is not appropriate for many types of systems and parts of solar arrays. The provision can be un-conservative for a fastener with tributary area less than a "single unit frame" and is overly conservative for elements of a solar array, such as main supports or members that have a tributary area of several solar modules. The provision may also be overly conservative if applied to a framing member of a building supporting multiple attachments from a solar array.

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

S53-12
Final Action: AS AM AMPC____ D

S60-12
1510.1

Proposed Change as Submitted

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

Revise as follows:

1510.1 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. Reroofing shall not be required to meet the minimum design slope requirement of one-quarter unit vertical in 12 units horizontal (2-percent slope) in Section 1507 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.

Reason: IBC 2006 and subsequent editions include a requirement in Section 1503.4-Roof Drainage that for roof drainage systems with roof drains or scuppers, secondary (emergency overflow) drains or scuppers also be provided in the event the primary roof drainage system becomes clogged.

Section 1510-Reroofing requires all materials and methods used in recovering or replacing an existing roof covering comply with the requirements of Chapter 15 (except the minimum roof slope requirement of ¼:12 can be waived for roofs that provide "…positive roof drainage."). This can be interpreted to require the secondary (emergency overflow) drains and scupper provision also apply in reroofing. Since many existing buildings were designed and constructed before the code included a secondary drainage requirement, the secondary drainage provision being applicable in reroofing and the need for adding secondary drains in exsiting buildings during reroofing can be a very costly and disruptive undertaking for owners and occupants.

This proposed code change adds an exception in Section 1510-Reroofing that waives the secondary drainage provision when reroofing existing buildings when the roof drains properly, that being hat provide for positive roof drainage. The term "positive roof drainage' is already defined in Section 202.

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

1510.1-S-GRAHAM

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

1510.1 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. Reroofing shall not be required to meet the minimum design slope requirement of one-quarter unit vertical in 12 units horizontal (2-percent slope) in Section 1507 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 and are not required to have secondary drains or scuppers.

Committee Reason: This code change adds an exception that recognizes in existing buildings without these drains, they would be difficult to add when reroofing. The modification addresses an unintended consequence of roofs with secondary drainage using the exception to eliminate the required drains.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment 1:

Cole Graveen, Raths, Raths & Johnson, Inc., representing self, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1510.1 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. Reroofing shall not be required to meet the minimum design slope requirement of one-quarter unit vertical in 12 units horizontal (2-percent slope) in Section 1507 for roofs that provide positive roof drainage.
2. For roofs that provide positive roof drainage, ~~R~~recovering or replacing an existing roof covering shall not ~~be required to meet the requirement for~~ require the secondary (emergency overflow) drains or scuppers ~~in~~ of Section 1503.4 to be added to the existing roof. ~~for roofs that provide positive roof drainage and are not required to have secondary drains or scuppers.~~

Commenter's Reason: The wording of the proposed change, as modified by the Committee is not clear. The wording proposed in this public comment is more concise and better reflects the intent of both the original change and the committee's modification. The intent of this public comment is not to change the meaning of either the original change or the committee's modification, but only to make the wording more clear.

Public Comment 2:

Jonathan Siu representing City of Seattle Dept of Planning & Development, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1510.1 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. Reroofing shall not be required to meet the minimum design slope requirement of one-quarter unit vertical in 12 units horizontal (2-percent slope) in Section 1507 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~~ and are not required to have secondary drains or scuppers~~. 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.

Commenter's Reason: The intent of the original proposal was to provide an exception to make sure secondary roof drains would not required to be installed if the only extent of the work was to re-cover or replace the existing roof covering. The Report of Hearings states the Structural Committee's reason for modifying the proposal was that it didn't want to give the false impression that existing secondary roof drains could be removed. However, as actually modified by the Committee, this exception is only allowed to apply where the secondary drains are not required. This modification essentially makes the exception useless—very few building owners would install secondary drainage if it is not required, and if the secondary drainage is required, the exception no longer applies, so the owner has to install the secondary drains. This goes against the whole intent of the original proposal.

The proposed modification in this public comment is intended to preserve the original intent of the proposal, but clarifies this exception cannot be used to remove a required, existing secondary drainage system, unless it is replaced by a code-compliant system.

S60-12

Final Action: AS AM AMPC____ D

S61-12
1510.2

Proposed Change as Submitted

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

Revise as follows:

1510.2 Structural and construction loads. Structural roof components shall be capable of supporting the roof-covering system and the material and equipment loads that will be encountered during installation of the system. <u>Existing structural assemblies shall comply with the requirements of Section 3404.</u>

Reason: Chapter 34 provides good guidance to the designer regarding the types of conditions that should be evaluated during alterations. This proposal provides a necessary reference for the purposes of linking those requirements.

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

1510.2-S-FISCHER

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee believes the existing wording is clear. The proposed reference to Section 3404 is not specific and would be confusing.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Michael D. Fischer, Kellen Company, representing Asphalt Roofing Manufacturers Association, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1510.1 General. Materials and methods of application used for recovering or replacing an existing roof covering shall comply with the requirements of Chapter 15 <u>and Section 3404</u>.

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

1510.2 Structural and construction loads. Structural roof components shall be capable of supporting the roof-covering system and the material and equipment loads that will be encountered during installation of the system. ~~Existing structural assemblies shall comply with the requirements of Section 3404.~~

Commenter's Reason: This proposal was submitted as part of a package. The proponent requested disapproval of the proposal due to concerns with other technical issues. The intent of this proposal is to make it clear that roof recovering or replacement shall meet the applicable requirements for alterations in Section 3404.

S61-12

Final Action: AS AM AMPC____ D

S62-12
1510.3 (New), 1510.4

Proposed Change as Submitted

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

Revise as follows:

1510.3 Roof replacement. Roof replacement shall include the removal of all existing layers of roof coverings down to the roof deck.

Exceptions:

1. Complete and separate roofing systems, such as standing-seam metal roof systems, that are designed to transmit the roof loads directly to the building's structural system and that do not rely on existing roofs and roof coverings for support, shall not require the removal of existing roof coverings.
2. Metal panel, metal shingle and concrete and clay tile roof coverings shall be permitted to be installed over existing wood shake roofs where applied in accordance with Section 1510.4.
3. The application of a new protective coating over an existing spray polyurethane foam roofing system shall be permitted without tear-off of existing roof coverings.
4. Where the existing roof assembly includes an ice barrier membrane that is adhered to the roof deck, the existing ice barrier membrane shall be permitted to remain in place and covered with an additional layer of ice barrier membrane in accordance with Section 1507.
5. 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.
6. Where the existing roof covering is wood shake, slate, clay, cement or asbestos-cement tile.
7. Where the existing roof has two or more applications of any type of roof covering.

~~**1510.3**~~ **1510.4** ~~**Recovering versus replacement**~~ **Roof recovering.** ~~New roof coverings shall not be installed without first removing all existing layers of roof coverings down to the roof deck~~ Roof recovering shall be prohibited where any of the following conditions occur:

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

Exceptions:

1. Complete and separate roofing systems, such as standing-seam metal roof systems, that are designed to transmit the roof loads directly to the building's structural system and that do not rely on existing roofs and roof coverings for support, shall not require the removal of existing roof coverings.
2. Metal panel, metal shingle and concrete and clay tile roof coverings shall be permitted to be installed over existing wood shake roofs when applied in accordance with Section 1510.4.
3. The application of a new protective coating over an existing spray polyurethane foam roofing system shall be permitted without tear-off of existing roof coverings.
4. Where the existing roof assembly includes an ice barrier membrane that is adhered to the roof deck, the existing ice barrier membrane shall be permitted to remain in place and

covered with an additional layer of ice barrier membrane in accordance with Section 1507.

Reason: The current text is confusing and contains directions on what NOT to do regarding roof recovering. The proposal reorganizes the text without making any technical changes in order to add clarity to the code. The revisions provide clear distinction between roof replacement and roof recovering

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

1510.3 (NEW)-S-FISCHER

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The proposed text is not clear and contains errors. The proponent requested disapproval, recognizing there was too much to fix with a floor modification.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Michael D. Fischer, Kellen company, representing Asphalt Roofing Manufacturers Association, requests Approval as Modified by this Public Comment.

Replace the proposal as follows:

~~**1510.3 Recovering versus replacement.** New roof coverings shall not be installed without first removing all existing layers of roof coverings down to the roof deck where any of the following conditions occur:~~

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

~~**Exceptions:**~~

1. ~~Complete and separate roofing systems, such as standing-seam metal roof systems, that are designed to transmit the roof loads directly to the building's structural system and that do not rely on existing roofs and roof coverings for support, shall not require the removal of existing roof coverings.~~
2. ~~Metal panel, metal shingle and concrete and clay tile roof coverings shall be permitted to be installed over existing wood shake roofs when applied in accordance with Section 1510.4.~~
3. ~~The application of a new protective coating over an existing spray polyurethane foam roofing system shall be permitted without tear-off of existing roof coverings.~~
4. ~~Where the existing roof assembly includes an ice barrier membrane that is adhered to the roof deck, the existing ice barrier membrane shall be permitted to remain in place and covered with an additional layer of ice barrier membrane in accordance with Section 1507.~~

1510.3 Roof replacement. Roof replacement shall include the removal of all existing layers of roof coverings down to the roof deck.

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

1510.3.1 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 manufacturers approved installation instructions.

2. Complete and separate roofing systems, such as standing-seam metal roof systems, that are designed to transmit the roof loads directly to the building's structural system and 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 1510.4.
4. The application of a new protective coating over an existing spray polyurethane foam roofing system shall be permitted without tear-off of existing roof coverings.

1510.3.1.1 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.

Commenter's Reason: This proposal was submitted as part of a package. The proponent requested disapproval of the proposal due to a scoping error and other technical issues. The intent of this proposal is to clarify the requirements for roof recover and roof replacement. In the new Section 1510.3, the requirements for roof replacement (and the exception for ice barrier membranes) remain intact. The new Section 1510.3.1 provides a much clearer path to identify those conditions where recover is permitted by the code. The current provisions for roof recover remain intact, except for two technical changes:
1. The current code contains a conflict related to the covering of wood shakes. The public comment provides a remedy by eliminating the prohibition contained in the source text for the new 1510.3.1.1, which is in conflict the application in accordance with Section 1510.4.
2. The code lists several prescriptive options for recover, but does not specifically provide for other conditions where products have been evaluated for recover applications. The modified proposal includes that option, but requires installation in accordance with the manufacturer's instructions.

S62-12
Final Action: AS AM AMPC____ D

S64-12
1510.7 (New)

Proposed Change as Submitted

Proponent: Al Godwin, CBO, CPM, representing Aon Fire Protection Engineering **(**al.godwin@aon.com)

Add new text as follows:

1510.7 Construction of sloped roof over flat roof. Construction of a new roof over an existing roof, in a manner that creates an attic or concealed space shall require the removal of any existing roofing material composed of tar, asphalt or roof insulation not designed for interior use from the newly created interior space.

Reason: It is not uncommon for building owners to convert a flat roof to a sloped roof. When doing so, the former roofing material should be removed from the newly created interior space.

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

1510.7 (NEW)-S-GODWIN

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: This code change addresses new roof construction under the reroofing provisions and it is poorly structured.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Al Godwin, CBO, CPM, representing Aon Fire Protection Engineering Corporation, requests Approval as Modified by this Public Comment.

Replace the proposal as follows:

1505.9 Enclosure of an existing roof. Construction of a new roof structure over an existing roof, in a manner that creates an attic or concealed space, shall require the removal of any formerly exposed roofing material composed of tar, asphalt or above roof deck insulation. This provision shall not apply to reroofing in accordance with Section 1510.

Commenter's Reason: Based on the Committee's recommendation, this modification moves the provision to Section 1505.9. It was difficult to find an appropriate section. Since photovoltaic systems appear under Section 1505, it seemed a good place for this provision. The provision has been reworded to not include minor air spaces that might occur under the reroofing process but to only include new structural construction.

S64-12

Final Action: AS AM AMPC____ D

S65-12
1511.1.1

Proposed Change as Submitted

THIS CODE CHANGE WILL BE HEARD BY THE IBC GENERAL COMMITTEE. SEE THE TENTATIVE HEARING ORDER FOR THIS COMMITTEE.

Proponent: Maureen Traxler, City of Seattle Department of Planning & Development (Maureen.traxler@seattle.gov); Thomas Meyers, City of Central, CO, representing self

Delete without substitution:

~~**1511.1.1 Structural fire resistance.** The structural frame and roof construction supporting the load imposed upon the roof by the photovoltaic panels/modules shall comply with the requirements of Table 601~~.

Reason:
(Traxler) This section is not needed because Table 601 will apply regardless of this section. In addition, the terminology used is not consistent with the terms used in Table 601, creating confusion about whether the “structural frame…supporting the load imposed upon the roof” is different than the primary structural frame and secondary members referenced in Table 601. If they are different, then Table 601 doesn’t have any applicable requirements. If they are the same, the section isn’t necessary because compliance with Table 601 is already required by Chapter 6.
(Meyers) This new section was added as part of a comprehensive code change submitted to the IFC and ultimately approved as modified by public comment at the Dallas Final Action Hearings. The new subsection 1511.1.1 has generated considerable confusion. It has been interpreted to require any of the stand-off rack frame used to mount solar panels to the roof to be fire resistance rated consistent with the Type of Construction used by the building. In the case of I-A construction, this interpretation would require the typical aluminum square tube “column” supports to exhibit 3 hour fire endurance. This is extremely excessive and very difficult to achieve in an exposed, exterior application.

It appears that the intent may have been to ensure that the underlying supporting roof structure be provided with the fire performance prescribed by Chapter 6 when supporting any loads imposed by the solar panel array system that includes the racking system. The code already ensures that in Chapter 6. Therefore, this section is completely redundant. As such, it should be eliminated to avoid confusion.

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

1511.1.1-S-TRAXLER.doc

Public Hearing Results

This code change was heard by the IBC General code development committee.

Committee Action: **Disapproved**

Committee Reason: The committee did agree with the intent that the photovoltaics were not considered part of the structure but there was concern with the deletion of the section in its entirety. Without this section the potential loading on the roof would not be properly addressed.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment 1:

Thomas Meyers and Stephen Thomas, Colorado Code Consulting, LLC representing the Colorado Chapter of ICC, requests Approval as Submitted.

Commenter's Reason: During the public hearing, come opponents indicated that they believed this section's intent is to direct the user to Table 601, Footnote A. Regardless of the interpretation of Table 601 Footnote A, the language used in this section is very confusing. As currently stated, it implies that the typical aluminum structural framework of a rack-mount PV system would have to be 1, 2, or 3 hour fire resistance rated depending upon the building's construction. This would be onerous, if not completely infeasible.

The committee indicated that it was not their intent to fire protect these elements. They were only interested in providing a reference to T601. T601 Footnote A exists without the cross reference provided at 1511. Deletion of this section by approving this public comment would still permit the enforcement of T601 without the unintentional added confusion created by the existing language.

Public Comment 2:

Thomas Meyers and Stephen Thomas, Colorado Code Consulting, LLC representing the Colorado Chapter of ICC, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

~~1511.1.1~~ 1509.9 Structural fire resistance. The structural frame and roof construction supporting ~~the load~~ imposed loads upon the roof by ~~the photovoltaic panels/modules~~ any rooftop structure shall comply with the requirements of Table 601. The fire resistance reduction permitted by Table 601, Footnote a shall not apply to roofs containing rooftop structures.

Reason: During the public hearing, some opponents indicated that they believed this section's intent is to direct the user to Table 601, Footnote A. Should this cross reference be necessary, this proposal would apply the requirement to ALL rooftop structures. This eliminates the current discriminatory condition where only solar PV is addressed. This clarity modification is provided as an alternative to our other public comment that would approved as submitted the original proposal to delete this confusing language altogether.

Public Comment 3:

Steven Pfeiffer, City of Seattle, representing Department of Planning & Development, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1511.1.1 Structural fire resistance. The structural frame and roof construction supporting the load imposed upon the roof by the photovoltaic panels/modules shall comply with the requirements of Table 601.

> **Exception:** The portions of the structure above the roof supporting only the panels/modules need not comply with the requirements of Table 601.

Commenter's Reason: The committee agreed that the photovoltaic panels and modules were not considered part of the structure but there was concern with the deletion of the section in its entirety. The roof, as regulated by Table 601, protects the building from any hazard presented by the photovoltaic equipment. The photovoltaic panels and their supports are not the roof structure.

S65-12

Final Action: AS AM AMPC____ D

S69-12
1603.1.3

Proposed Change as Submitted

Proponent: Edwin Huston, National Council of Structural Engineers Association (NCSEA), representing NCSEA Code Advisory Subcommittee – General Requirements Subcommittee (huston@smithustoninc.com)

Revise as follows:

1603.1.3 Roof snow load data. The ground snow load, P_g, shall be indicated. In areas where the ground snow load, P_g, exceeds 10 pounds per square foot (psf) (0.479 kN/m2), the following additional information shall also be provided, regardless of whether snow loads govern the design of the roof:

1. Flat-roof snow load, P_f.
2. Snow exposure factor, C_e.
3. Snow load importance factor, *I*.
4. Thermal factor, C_t.
5. Drift surcharge load, p_d, where the sum of p_d and P_f exceeds 20 pounds per square foot (psf).
6. Width of snow drift, *w.*

Reason: The addition of loading information and design assumptions to drawings has been valuable to owners and the engineers who are tasked with re-evaluating existing structures. This additional requirement of snow drift design information supplements the information already required and indicates how the registered design professional interpreted the design codes relative to snow drift intensity and width.

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

1603.1.3-S-HUSTON

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

1603.1.3 Roof snow load data. The ground snow load, P_g, shall be indicated. In areas where the ground snow load, P_g, exceeds 10 pounds per square foot (psf) (0.479 kN/m2), the following additional information shall also be provided, regardless of whether snow loads govern the design of the roof:

1. Flat-roof snow load, P_f.
2. Snow exposure factor, C_e.
3. Snow load importance factor, *I*.
4. Thermal factor, C_t.
5. Drift surcharge load(s), p_d, where the sum of p_d and P_f exceeds 20 pounds per square foot (psf).
6. Width of snow drift(s), *w.*

Committee Reason: The committee agreed that the drift load and the width of snow drift are important to have on the plans. The increased transparency it affords makes it easier on the plans examiner. It also is beneficial for alterations to existing buildings. The modification is a clarification that recognizes there can be multiple drifts in some cases.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Daniel J. Walker, P.E., Thomas Associates, Inc., representing Metal Building Manufacturers Association, requests Disapproval.

Commenter's Reason: The concept of placing key design load criteria on construction documents has merit, but this proposal that would include snow drift information is excessive due to the complexity of conveying this information for a roof with reentrant corners, multiple steps, parapets, rooftop equipment, etc. All of the other design data that the code requires to be included on the construction documents is a single value or list of values. Snow drift surcharge and the width of snow drifts could involve information that would have to be conveyed with many diagrams that would be more appropriate for engineering calculations than construction documents. It would probably lead to more questions and confusion than a source of valuable information.

S69-12
Final Action: AS AM AMPC____ D

S71-12
1603.1.7

Proposed Change as Submitted

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov) (gregory.p.wilson@dhs.gov), Rebecca C. Quinn, RCQuinn Consulting, Inc., representing Department of Homeland Security, Federal Emergency Management Agency (rcquinn@earthlink.net)

Revise as follows:

1603.1.7 Flood design data. For buildings located in whole or in part in *flood hazard areas* as established in Section 1612.3, the documentation pertaining to design, if required in Section 1612.5, shall be included and the following information, referenced to the datum on the community's Flood Insurance Rate Map (FIRM), shall be shown, regardless of whether flood loads govern the design of the building:

1. *Risk Category* assigned according to ASCE 24.
~~1.~~ 2. In *flood hazard areas* not subject to high-velocity wave action, the elevation of the proposed lowest floor, including the basement.
~~2.~~3. In *flood hazard areas* not subject to high-velocity wave action, the elevation to which any nonresidential building will be dry flood proofed.
~~3.~~ 4. In *flood hazard areas* subject to high-velocity wave action, the proposed elevation of the bottom of the lowest horizontal structural member of the lowest floor, including the basement.

Reason: The current edition of ASCE 24 uses the assigned occupancy/structure category primarily to determine elevation of buildings above the design flood elevation, in keeping with the general approach that more important buildings be designed for less frequent environmental loads. The next edition of ASCE 24 will include the Risk Category table from ASCE 7-10. The ASCE committee recognized that ASCE 7-10 eliminated the lists of buildings for each category and determined it important to ensure that the assignment of risk category be guided by definitions that are specifically developed to ensure that buildings in flood hazard areas are appropriately protected. Therefore, the next edition of ASCE 24 requires the user to reevaluate and possibly reassign a risk category specifically for the purpose of flood loads and flood resistant construction requirements.

ASCE began the process of updating ASCE 24-05 in early 2011 and the next edition is expected to be published late 2012 or early 2013. The ASCE committee expects to have the near-final draft prepared and available at least a month before the Group A hearings and copies will be provided to the ICC committee.

Cost Impact: The code change proposal will not increase the cost of construction. The definitions of each risk category that will be in the revised ASCE 24 and used only for the purpose of assigning risk category for flood-resistant design essentially retain the descriptions from the 2012 IBC Table 1604.5 of which buildings fall into each of the risk categories.

Analysis: Will the proposal introduce a conflict with Section 1604.5?

1603.1.7-S-INGARGIOLA-WILSON-QUINN.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: There was concern with having to consult an additional table in a standard for a risk category for flood purposes. Consideration should be given to identifying it as a flood risk category.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

John Ingargiola, Gregory Wilson representing Department of Homeland Security, Federal Emergency Management Agency, Rebecca Quinn, RCQuinn Consulting, Inc, representing Department of Homeland Security, Federal Emergency Management Agency, request Approval as Modified by this Public Comment.

Modify the proposal as follows:

1603.1.7 Flood design data. For buildings located in whole or in part in *flood hazard areas* as established in Section 1612.3, the documentation pertaining to design, if required in Section 1612.5, shall be included and the following information, referenced to the datum on the community's Flood Insurance Rate Map (FIRM), shall be shown, regardless of whether flood loads govern the design of the building:

1. ~~*Risk Category*~~ Flood design class assigned according to ASCE 24*.*
2. In *flood hazard areas* not subject to high-velocity wave action, the elevation of the proposed lowest floor, including the basement.
3. In *flood hazard areas* not subject to high-velocity wave action, the elevation to which any nonresidential building will be dry flood proofed.
4. In *flood hazard areas* subject to high-velocity wave action, the proposed elevation of the bottom of the lowest horizontal structural member of the lowest floor, including the basement.

Commenter's Reason: The near-final draft of ASCE 24 based on the third ballot no longer uses the structure/risk category designation. Instead, ASCE 24-12 will require each building and structure to be assigned to a "Flood Design Class", which is then used throughout the standard to specify elevation requirements and floodproofing limitations.

S71-12

Final Action: AS AM AMPC____ D

S72-12

1603.1.8.1 (New), 1607.12.5 (New), 1607.12.5.1 (New), 1607.12.5.2 (New), 1607.12.5.3 (New), 1607.12.5.4 (New)

Proposed Change as Submitted

Proponent: Edwin Huston, National Council of Structural Engineers Associations (NCSEA), representing NCSEA Code Advisory Subcommittee – General Requirements Subcommittee (huston@smithhustoninc.com)

Add new text as follows:

1603.1.8.1 Solar Photovoltaic (PV) Panels/Modules. The Roof/PV live load used in the design of Solar PV Panels shall be indicated on the construction documents.

1607.12.5 Solar Photovoltaic (PV) panels/modules. Solar PV panels/modules shall be designed in accordance with Sections 1607.12.5.1 through 1607.12.5.4, as applicable.

1607.12.5.1 Roof/PV live load. The roof/PV live load is a 20 psf uniform load. Unless each Solar PV panel/module is clearly and permanently marked "Do not walk on this surface – not intended for maintenance access or pedestrian traffic", and appropriate maintenance access paths are provided a non-concurrent 300 pound concentrated load as set forth in Table 1607.1 shall also be applied. The individual Solar PV panels/modules shall be designed to withstand the Roof/PV live load, in combination with other applicable loads.

1607.12.5.2 PV panels/modules. Solar PV panels/modules designed to be installed over and supported by a roof, shall have the structural supports of the roof designed to accommodate the full dead load, including the Solar PV panels/modules dead load; the Roof/PV live load in the areas of the Solar PV panels/modules in combination with other applicable loads. The roof area underneath any Solar PV panels/modules shall also be designed for load combinations including roof live load, in combination with other applicable loads, without the Solar PV panels/modules.

1607.12.5.3 PV panels/modules installed as an independent structure. Solar PV panels/modules that are independent structures and do not have accessible /occupied space underneath are not required to accommodate a roof/PV live load, provided they are marked as required in Section 1607.12.5.1, and the area under the structure is restricted to keep the public away. All other loads and combinations per Section 1605 shall be accommodated.

Solar PV panels/modules that are designed to be the roof, and span to structural supports, and have accessible/occupied space underneath shall have the panels/modules and all supporting structure designed to support a Roof/PV live load, as defined in section 1607.12.5.1 in combination with other applicable loads. Solar PV panels/modules in this application are not permitted to be classified as "not accessible" per 1607.12.5.1.

1607.12.5.4 Ballasted systems. Solar PV panels/modules installed on a roof as a ballasted system need not be rigidly attached to the roof or supporting structure. Ballasted systems shall be designed and installed only on roofs with slopes of ½" per foot or less. The structural supports of the roof under a ballasted system shall be designed, or analyzed, per section 1604.4; checked in accordance with Section 1604.3.6 for deflections; and checked in accordance with Section 1611 for ponding. The ballasted system shall be designed to resist sliding and uplift resulting from lateral and vertical forces as required by Section 1605, using a coefficient of friction determined by acceptable engineering principles.

Reason: This new section is bringing in requirements for Solar PV panels that is currently absent in the code.

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

1603.1.8.1 (NEW)-S-HUSTON

Public Hearing Results

Committee Action: Approved as Modified

Modify proposal as follows:

1603.1.8.1 Solar photovoltaic (PV) panels~~/~~ or modules. The ~~Roof/PV live~~ dead load ~~used in the design~~ of ~~Solar PV Panels~~ solar PV panels or modules, including accessories, shall be indicated on the construction documents.

1607.12.5 Solar photovoltaic (PV) panels~~/~~ or modules. Solar PV panels~~/~~ or modules shall be designed in accordance with Sections 1607.12.5.1 through 1607.12.5.4, as applicable.

1607.12.5.1 Roof/PV live load. ~~The roof/PV live load is a 20 psf uniform load. Unless each Solar PV panel/module is clearly and permanently marked "Do not walk on this surface – not intended for maintenance access or pedestrian traffic", and appropriate maintenance access paths are provided a non-concurrent 300 pound concentrated load as set forth in Table 1607.1 shall also be applied. The individual Solar PV panels/modules shall be designed to withstand the Roof/PV live load, in combination with other applicable loads.~~ Roof surfaces to be covered by solar PV panels or modules shall be designed for the roof live load, L_r, assuming that the PV panels or module are not present. The roof/PV live load in areas covered by solar PV panels or modules shall be in addition to the panel loading unless the area covered by each solar PV panel or module is inaccessible. Areas where the clear space between the panels and the rooftop is 24 inches or less shall be considered inaccessible. Roof surfaces not covered by PV panels shall be designed for the roof live load.

1607.12.5.2 PV panels~~/~~ or modules. ~~Solar PV panels/modules designed to be installed over and supported by a roof, shall have the structural supports of the roof designed to accommodate the full dead load, including the Solar PV panels/modules dead load; the Roof/PV live load in the areas of the Solar PV panels/modules in combination with other applicable loads. The roof area underneath any Solar PV panels/modules shall also be designed for load combinations including roof live load, in combination with other applicable loads, without the Solar PV panels/modules.~~ The structure of a roof that supports solar PV panels or modules shall be designed to accommodate the full solar PV panels or modules and ballast dead load, including concentrated loads from support frames in combination with the loads from Section 1607.12.5.1 and other applicable loads. Where applicable, snow drift loads created by the PV panels or modules shall be included.

1607.12.5.3 PV panels~~/~~ or modules installed as an independent structure. Solar PV panels~~/~~ or modules that are independent structures and do not have accessible /occupied space underneath are not required to accommodate a roof/PV live load, provided ~~they are marked as required in Section 1607.12.5.1, and~~ the area under the structure is restricted to keep the public away. All other loads and combinations in accordance with Section 1605 shall be accommodated.

Solar PV panels~~/~~ or modules that are designed to be the roof, and span to structural supports, and have accessible/occupied space underneath shall have the panels~~/~~ or modules and all supporting structure designed to support a roof/PV live load, as defined in Section 1607.12.5.1 in combination with other applicable loads. Solar PV panels~~/~~ or modules in this application are not permitted to be classified as not accessible in accordance with Section 1607.12.5.1.

1607.12.5.4 Ballasted systems. Solar PV panels~~/~~ or modules installed on a roof as a ballasted system need not be rigidly attached to the roof or supporting structure. Ballasted systems shall be designed and installed only on roofs with slopes of ~~½"~~ 1 inch per foot or less. The structural supports of the roof under a ballasted system 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. The ballasted system shall be designed to resist sliding and uplift resulting from lateral and vertical forces as required by Section 1605, using a coefficient of friction determined by acceptable engineering principles. In sites where the Seismic Design Category is C or above, the system shall be designed to accommodate seismic displacement determined by nonlinear response-history analysis or shake-table testing, using input motions consistent with ASCE 7 lateral and vertical seismic forces for non-structural components on roofs.

Committee Reason: This code change adds needed provisions for live loads related to solar photovoltaic panels and modules. The modification, which represents the consensus of the structural engineering community and the industry, reflects prior committee actions related to photovoltaics. It also clarifies treatment of live loads snow drifts, load combinations as well as seismic considerations.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment 1:

Joseph H. Cain, P.E., SolarCity Corporation, representing self and John Smirnow, Solar Energy Industries Association (SEIA), requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1603.1.8.1 ~~Solar photovoltaic (PV) panels or modules~~ Photovoltaic panel systems. The dead load of ~~solar PV panels or modules~~ rooftop mounted photovoltaic panel systems, including ~~accessories~~ rack support systems, shall be indicated on the construction documents.

1607.12.5 ~~Solar photovoltaic (PV) panels or modules~~ Photovoltaic panel systems. ~~Solar PV panels or modules~~ Roof structures that provide support for photovoltaic panels sytems shall be designed in accordance with Sections 1607.12.5.1 through 1607.12.5.4, as applicable.

(Portions of proposal not show remain unchanged)

Commenter's Reason: This change is intended to clarify the requirements using language that correlates with newly revised and approved terms while using language that can be easily understood by all users of the code. These revisions are provided in response to comments from the Structural Committee as part of their approval as modified of S72-12.

Sections 1603.1.8.1 and 1607.12.5 are revised for clarity, using newly defined term "photovoltaic panel system," as approved in S5-12.

Language is revised to clarify that this section applies to roof loads for design of the roof structure, not to the design of photovoltaic panels or modules themselves.

Public Comment 2:

Joseph H. Cain, P.E., SolarCity Corporation, representing self and John Smirnow, Solar Energy Industries Association (SEIA), requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1607.12.5.1 Roof~~/PV~~ live load. Roof ~~surfaces to be covered by solar PV panels or modules~~ structures that provide support for photovoltaic panel systems shall be designed for ~~the~~ roof live load, L_r, ~~assuming that the PV panels or module are~~ for the load case when the photovoltaic panel system is not present. ~~The roof/PV live load in areas covered by solar PV panels or modules shall be in addition to the panel loading unless the area covered by each solar PV panel or module is inaccessible.~~ Where roof surfaces to be covered with photovoltaic panel systems are inaccessible, the design of covered portions of roof structures need not include roof live load. Areas where the clear space between the photovoltaic panels and the rooftop is 24 inches or less, or where signs are posted prohibiting storage under the panels, shall be considered inaccessible. Roof surfaces not covered by PV panels shall be designed for ~~the~~ roof live load.

(Portions of proposal not show remain unchanged)

Commenter's Reason: This change is intended to clarify the requirements using language that correlates with newly revised and approved terms while using language that can be easily understood by all users of the code. These revisions are provided in response to comments from the Structural Committee as part of their approval as modified of S72-12.

Section 1607.12.5.1 is revised for clarity, using newly defined term "photovoltaic panel system," as approved in S5-12. Language is revised to clarify that this section applies to roof loads for design of the roof structure, not to the design of photovoltaic panels or modules themselves. Language is revised to clarify this section is for design of roof structures, not the design of roof surfaces. An option for signage is included, consistent with the language in Interpretation Report IR 16-8, Solar Photovoltaic and Thermal Systems Review and Approval Requirements," by California Department of General Services, Division of the State Architect.

Public Comment 3:

Joseph H. Cain, P.E., SolarCity Corporation, representing self and John Smirnow, Solar Energy Industries Association (SEIA), requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1607.12.5.2 ~~PV panels or modules~~ Other roof loads. ~~The~~ Roof structures ~~of a roof~~ that provide supports ~~solar PV panels or modules~~ for photovoltaic panel systems shall be designed to ~~accommodate the full solar PV panels or modules and~~ resist applicable loads from rack support systems. Design loads shall include photovoltaic panel system dead load including ballast dead load, ~~including concentrated loads from support frames~~ if any in combination with ~~the~~ roof live load~~s~~ from Section 1607.12.5.1 and other applicable loads. Where applicable, snow drift loads created by the ~~PV panels or modules~~ photovoltaic panel systems shall be included.

(Portions of proposal not show remain unchanged)

Commenter's Reason: This change is intended to clarify the requirements using language that correlates with newly revised and approved terms while using language that can be easily understood by all users of the code. These revisions are provided in response to comments from the Structural Committee as part of their approval as modified of S72-12.

Section 1607.12.5.2 is revised for clarity, using newly defined term "photovoltaic panel system," as approved in S5-12. Language is revised to clarify that this section applies to roof loads for design of the roof structure, not to the design of photovoltaic panels or modules themselves. Statements have been rearranged for clarity.

Public Comment 4:

Joseph H. Cain, P.E., SolarCity Corporation, representing self and John Smirnow, Solar Energy Industries Association (SEIA), requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

~~**1607.12.5.3 PV panels/ or modules installed as an independent structure.** Solar PV panels/ or modules that are independent structures and do not have accessible /occupied space underneath are not required to accommodate a roof/PV live load, provided the area under the structure is restricted to keep the public away. All other loads and combinations in accordance with Section 1605 shall be accommodated.~~

~~Solar PV panels/ or modules that are designed to be the roof, and span to structural supports, and have accessible/occupied space underneath shall have the panels/ or modules and all supporting structure designed to support a roof/PV live load, as defined in Section 1607.12.5.1 in combination with other applicable loads. Solar PV panels/ or modules in this application are not permitted to be classified as not accessible in accordance with Section 1607.12.5.1.~~

1607.12.5.3 Freestanding photovoltaic panel systems. Design loads for freestanding, ground mounted photovoltaic panel systems with no occupied space underneath need not include roof live load. All other loads and load combinations in accordance with Section 1605 shall be considered.

Photovoltaic panel systems mounted on raised support structures with open grid framing and no roof deck, and with accessible and occupied space underneath, shall have the supporting structure designed to support a reducible roof live load, in combination with other applicable loads. Solar PV panels or modules in this application are not permitted to be classified as inaccessible per Section 1607.12.5.1.

(Portions of proposal not show remain unchanged)

Commenter's Reason: This change is intended to clarify the requirements using language that correlates with newly revised and approved terms while using language that can be easily understood by all users of the code. These revisions are provided in response to comments from the Structural Committee as part of their approval as modified of S72-12.

Section 1607.12.5.3 is revised for clarity, using newly defined term "photovoltaic panel system," as approved in S5-12. The term "freestanding" is used to replace "independent," to be consistent with the language in ICC-ES AC428, "Acceptance Criteria for Modular Framing Systems used to Support Photovoltaic (PV) Modules." Language is revised to clarify that the first paragraph applies to ground mounted systems with no occupancy below, and the second paragraph applies to freestanding structures with occupancy below, such as solar support structures over vehicle parking spaces. Language is clarified to indicate this section applies to design of rack support systems and support structures, not to the design of photovoltaic panels or modules themselves. Language has been revised to "structures with open grid framing and no roof deck," consistent with Interpretation Report IR 16-8, Solar Photovoltaic and Thermal Systems Review and Approval Requirements" by California Department of General Services, Division of the State Architect. Statements have been rearranged for clarity**.**

Public Comment 5:

Joseph H. Cain, P.E., SolarCity Corporation, representing self and John Smirnow, Solar Energy Industries Association (SEIA), requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1607.12.5.4 Ballasted <u>photovoltaic panel</u> systems. ~~Solar PV panels/ or modules installed on a roof as a ballasted system need not be rigidly attached to the roof or supporting structure. Ballasted systems shall be designed and installed only on roofs with slopes of ½" 1 inch per foot or less. The structural supports of the roof under a~~ <u>Roof structures that provide support for</u> ballasted <u>photovoltaic panel</u> system<u>s</u> 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. ~~The ballasted system shall be designed to resist sliding and uplift resulting from lateral and vertical forces as required by Section 1605, using a coefficient of friction determined by acceptable engineering principles. In sites where the Seismic Design Category is C or above, the system shall be designed to accommodate seismic displacement determined by nonlinear response-history analysis or shake-table testing, using input motions consistent with ASCE 7 lateral and vertical seismic forces for non-structural components on roofs.~~

<u>**1613.5 Ballasted photovoltaic panel systems.** Ballasted, roof-mounted photovoltaic panel systems need not be rigidly attached to the roof or supporting structure. Ballasted non-penetrating systems shall be design and installed only on roofs with slopes of 1 inch per foot or less. Ballasted non-penetrating systems shall be designed to resist sliding and uplift resulting from lateral and vertical forces as required by Section 1605, using a coefficient of friction determined by acceptable engineering principles. In structures assigned to, Seismic Design Category C, D, E or F, ballasted non-penetrating the systems shall be designed to accommodate seismic displacement determined by nonlinear response-history analysis or shake-table testing, using input motions consistent with ASCE 7 lateral and vertical seismic forces for non-structural components on roofs.</u>

(Portions of proposal not show remain unchanged)

Commenter's Reason: This change is intended to clarify the requirements using language that correlates with newly revised and approved terms while using language that can be easily understood by all users of the code. These revisions are provided in response to comments from the Structural Committee as part of their approval as modified of S72-12.

Section 1607.12.5.3 is revised for clarity, using newly defined term "photovoltaic panel system," as approved in S5-12. Language is revised and re-ordered to clarify those statements in the first paragraph apply to all ballasted photovoltaic panel systems, and the statements in the second paragraph apply only to those ballasted systems that are "non-penetrating," and do not have anchorage to the roof structure. The second paragraph is relocated to new Section 1613.5, under Section 1613 Earthquake loads, as it is not appropriate under Section 1607.12 Roof loads.

S72-12

Final Action: AS AM AMPC____ D

S75-12
Table 1604.3, 1607.14, 1607.14.1

Proposed Change as Submitted

Proponent: Edwin Huston, National Council of Structural Engineers Associations (NCSEA), representing NCSEA Code Advisory Subcommittee – General Requirements Subcommittee (huston@smithhustoninc.com)

Revise as follows:

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

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

(Portions of table not shown remain unchanged)

1607.14 Interior walls and partitions. Interior walls and partitions that exceed 6 feet (1829 mm) in height, including their finish materials, shall have adequate strength and stiffness to resist the loads to which they are subjected but not less than a horizontal load of 5 psf (0.240 kN/m2).

> **Exception:** Fabric partitions complying with Section 1607.14.1 shall not be required to resist the minimum horizontal load of 5 psf (0.24 kN/m2).

1607.14.1 Fabric partitions. Fabric partitions that exceed 6 feet (1829 mm) in height, including their finish materials, shall have adequate strength and stiffness to resist the following load conditions:

1. A horizontal distributed load of 5 psf (0.24 kN/m2) applied to the partition framing. The total area used to determine the distributed load shall be the area of the fabric face between the framing members to which the fabric is attached. The total distributed load shall be uniformly applied to such framing members in proportion to the length of each member.
2. A concentrated load of 40 pounds (0.176 kN) applied to an 8-inch diameter (203 mm) area [50.3 square inches (32 452 mm2)] of the fabric face at a height of 54 inches (1372 mm) above the floor.

Reason: Currently Table 1604.3 does not have deflection limits for Live Loads on Interior walls. The 5.0psf requirement in section 1607.14 is classified as a live load and would not require a deflection check. Under the legacy Uniform Building Code this load was treated as an “other load” and was required to meet the deflection limits similar to those in IBC Table 1604.3. To avoid confusion for walls, and to require deflection checks on interior walls, the proposed code change is necessary.

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

T1604.3#2-S-HUSTON.doc

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This code change separates the deflection limits for interior partitions from those for exterior walls. Furthermore, it appropriately bases the interior partition limits on live load rather than wind.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Mark Nowak, MNowak Consulting, LLC, representing Steel Framing Alliance, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1607.14 Interior walls and partitions. Interior walls and partitions that exceed 6 feet (1829 mm) in height, including their finish materials, shall have adequate strength and stiffness to resist the loads to which they are subjected but not less than a horizontal load of 5 psf (0.240 kN/m2).

Exception<u>s</u>:

<u>1.</u> Fabric partitions complying with Section 1607.14.1 shall not be required to resist the minimum horizontal load of 5 psf (0.24 kN/m2).

<u>2.</u> <u>Interior non-load bearing walls and partitions of light-frame construction not exceeding 20 feet in height shall not be required to verify stiffness in compliance with the deflection limits of Table 1604.3.</u>

(Portions of code change proposal not shown remain unchanged)

Commenter’s Reason: The purpose of this public comment is to address a conflict made transparent by proposal S75-12 by adding partition wall deflection limits for all materials used in partition wall framing without ensuring coordination with existing prescriptive partition wall wood framing requirements which were only partially addressed in a separate proposal S285-12. This public comment will fully resolve the conflict and ensure that partition walls of conventional wood, cold-formed steel framing, engineered wood, and other light-frame materials are treated equitably with regard to conditions where deflection checks are and are not required by the code.

The following analysis (even when accounting for system stiffness) shows that the prescriptive conventional wood stud partition wall framing requirements in Section 2308 (Table 2308.9.1), do not meet the minimum deflection criteria instituted in proposal S75-12; thus, requiring this PC to ensure coordination for conditions addressed within the scope of Table 2308.9.1 (i.e., partition walls up to 20 feet in height) for light frame partition wall construction.

Wood Stud Size	Orientation	Species	Grade	E (in^2)	MOI (in^4)	Stud Spacing	Partition Height	Deflection (in)	Deflection Limit, L/
2x3	flatwise	SPF-South	Std	900,000	0.703	16	10	1.58	76
2x4	flatwise	SPF-South	Std	900,000	0.984	24	10	1.69	71
2x3	edgewise	SPF-South	Std	900,000	1.953	16	10	0.57	211
2x4	edgewise	SPF-South	Std	900,000	5.359	24	14	1.19	141
2x5	edgewise	SFP-South	Std	900,000	11.39	24	16	0.96	200
2x6	edgewise	SPF-South	Std	900,000	20.8	24	20	1.28	187

S75-12

Final Action: AS AM AMPC____ D

S77-12
Table 1604.3

Proposed Change as Submitted

Proponent: John Woestman, Kellen Company, representing Builders Masonry Veneer Manufacturers Association (MVMA) (jwoestman@kellencompany.com)

Revise as follows:

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

CONSTRUCTION	L	S OR W[f]	D + L[d, g]
Exterior walls and interior partitions:			
With plaster or stucco finishes	---	//360	---
With other brittle finishes[i]	---	//240	---
With flexible finishes	---	//120	---

<u>j. Includes adhered masonry veneer.</u>

(Portions of Table not shown remain unchanged)

Reason: This code proposal should help with a consistent deflection limit applied to wall systems with adhered masonry veneer.
Adhered masonry veneer does not have the large, flat, monolithic surface of plaster or stucco finishes. As such, adhered masonry veneer can accommodate more deflection.

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

T1604.3-S-WOESTMAN.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee believes that the deflection limit table is already clear on the treatment of adhered masonry veneer and there was no justification for adding the proposed footnote.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

John Woestman, Kellen Company, representing Masonry Veneer Manufacturers Association (MVMA), requests Approval as Submitted.

Commenter's Reason: This table in the IBC, since the 2000 IBC, has had a deflection limit for walls with brittle finishes of L/240. However, added to the 2012 IBC is the line item "With plaster or stucco finishes" and a deflection limit of L/360.
Unfortunately, we are seeing differences in interpretation regarding which deflection limit should apply to adhered masonry veneer.
Adhered masonry veneer, while it has similarities to plaster or stucco finishes, also has important differences. Unlike plaster or stucco finishes, adhered masonry veneer consists of numerous small units (i.e. manufactured stones, porcelain tiles, and the like) while plaster or stucco has large, flat, monolithic surfaces. Cracks as a result of deflection of adhered masonry veneer wall systems are much less detrimental to adhered masonry veneer.

We're proposing the footnote to Table 1604.3 to clarify the IBC deflection limit of L/240 applies to walls "With other brittle finishes" of adhered masonry veneer, as the IBC clearly required prior to the 2012 edition.

S77-12

Final Action: AS AM AMPC____ D

S79-12
202, 1602.1, 1604.4, 1610.1 1613.5.6.1

Proposed Change as Submitted

Proponent: Charles S. Bajnai, Chesterfield County, VA, ICC Building Code Action Committee

Delete without substitution:

SECTION 202
DEFINITIONS

DIAPHRAGM. A horizontal or sloped system acting to transmit lateral forces to the vertical-resisting elements. When the term "diaphragm" is used, it shall include horizontal bracing systems.

~~**Diaphragm flexible.** A diaphragm is flexible for the purpose of distribution of story shear and torsional moment where so indicated in Section 12.3.1 of ASCE 7.~~

~~**Diaphragm, rigid.** A diaphragm is rigid for the purpose of distribution of story shear and torsional moment when the lateral deformation of the diaphragm is less than or equal to two times the average story drift.~~

Revise as follows:

SECTION 1602
DEFINITIONS AND NOTATIONS

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

DIAPHRAGM.
Diaphragm, blocked.
Diaphragm boundary.
Diaphragm chord.
~~**Diaphragm flexible.**~~
~~**Diaphragm, rigid.**~~

(Portions of text not shown remains unchanged)

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

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

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

The total lateral force shall be distributed to the various vertical elements of the lateral force-resisting system in proportion to their rigidities, considering the rigidity of the horizontal bracing system or diaphragm. Rigid elements assumed not to be a part of the lateral force-resisting system are permitted to be incorporated into buildings provided their effect on the action of the system is considered and provided for in the design~~. Except where diaphragms are flexible, or are permitted to be analyzed as flexible,~~ Provisions shall be made for the increased forces induced on resisting elements of the structural system resulting from torsion due to eccentricity between the center of application of the lateral forces and the center of rigidity of the lateral force-resisting system, except where diaphragms are considered as flexible, permitted to be idealized as flexible or semi-rigid, in accordance with Section 12.3.1 of ASCE for seismic loads or Chapter 26 of ASCE 7 for wind loads.

Every structure shall be designed to resist the overturning effects caused by the lateral forces specified in this chapter. See Section 1609 for wind loads, Section 1610 for lateral soil loads and Section 1613 for earthquake loads.

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.

> **Exception:** Foundation walls extending not more than 8 feet (2438 mm) below grade and laterally supported at the top by ~~flexible~~ diaphragms considered as flexible, permitted to be idealized as flexible or semi-rigid, in accordance with Section 12.3.1 of ASCE for seismic loads or Chapter 26 of ASCE for wind loads shall be permitted to be designed for active pressure.

1613.3.5.1 Alternative seismic design category determination. Where S_1 is less than 0.75, the *seismic design category* is permitted to be determined from Table 1613.3.5(1) alone when all of the following apply:

1. In each of the two orthogonal directions, the approximate fundamental period of the structure, T_a, in each of the two orthogonal directions determined in accordance with Section 12.8.2.1 of ASCE 7, is less than 0.8 T_s determined in accordance with Section 11.4.5 of ASCE 7.
2. In each of the two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than T_s.
3. Equation 12.8-2 of ASCE 7 is used to determine the seismic response coefficient, C_s.
4. The diaphragms are rigid as defined in Section 12.3.1 of ASCE 7 or, for diaphragms that are considered flexible, permitted to be idealized as flexible or semi-rigid in accordance with Section 12.3.1 of ASCE 7, the distances between vertical elements of the seismic force-resisting system do not exceed 40 feet (12 192 mm).

Reason: The ICC Building Code Action Committee was asked to look at clearing up potential conflicts between the references to, and definitions of, flexible and rigid diaphragms in the IBC and ASCE-7-10. The BCAC did identify potential conflicts between the IBC's definition of a rigid diaphragm and the ASCE 7-10 criteria for classifying a diaphragm as rigid, semi-rigid or flexible. Also, it is considered inappropriate to include enforceable code requirements or references to standards as part of a definition. Thus, by this proposal, the BCAC proposes to remove the separate definitions for flexible and rigid diaphragms from the IBC and supply direct references in IBC Chapter 16 to the relevant requirements in the ASCE 7 seismic and wind chapters for when a diaphragm can be idealized as flexible or semi-rigid. This reference only occurs in the IBC in the sections noted in the code change proposal. In practical application, the code user will be turning to the requirements of ASCE-7 to categorize the diaphragm and perform the design. Therefore, there is no real need or advantage to provide the definitions in the IBC and this will prevent future maintenance of the terms and/or conflict between them.

For reference, ASCE 7-10 states,

12.3.1 Diaphragm Flexibility

The structural analysis shall consider the relative stiffnesses of diaphragms and the vertical elements of the seismic force-resisting system. Unless a diaphragm can be idealized as either flexible or rigid in accordance with Sections 12.3.1.1, 12.3.1.2, or 12.3.1.3, the structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e., semirigid modeling assumption).

12.3.1.1 Flexible Diaphragm Condition

Diaphragms constructed of untopped steel decking or wood structural panels are permitted to be idealized as flexible if any of the following conditions exist:

a. In structures where the vertical elements are steel braced frames, steel and concrete composite braced frames or concrete, masonry, steel, or steel and concrete composite shear walls.
b. In one-and two-family dwellings.
c. In structures of light-frame construction where all of the following conditions are met:
1. Topping of concrete or similar materials is not placed over wood structural panel diaphragms except for nonstructural topping no greater than 1 ½" in (38mm) thick.
2. Each line of vertical elements of the seismic force-resisting system complies with the allowable story drift of Table 12.12-1

12.3.1.2 Rigid Diaphragm Condition
Diaphragms of concrete slabs or concrete filled metal deck with span-to-depth ratios of 3 or less in structures that have no horizontal irregularities are permitted to be idealized as rigid.
12.3.1.3 Calculated Flexible Diaphragm Condition
Diaphragms not satisfying the conditions of Sections 12.3.1.1 or 12.3.1.2 are permitted to be idealized as flexible where the computed maximum in-plane deflection of the diaphragm under lateral load is more than two times the average story drift of adjoining vertical elements of the seismic force-resisting system of the associated story under equivalent tributary lateral load as shown in Fig. 12.3-1. The loadings used for this calculation shall be those prescribed by Section 12.8.

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

1604.4-S-BAJNAI-BCAC.doc

Public Hearing Results

Committee Action: Disapproved

Committee Reason: This proposal would introduce the term "semi-rigid diaphragm" into the IBC and actually conflict with ASCE 7. A public comment was suggested in hopes the various stakeholders are able to work out some of the conflicts.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment 1:

Chuck Bajnai, Chesterfield County, VA, representing ICC Building Code Action Committee, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

SECTION 202
DEFINITIONS

DIAPHRAGM. A horizontal or sloped system acting to transmit lateral forces to the vertical-resisting elements. When the term "diaphragm" is used, it shall include horizontal bracing systems.

Revise as follows:

SECTION 1602
DEFINITIONS AND NOTATIONS

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

DIAPHRAGM.
Diaphragm, blocked.
Diaphragm boundary.
Diaphragm chord.

(Portions of text not shown remains unchanged)

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

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

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

The total lateral force shall be distributed to the various vertical elements of the lateral force-resisting system in proportion to their rigidities, considering the rigidity of the horizontal bracing system or diaphragm. Rigid elements assumed not to be a part of the lateral force-resisting system are permitted to be incorporated into buildings provided their effect on the action of the system is considered and provided for in the design. <u>A diaphragm is rigid for the purpose of distribution of story shear and torsional moment when the lateral deformation of the diaphragm is less than or equal to two times the average story drift. Where required by ASCE 7,</u> provisions shall be made for the increased forces induced on resisting elements of the structural system resulting from torsion due to eccentricity between the center of application of the lateral forces and the center of rigidity of the lateral force-resisting system, ~~except where diaphragms are considered as flexible, permitted to be idealized as flexible or semi-rigid, in accordance with Section 12.3.1 of ASCE for seismic loads or Chapter 26 of ASCE 7 for wind loads~~.

Every structure shall be designed to resist the overturning effects caused by the lateral forces specified in this chapter. See Section 1609 for wind loads, Section 1610 for lateral soil loads and Section 1613 for earthquake loads.

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.

> **Exception:** Foundation walls extending not more than 8 feet (2438 mm) below grade and laterally supported at the top by <u>flexible</u> diaphragms ~~considered as flexible, permitted to be idealized as flexible or semi-rigid, in accordance with Section 12.3.1 of ASCE for seismic loads or Chapter 26 of ASCE for wind loads~~ shall be permitted to be designed for active pressure.

1613.3.5.1 Alternative seismic design category determination. Where S_1 is less than 0.75, the *seismic design category* is permitted to be determined from Table 1613.3.5(1) alone when all of the following apply:

1. In each of the two orthogonal directions, the approximate fundamental period of the structure, T_a, in each of the two orthogonal directions determined in accordance with Section 12.8.2.1 of ASCE 7, is less than 0.8 T_s determined in accordance with Section 11.4.5 of ASCE 7.
2. In each of the two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than T_s.
3. Equation 12.8-2 of ASCE 7 is used to determine the seismic response coefficient, C_s.
4. The diaphragms are rigid <u>or are permitted to be idealized as rigid in accordance with</u> ~~as defined in~~ Section 12.3.1 of ASCE 7 or, for diaphragms ~~that are considered flexible,~~ permitted to be idealized as flexible ~~or semi-rigid~~ in accordance with Section 12.3.1 of ASCE 7, the distances between vertical elements of the seismic force-resisting system do not exceed 40 feet (12 192 mm).

Commenter's Reason: The purpose of this public comment is to address issues raised by the FEMA Code Resource Support Community, NCSEA and others, including members of the ICC BCAC work group which developed this change. Four revisions are made to the original proposal:

1) IBC Section 1604.4 is further revised to eliminate conflicts between the proposed language and the ASCE 7 wind load provisions. For wind loads, an automatic exemption from torsional requirements only applies to one-story buildings less than 30 feet in height, one- and two-story light frame buildings, and one- and two-story buildings with flexible diaphragms. Buildings three or more stories in height with flexible diaphragms are not exempt from torsional wind load cases unless additional exemptions in ASCE 7-10 Appendix D based on building dimensions and symmetry of the vertical MWFRS apply. Thus, to avoid having the IBC incorrectly exempt a building from consideration of torsional effects, a simple reference to ASCE 7 is provided in lieu of the extended reference to the wind and seismic sections.
2) Also, the traditional building code definition of a rigid diaphragm is restored to Section 1604.4. This is necessary to avoid requiring semi-rigid analysis per ASCE 7 for a large number of buildings for which such an analysis has not been done in the past and is neither necessary nor an effective use of the engineer's time.
3) The original 2012 IBC language for IBC Section 1610.1 is restored. This section is intended for design of foundation walls to resist active or passive soil pressure, which is a function solely of the soil classification and the diaphragm flexibility. Wind and seismic design requirements do not come into play. Also, a semi-rigid diaphragm will probably be too stiff to permit the use of active pressures. The revisions will leave selecting the appropriate criteria to justify a flexible diaphragm assumption to the engineer's judgment.

4) IBC Section 1613.5.6.1, Item #4 is further revised to more closely mirror ASCE 7 Section 11.6. The key alignment is to use the "permitted to be idealized as flexible" language from ASCE 7 Sections 12.3.1.2 and 12.3.1.3. Thus, the current "considered flexible" phrasing should be deleted and replaced with the ASCE statement. Also, the 40-foot limitation does not apply when a semi-rigid modeling assumption is used because the actual stiffness of the diaphragm will be taken into account. Thus, the reference to semi-rigid diaphragms should be deleted.

Public Comment 2:

Philip Brazil, P.E., S.E., representing self, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

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

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

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

The total lateral force shall be distributed to the various vertical elements of the lateral force-resisting system in proportion to their rigidities, considering the rigidity of the ~~horizontal bracing system or~~ diaphragms. Rigid elements assumed not to be a part of the lateral force-resisting system are permitted to be incorporated into buildings provided their effect on the action of the system is considered and provided for in the design. Provisions shall be made for the increased forces induced on resisting elements of the structural system resulting from torsion due to eccentricity between the center of application of the lateral forces and the center of rigidity of the lateral force-resisting system, except where diaphragms are ~~considered as flexible,~~ permitted to be idealized as flexible ~~or semi-rigid,~~ in accordance with Section 12.3.1 of ASCE 7 for seismic loads or Chapter 26 of ASCE 7 for wind loads.

Every structure shall be designed to resist the overturning effects caused by the lateral forces specified in this chapter. See Section 1609 for wind loads, Section 1610 for lateral soil loads and Section 1613 for earthquake loads.

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.

Exception: Foundation walls extending not more than 8 feet (2438 mm) below grade and laterally supported at the top by diaphragms ~~considered as flexible,~~ permitted to be idealized as flexible ~~or semi-rigid,~~ in accordance with Section 12.3.1 of ASCE 7 for seismic loads or Chapter 26 of ASCE 7 for wind loads shall be permitted to be designed for active pressure.

1613.3.5.1 Alternative seismic design category determination. Where S_1 is less than 0.75, the *seismic design category* is permitted to be determined from Table 1613.3.5(1) alone when all of the following apply:

1. In each of the two orthogonal directions, the approximate fundamental period of the structure, T_a, in each of the two orthogonal directions determined in accordance with Section 12.8.2.1 of ASCE 7, is less than 0.8 T_s determined in accordance with Section 11.4.5 of ASCE 7.
2. In each of the two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than T_s.
3. Equation 12.8-2 of ASCE 7 is used to determine the seismic response coefficient, C_s.
4. The diaphragms are ~~rigid as defined in~~ permitted to be idealized as rigid in accordance with Section 12.3.1 of ASCE 7 or, for diaphragms ~~that are considered flexible,~~ permitted to be idealized as flexible ~~or semi-rigid~~ in accordance with Section 12.3.1 of ASCE 7, the distances between vertical elements of the seismic force-resisting system do not exceed 40 feet (12 192 mm).

(Portions of code change proposal not shown remain unchanged)

Reason: The purpose for the public comment is to adjust the original proposal so that it is compatible with ASCE 7-10. The original proposal contained several conflicts with ASCE 7-10 and the public comment eliminates them so that the IBC effectively scopes the technical provisions of ASCE 7-10 for diaphragms.

For the seismic design requirements of ASCE 7-10, Section 12.3.1 requires the structural analysis to explicitly include consideration of diaphragm stiffness (e.g., semi-rigid) unless the diaphragm can be idealized as flexible or rigid. Procedures permitting diaphragms to be idealized as flexible or rigid are specified in Sections 12.3.1.1, 12.3.1.2 and 12.3.1.3.

For the wind load requirements of ASCE 7-10, the definition of "diaphragm" in Section 26.2 includes the statement that “diaphragms constructed of wood structural panels are permitted to be idealized as flexible.”

In the second paragraph of IBC Section 1604.4, the public comment also deletes “horizontal bracing system,” which is redundant given the definition of “diaphragm” in IBC Section 202 that includes horizontal bracing systems. The deletion also eliminates an internal conflict in that the first sentence requires consideration of the rigidity of the “horizontal bracing system or diaphragm” but the last sentence only exempts qualifying diaphragms from the requirement for considering the increased forces resulting from torsion.

S79-12

Final Action: AS AM AMPC____ D

S81-12
1604.5.1

<u>Proposed Change as Submitted</u>

Proponent: Edwin Huston, National Council of Structural Engineers Associations (NCSEA), representing NCSEA Code Advisory Subcommittee – General Requirements Subcommittee (huston@smithhustoninc.com)

Revise as follows:

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

> <u>**Exception:** A single public assembly room with an occupant load of less than 500 shall be allowed in a *Risk Category II* building or structure and not be considered a multiple occupancy or a separate occupancy.</u>

Reason: The revision to 1604.5.1 will allow a single, modest meeting room or auditorium within an office building (a Risk Category II Building) without requiring the entire building to be designed as a Risk Category III.

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

1604.5.1-S-HUSTON

<u>Public Hearing Results</u>

Committee Action: **Disapproved**

Committee Reason: The proposed exception for multiple occupancies needs further clarification. The committee would prefer to see some information presented on the occupant load trigger of 500 that was originally proposed.

Assembly Action: **None**

<u>Individual Consideration Agenda</u>

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

Public Comment:

Edwin Huston, National Council of Structural Engineers Associations (NCSEA), representing NCSEA Code Advisory Subcommittee – General Requirements Subcommittee, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

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

~~Exception:~~ ~~A single public assembly room with an occupant load of less than 500 shall be allowed in a *Risk Category II* building or structure and not be considered a multiple occupancy or a separate occupancy.~~ <u>For the purposes of assigning a Risk Category in Table 1604.5 only, an Office building that would be assigned to Risk Category II on the basis of its primary occupancy, and has an occupant load of less than 4,500, shall be allowed to contain one assembly room or area with an occupant load of less than 500. All other requirements for Use, Occupancy and Means of Egress would remain as required by all other provisions of this Code.</u>

Commenter's Reason: This code change would allow a single, modest meeting room or auditorium within an office building (a Risk Category II Building) without requiring the entire building to be designed as a Risk Category III. The total occupancy load of the combined uses would be less than the 5000 as currently allowed by the table. The 500 occupant load also matches what is allowed for an adult educational use.

This Public Comment has revised the change to have the exception clarify that any other requirements relating to Use, Occupancy and Means of Egress (all non- structural concerns) are not to be altered. This change would actually reduce the cost of construction of certain office buildings.

S81-12

Final Action: AS AM AMPC____ D

S86-12
1605.2

<u>Proposed Change as Submitted</u>

Proponent: Edwin Huston, National Council of Structural Engineers Associations (NCSEA), representing NCSEA Code Advisory Subcommittee – General Requirements Subcommittee (huston@smithhustoninc.com)

Revise as follows:

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

$1.4(D + F)$	**(Equation 16-1)**
$1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$	**(Equation 16-2)**
$1.2(D + F) + 1.6(L_r \text{ or } S \text{ or } R) + 1.6H + (f_1L \text{ or } 0.5W)$	**(Equation 16-3)**
$1.2(D + F) + 1.0W + f_1L + 1.6H + 0.5(L_r \text{ or } S \text{ or } R)$	**(Equation 16-4)**
$1.2(D + F) + 1.0E + f_1L + 1.6H + f_2S$	**(Equation 16-5)**
$0.9D + 1.0W + 1.6H$	**(Equation 16-6)**
$0.9(D + F) + 1.0E + 1.6H$	**(Equation 16-7)**

where:

f_1 = 1 for places of public assembly live loads in excess of 100 pounds per square foot (4.79 kN/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.
3. <u>Crane wheel 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. Alternatively, industry standard reference documents citing additional crane load combinations shall be permitted for the design of buildings subject to horizontal and vertical crane loads.</u>

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

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

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

Exceptions:

1. Crane ~~hook~~ wheel 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. Alternatively, industry standard reference documents citing additional crane load combinations shall be permitted for the design of buildings subject to horizontal and vertical crane loads.
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.

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

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

Exceptions:

1. Crane ~~hook~~ wheel 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. Alternatively, industry standard reference documents citing additional crane load combinations shall be permitted for the design of buildings subject to horizontal and vertical crane loads.
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: Current code language does not completely or adequately address the issue of load combinations for the design of buildings with bridge cranes. This includes buildings and other structures that have multiple crane runways adjacent to one another and/or multiple cranes on the same runway. An exception pointing to industry standard reference documents, such as the Association of Iron and Steel Technology (AIST) “Technical Report No. 13 - Guide for the Design and Construction of Mill Buildings”, allows the engineer to utilize such resources when determining additional load combinations that may control in the design of such buildings.

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

1605.2-S-HUSTON

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: This code change had many wording problems that need to be worked out. The committee finds the phrase "Alternatively industry standard reference documents shall be permitted……" to be problematic.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Daniel J. Walker, P.E., Thomas Associates, Inc, representing Metal Building Manufacturers Association, requests Approval as Modified by this Public Comment.

Replace the proposal as follows:

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

$1.4(D + F)$ **(Equation 16-1)**
$1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$ **(Equation 16-2)**
$1.2(D + F) + 1.6(L_r \text{ or } S \text{ or } R) + 1.6H + (f_1L \text{ or } 0.5W)$ **(Equation 16-3)**
$1.2(D + F) + 1.0W + f_1L + 1.6H + 0.5(L_r \text{ or } S \text{ or } R)$ **(Equation 16-4)**
$1.2(D + F) + 1.0E + f_1L + 1.6H + f_2S$ **(Equation 16-5)**
$0.9D + 1.0W + 1.6H$ **(Equation 16-6)**
$0.9(D + F) + 1.0E + 1.6H$ **(Equation 16-7)**

where:

f_1 = 1 for places of public assembly live loads in excess of 100 pounds per square foot (4.79 kN/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.
3. <u>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.</u>

Commenter's Reason: The committee disapproved the original proposal because in addition to trying to make all of the load combinations consistent with respect to crane loads plus other transient loads, it tried to permit alternate industry standard reference documents for the load combinations that include crane loads and this was found to be problematic. This public comment seeks to just correct the inconsistency between the allowable stress load combinations and the strength load combinations that currently exists in the IBC, i.e. the proposed new Exception 3 in Section 1605.2 mirrors the exception that already exists for the allowable load combinations and alternate allowable load combinations in Sections 1605.3.1 and 1605.3.2, respectively, in Exception 1

S86-12
Final Action: AS AM AMPC____ D

S87-12
202, Table 1607.1

Proposed Change as Submitted

Proponent: Gary J. Ehrlich, P.E., National Association of Home Builders (NAHB) (gehrlich@nahb.org)

Delete without substitution:

SECTION 202
DEFINITIONS

~~**MARQUEE.** A *canopy* that has a top surface which is sloped less than 25 degrees from the horizontal and is located less than 10 feet (3.05 m) from operable openings above or adjacent to the level of the marquee.~~

Revise as follows:

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
~~21. Marquees~~	~~75~~	~~-~~
26. Roofs		
All roof surfaces subject to maintenance workers		300
Awnings and canopies:		
Fabric construction supported by a skeleton structure	5 Nonreducible	
All other construction	20[n]	
Ordinary flat, pitched, and curved roofs (that are not occupiable)	20	
Where primary roof members are exposed to a work floor, at single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs:		
Over manufacturing, storage warehouses, and repair garages		2,000
All other primary roof members		300
Occupiable roofs:		
Roof gardens	100	
Assembly areas	100[m]	
All other similar areas	Note 1	Note 1

n Where a canopy has a top surface sloped less than 25 degrees from the horizontal and is located less than 10 feet (3.05 m) from operable openings above or adjacent to the level of the canopy, the minimum live load shall be taken as the live load of the adjacent room or space, but not less than 40psf. The maximum live load for canopies less than or equal to 100 square feet in area shall be 60psf.

(Portions of Table and footnotes not shown remain unchanged)

Reason: The purpose of this amendment is to revise the 2012 IBC language regarding canopies and marquees. The language approved for the 2012 IBC will substantially change the design requirements for many small porch and patio roofs on buildings nowhere near public streets. These roofs are currently designed for standard roof live loads or local ground snow loads (typically in the range of 20 or 30 pounds per square foot). These elements will now need to be designed for 75psf if they happen to be less than

10 feet vertically from a window above or horizontally from a window at the level of the canopy. This represents a substantial increase in design requirements for apartment or condominium complexes with these elements, as well as a substantial issue for renovations. This change deletes the definition for marquees in its entirety and transfers the language regarding canopy slope and ability to access the top surface from nearby openings to a footnote on the standard canopy live load. It also requires the window to be operable. The live load for the accessible canopy condition is set to the adjacent occupancy, with a minimum floor of 40psf (equivalent to the traditional load for a residential deck). To avoid effectively further raising the live load requirement from 75psf to 100psf for a small canopy accessible from an egress hallway or stair, a maximum live load of 60psf is established for canopies not exceeding 100 square feet in area (similar to what the traditional load cases were for residential balconies).

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

T1607.1-S-EHRLICH.doc

Public Hearing Results

Committee Action: Disapproved

Committee Reason: The proposal would remove the definition of marquee which in turn leaves Section 3106 without the definition that ties it into code requirements. The increased canopy loads may have been an unintended consequence of prior code changes, but come up with an alternative that leaves the definition of marquees.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Gary J. Ehrlich, P.E, representing National Association of Home Builders (NAHB), requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

MARQUEE. A canopy that has a top surface which is sloped less than 25 degrees from the horizontal and is located less than 10 feet (3.05 m) from operable openings above or adjacent to the level of the marquee.

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
21. Marquees, except one- and two-family dwellings	75	—
~~21~~22. Office Buildings		
~~22~~23. Penal Institutions		
~~23~~24. Recreational uses		
~~24~~25. Residential One- and two-family dwellings Uninhabitable attics without storage[i] Uninhabitable attics with storage [i, j, k] Habitable attics and sleeping areas[k] Canopies, including marquees All other areas Hotels and multifamily dwellings Private rooms and corridors serving them Public rooms[m] and corridors serving them	 10 20 30 20 40 40 100	
~~25~~26. Roofs All roof surfaces subject to maintenance workers Awnings and canopies: Fabric construction supported by a skeleton structure All other construction, except one- and two-family dwellings Ordinary flat, pitched, and curved roofs (that are not occupiable)	 5 nonreducible 20[a]	 300

Where primary roof members are exposed to a work floor, at single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs:	20	
Over manufacturing, storage warehouses, and repair garages		
All other primary roof members		
Occupiable roofs:		
Roof gardens		
Assembly areas		2,000
All other similar areas		300
	100	
	100[m]	
	Note l	Note l

a. ~~Where a canopy has a top surface sloped less than 25 degrees from the horizontal and is located less than 10 feet (3.05 m) from operable openings above or adjacent to the level of the canopy, the minimum live load shall be taken as the live load of the adjacent room or space, but not less than 40psf. The maximum live load for canopies less than or equal to 100 square feet in area shall be 60psf.~~

(Portions of Table and footnotes not shown remain unchanged)

Commenter's Reason: The purpose of this public comment is to revise our proposal to address issues raised by the IBC Structural Committee and testimony from the floor. The two primary issues were that the committee and testifiers noted a need to retain a definition for marquees, to go with the provisions of Section 3106, and to retain the higher live load for the types of appurtenances likely to be climbed by drunk football fans or used by rock bands filming videos.

NAHB's concern with NCSEA's change last cycle is that it could result in a significant design load increase for canopies and canopy-like structures (porch and patio roofs) associated with Group R-3 dwellings and townhouses and with Group R-2 low-rise apartment and condominium buildings. In addressing this issue, it is difficult to separate Group R-2 buildings in urban environments, where NCSEA's concerns may be applicable, with Group R-2 buildings in planned communities in the suburbs where many of the issues likely do not exist. Addressing Group R-3 dwellings and townhouses, however, can be more easily accomplished. The occupant load of Group R-3 structures is low, so even if a flat or low-slope canopy or canopy-like (porch or patio) roof is used for egress or the family chooses to sit on it to watch fireworks the loads are light and the standard 20psf roof live load is sufficient.

So, the proposal is amended to replace the proposed footnote with an added line under table 1607.1 Item 21 – Residential – One and two-family dwellings for canopies (including marquees) with a live load of 20psf, regardless of roof slope, access or support conditions. This will restore the traditional design requirement for Group R-3 dwellings and maintain consistency with the IRC.

This public comment also restores the definition for marquees as requested by the committee to coordinate with the design provisions for marquees in IBC Section 3106. It is noted that Section 3106.5 indicates that a "marquee" must be supported entirely off of the building, which leaves a potential conflict with the definition in that a canopy supported at both the building and on independent columns becomes a "marquee" if it has a low-slope roof. It is left to future code cycles to address this conflict.

S87-12

Final Action: AS AM AMPC____ D

S94-12
1607.10.2

<u>Proposed Change as Submitted</u>

Proponent: Gary R. Searer/Wiss, Janney, Elstner Associates, Inc., representing self

Revise as follows:

1607.10.2 Alternative uniform live load reduction. As an alternative to Section 1607.10.1 and subject to the limitations of Table 1607.1, uniformly distributed live loads are permitted to be reduced in accordance with the following provisions. Such reductions shall apply to slab systems, beams, girders, columns, piers, walls and foundations.

1. A reduction shall not be permitted where the live load exceeds 100 psf (4.79 kN/m^2) except that the design live load for members supporting two or more floors is permitted to be reduced by a maximum of 20 percent.

   Exception: For uses other than storage, where *approved*, additional live load reductions shall be permitted where shown by the *registered design professional* that a rational approach has been used and that such reductions are warranted.

2. A reduction shall not be permitted in passenger vehicle parking garages except that the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent.
3. For live loads not exceeding 100 psf (4.79 kN/m^2), the design live load for any structural member supporting 150 square feet (13.94 m^2) or more is permitted to be reduced in accordance with Equation 16-24.
4. For one-way slabs, the area, *A*, for use in Equation 16-24 shall not exceed the product of the slab span and a width normal to the span of 0.5 times the slab span.

$R = 0.08(A – 150)$ **(Equation 16-24)**

For SI: $R = 0.861(A – 13.94)$

Such reduction shall not exceed the smallest of:

1. 40 percent for ~~horizontal~~ members <u>supporting one floor</u>;
2. 60 percent for ~~vertical~~ members <u>supporting two or more floors</u>; or
3. *R* as determined by the following equation.

$R = 23.1(1+ D/L_o)$ **(Equation 16-25)**

where:

A = Area of floor supported by the member, square feet (m^2).
D = Dead load per square foot (m^2) of area supported.
L_o = Unreduced live load per square foot (m^2) of area supported.
R = Reduction in percent.

Reason: The alternate live load reductions contained in Section 1607.9.2 originated in the Uniform Building Code and were the primary live load reduction formulas used in the western United States for decades. When the live load reductions were brought into the IBC, they were incorporated as an alternate to Section 1607.9.1. During the incorporation of these reductions into the IBC, the maximum reductions were changed from “40 percent for members receiving load from one level only” and “60 percent for other members” (in the 1997 UBC) to the current 40/60 differentiation between horizontal and vertical members. This current differentiation does not match the original wording (because some horizontal members receive live load from more than one floor and because many vertical elements do not receive live load from more than one floor) and does not match the differentiation in Section 1607.9.1, which, like the UBC, differentiates reductions based on whether a member supports one floor or more than one floor: "L shall not be less than $0.50L_o$ for members supporting one floor and L shall not be less than $0.40L_o$ for members supporting two or more floors." The premise behind differentiating between supporting one floor or more than one floor is basically probability-based, and reasonably assumes that the probability that two or more floors are experiencing a relatively large live load is smaller than that of a single floor experiencing a relatively large live load; hence the larger reduction for elements that support more than one floor. The same premise cannot be said of differentiating live load reductions based on horizontality or verticality of the element under consideration.

Since basing allowable live load reductions on number of floors supported as opposed to whether a member is horizontal or vertical makes more sense, this proposal restores the original intent of the UBC provision and brings the provision into better alignment with Section 1607.9.1.

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

1607.9.2-S-SEARER.doc

Public Hearing Results

Committee Action: Disapproved

Committee Reason: The proposed clarification to the alternative live load reduction method, seemed reasonable but the omission of roof loads was not adequately explained.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Gary Searer, Wiss Janney, Elstner Associates, Inc. (WJE), representing self, requests Approval as Submitted.

Commenter's Reason: During the code hearings and the consideration of S94, the IBC-Structural Committee raised a question regarding reduction of roof live loads in the 1997 UBC versus how they are handled in the 2012 IBC. Since no one had a copy of the language from the 1997 UBC, the Committee opted to disapprove the proposed change until the question could be answered.

As it turns out, roof live loads are not an issue, because roof live load reductions are handled via a different method. The language in this code change proposal is well thought out. The proposal corrects a mistake that was made years ago in moving the UBC language into the IBC.

The alternate live load reductions contained in Section 1607.9.2 originated in the *Uniform Building Code* and were the primary live load reduction formulas used in the western United States for decades. When the live load reductions were brought into the IBC, they were incorporated as an alternate to Section 1607.9.1. During the incorporation of these reductions into the IBC, the maximum reductions were changed from "40 percent for members receiving load from one level only" and "60 percent for other members" (in the 1997 UBC) to the current 40/60 differentiation between horizontal and vertical members.

This current differentiation does not match the original wording (because some horizontal members receive live load from more than one floor and because many vertical elements do not receive live load from more than one floor) and does not match the differentiation in IBC Section 1607.9.1, which, like the UBC, differentiates reductions based on whether a member supports one floor or more than one floor: "L shall not be less than $0.50L_o$ for members supporting one floor and L shall not be less than $0.40L_o$ for members supporting two or more floors."

The premise behind differentiating between supporting one floor or more than one floor is basically probability-based, and reasonably assumes that the probability that two or more floors are experiencing a relatively large live load is smaller than that of a single floor experiencing a relatively large live load; hence the larger reduction for elements that support more than one floor. The same premise cannot be said of differentiating live load reductions based on horizontality or verticality of the element under consideration, which is what the existing language does.

To correct this error, I respectfully ask that this code change be considered for approval as submitted.

S94-12

Final Action: AS AM AMPC____ D

S97-12
1609.1.1, Chapter 35 (New)

Proposed Change as Submitted

Proponent: Ray C. Minor, P.E., Hapco, representing self (ray.minor@hapco.com)

Revise as follows:

1609.1.1 Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapters 26 to 30 of ASCE 7 or provisions of the alternate all-heights method in Section 1609.6. The type of opening protection required, the ultimate design wind speed, *Vult*, 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 AF&PA WFCM.
3. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AISI S230.
4. Designs using NAAMM FP 1001.
5. Designs using TIA-222 for antenna-supporting structures and antennas, provided the horizontal extent of Topographic Category 2 escarpments in Section 2.6.6.2 of TIA-222 shall be 16 times the height of the escarpment.
6. Wind tunnel tests in accordance with Chapter 31 of ASCE 7.
7. <u>Luminaire support structures designed in accordance with AASHTO LTS-5.</u>

The wind speeds in Figures 1609A, 1609B and 1609C are ultimate design wind speeds, *Vult*, and shall be converted in accordance with Section 1609.3.1 to nominal design wind speeds, *Vasd*, when the provisions of the standards referenced in Exceptions 1 through 5 <u>and 7</u> are used.

Add new standard to Chapter 35 as follows:

<u>AASHTO</u>

<u>American Association of State Highway and Transportation Officials</u>
<u>444 North Capitol Street, NW Suite 249</u>
<u>Washington, DC 20001</u>

<u>LTS-5 Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals</u>

Reason: AASHTO LTS-5 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 type structures are also used for non-roadway applications such as sports lighting and parking lot lighting which may fall under the jurisdiction of the IBC. AASHTO LTS-5 incorporates the results of wind tunnel tests specific to shapes of these structures and the equipment they support. The wind pressure calculations are based on ASCE-7 except with some refinements such as more detailed drag coefficients. Stadium lighting poles involved in several recent failures would not meet the fatigue requirements of AASHTO LTS-5 primarily because the base plates were too thin. These failures most likely would not have occurred if the poles were designed to AASHTO LTS-5.

AASHTO LTS-5 is developed by an AASHTO committee with a consensus procedure.

There are other exceptions as precedents for this exception, including similar specifications for flagpoles and communications antennae. The flagpole specification NAAMM 1001 Guide Specification for Design of Metal Flagpoles includes flag wind load equations but otherwise uses the AASHTO LTS-5 procedures for flagpoles

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

1609.1.1-S-MINOR

Public Hearing Results

Note: For staff analysis of the content of AASHTO LTS-5 relative to CP#28, Section 3.6, please visit: http://www.iccsafe.org:8888/cs/codes/Documents/2012-13cycle/Proposed-A/00a_updates.pdf

Committee Action: Disapproved

Committee Reason: The committee is not convinced that luminaire support structures need to be addressed in the code. These are typically in the right-of-way and not regulated by the IBC.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment 1:

Ray C. Minor, Hapco, representing self, requests Approval as Submitted

Commenter's Reason: The IBC-S Code Committee's stated reason for disapproval was "The committee is not convinced that luminaire support structures need to be addressed in the code. These are typically in the right-of-way and not regulated by the IBC." My reply to this is that the two largest manufactures of luminaire support structures in the US (Hapco and Valmont) estimate that half of these structures they sell are for non-roadway use.

Except for using an earlier version of the AASHTO specification, the proposed change is already in the Florida Building Code-2010:

1609.1.1 Exception 7. Designs using AASHTO LTS-4 Structural Specifications for Highway Signs, Luminaires and Traffic Signals.

Public Comment 2:

Michael Fedlberg, P.E., Minnesota, Florida/Valmont Industries Inc., representing self, requests Approval as Submitted.

Commenter's Reason: AASHTO LTS-5 Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals is the only Specification dedicated to pole type structures. It covers them in detail and is highly respected by all pole manufacturers in the United States. ASCE 7 with Commentary does not have special provisions for structural supports for signs and luminaires. These metal pole type structures fall in the same category as Chimneys, Tanks and Similar Structures under Flexible Buildings and Structures in ASCE 7. Since ASCE 7 with Commentary does not provide guidelines for design of pole type structures, a reliable source must be used to determine appropriate formulas that are recognized and documented. In the case of lightpoles and similar structures, a logical source for these formulas and guidelines is the ASSHTO Specification LTS-5. Both ASCE 7 and ICC already recognize the NAAMM FP 1001 as an acceptable specification for the design of metal flagpoles. However, the procedures used to determine design loads for metal flagpole set forth in the AASHTO Specifications, please see the introduction to the NAAMM FP 1001 attached. I believe that ICC should accord the same recognition to the AASHTO Specification LTS-5 and accept the change proposed by Ray C. Minor.

The design of safe flagpoles requires knowledge of the loads to which they will be subjected. The principal loads acting on flagpoles are the wind loads, and it is these loads which must be most carefully determined. Maximum wind speeds to which flagpoles are exposed depend on the geographical location, whether or not it is in the center of a large city, the outskirts of a small town, the seashore at ground level or on the roof of a high rise building. Wind speeds are generally higher along coastal areas than inland. They are also higher in open country than in the center of cities, and wind speed increases with height above ground.

ASCE 7-05, page 300, third paragraph states "It is not the intent of this standard to exclude the use of other recognized literature for the design of special structures,….. For the design of flagpoles, see ANSI/NAAMM FP1001-97, 4th Ed., Guide Specifications for Design of Metal Flagpoles." This 5th Edition of the Guide follows the same design procedure as the 4th Edition.

The wind will exert a force on the pole itself as well as on the flag, and these two forces must be taken into consideration to determine the total load. Different size flags are flown from different poles, and it is important that flagpoles be selected which are capable of supporting the largest size flag intended to be flown under the highest speed wind to which it will be subjected. Loads on the flagpole are resisted by the mounting and the foundation or building structure (roof or wall) to which it is secured. The procedures used to determine wind loads on flagpoles are those set forth in the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals. There is sufficient similarity between flagpoles and the poles used for signs, lights and signals to justify this approach. Furthermore, there has been a vast amount of knowledge and data accumulated by AASHTO on the requirements for pole design because of experience with many types of poles installed all across the country which are subjected to the wind conditions occurring in these varied geographical locations. NAAMM believes that the procedures developed by AASHTO over the years provide a sound basis for the determination of flagpole loadings without the flag flying

However, a flagpole's function is to fly flags, and hence this standard presents procedures for determining the loads applied to poles as a result of the wind loads on flags. The original procedures set forth in the first edition of this standard in 1983 were developed by NAAMM as a result of a laboratory test program conducted in the fall of 1979 in which flags attached to a flagpole were subjected to winds generated by an aircraft engine and propeller. There were limitations to flag sizes and wind speeds in this program. Recognizing the limitations of the laboratory test program, NAAMM initiated a program of actual flight testing of flags in sizes ranging from 5 ft x 8 ft (1.5 m x 2.4 m) through 20 ft x 30 ft (6.1 m x 9.1 m) and at air speeds from 60 mph (27 m/s) up to 110 mph (49 m/s). This flight test program, completed in the fall of 1984, yielded the most complete and reliable data obtained to date on the loading of flags under high speed wind conditions. The results of this test program provided the basis for the development of the flag drag formulas given in the later editions of these guide specifications.

In the determination of the pole design, the inclusion of the wind load on the flag with the wind load on the pole, provides an added degree of safety for the flagless pole. Flags are not always lowered when a high speed wind occurs. Under such a circumstance the flag can be ripped off of the pole. Some flags are made of materials such as nylon which are very strong and resist the tendency to rip away as flags in years past were prone to do. NAAMM recommends that flagpole designers consider both pole and flag loads when selecting a flagpole design. Building codes that do not take into account the load caused by the flag drag do not require a design as safe as that required by this standard. Nevertheless, the designer shall check to be certain that his design meets or exceeds the requirements of the governing building code.

This 5th edition of the standard has replaced the basic wind speed map found in the previous editions with the new wind speed map in ASCE 7-05 which is based on 3-second gust speeds.

Public Comment 3:

Carl J. Macchietto, Valmont Industries Inc., requests Approval as Submitted

Commenter's Reason: Valmont is in full support of this proposal. Approximately 50 percent of light pole structures are not on roadways. They are located in parking lots, building security lighting, and athletic fields. Referencing the AASHTO Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals seems prudent given that this specification specializes in the design of these types of structures.

S97-12

Final Action: AS AM AMPC____ D

S98-12
1609.1.1, 1609.3.1

Proposed Change as Submitted

Proponent: Randall Shackelford, P.E., Simpson Strong-Tie Company, Inc. (rshackelford@strongtie.com)

Revise as follows:

1609.1.1 Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapters 26 to 30 of ASCE 7 or provisions of the alternate all-heights method in Section 1609.6. The type of opening protection required, the ultimate design wind speed, V_{ult}, and the exposure category for a site is permitted to be determined in accordance with Section 1609 or ASCE 7. Wind shall be assumed to come from any horizontal direction and wind pressures shall be assumed to act normal to the surface considered.

Exceptions:

1. Subject to the limitations of Section 1609.1.1.1, the provisions of ICC 600 shall be permitted for applicable Group R-2 and R-3 buildings.
2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AF&PA WFCM.
3. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AISI S230.
4. Designs using NAAMM FP 1001.
5. Designs using TIA-222 for antenna-supporting structures and antennas, provided the horizontal extent of Topographic Category 2 escarpments in Section 2.6.6.2 of TIA-222 shall be 16 times the height of the escarpment.
6. Wind tunnel tests in accordance with Chapter 31 of ASCE 7.

The wind speeds in Figures 1609A, 1609B and 1609C are ultimate design wind speeds, V_{ult}, and shall be converted in accordance with Section 1609.3.1 to nominal design wind speeds, V_{asd}, when the provisions of the standards referenced in Exceptions ~~1~~ <u>3</u> through 5 are used.

1609.3.1 Wind speed conversion. When required, the ultimate design wind speeds of Figures 1609A, 1609B and 1609C shall be converted to nominal design wind speeds, V_{asd}, using Table 1609.3.1 or Equation 16-33.

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

where:

V_{asd}= nominal design wind speed applicable to methods specified in Exceptions ~~1~~ <u>3</u> through 5 of Section 1609.1.1 <u>and other standards not based on ultimate wind speeds</u>.
V_{ult}= ultimate design wind speeds determined from Figures 1609A, 1609B or 1609C.

Reason: The 2012 WFCM, as referenced in Exception 2 above, is based on Ultimate Wind Speeds, V_{ult}, and therefore does not require conversion of the ultimate wind speed to the nominal wind speed, V_{asd}.
Further, the WFCM is the reference standard for wood framing in the ICC-600, so conversion should not take place when using ICC-600 to design wood framing. A committee has been appointed to revise ICC-600, and this code change is written assuming that the basis of ICC-600 will be changed to V_{ult} windspeeds, with conversion factors in the standard for converting to V_{asd} where needed. If by the Public Comment deadline it is not clear that this will be the case, I will prepare a Public Comment to restore Exception 1 to the list of items where conversion is required.

If this code change is not approved, structures designed using the 2012 WFCM with converted windspeeds will be designed for pressures that are only 60% of the pressures they should be designed for.

Section 1609.3.1 needs to be revised for similar reasons. Also, there are other building materials that require testing to "nominal" windspeeds, such as composition shingles in Section 1507.2.7.1. So nominal wind speeds, V_{asd} , is not just used in the Exceptions to 16009.1.1.

Cost Impact: This is not really a fair question for this code change. Yes, there will be a cost impact, because it would definitely be cheaper to design to wind loads that are 40% too low. But you don't want to do that.

1609.1.1-S-SHACKELFORD.doc

Public Hearing Results

Committee Action: Approved as Modified

Modify proposal as follows:

1609.1.1 Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapters 26 to 30 of ASCE 7 or provisions of the alternate all-heights method in Section 1609.6. The type of opening protection required, the ultimate design wind speed, V_{ult}, and the exposure category for a site is permitted to be determined in accordance with Section 1609 or ASCE 7. Wind shall be assumed to come from any horizontal direction and wind pressures shall be assumed to act normal to the surface considered.

Exceptions:

1. Subject to the limitations of Section 1609.1.1.1, the provisions of ICC 600 shall be permitted for applicable Group R-2 and R-3 buildings.
2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AF&PA WFCM.
3. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AISI S230.
4. Designs using NAAMM FP 1001.
5. Designs using TIA-222 for antenna-supporting structures and antennas, provided the horizontal extent of Topographic Category 2 escarpments in Section 2.6.6.2 of TIA-222 shall be 16 times the height of the escarpment.
6. Wind tunnel tests in accordance with Chapter 31 of ASCE 7.

The wind speeds in Figures 1609A, 1609B and 1609C are ultimate design wind speeds, V_{ult}, and shall be converted in accordance with Section 1609.3.1 to nominal design wind speeds, V_{asd}, when the provisions of the standards referenced in Exceptions 3 through 5 are used.

1609.3.1 Wind speed conversion. When required, the ultimate design wind speeds of Figures 1609A, 1609B and 1609C shall be converted to nominal design wind speeds, V_{asd}, using Table 1609.3.1 or Equation 16-33.

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

where:

V_{asd}= nominal design wind speed applicable to methods specified in Exceptions 3 through 5 of Section 1609.1.1 ~~and other standards not based on ultimate wind speeds~~.
V_{ult}= ultimate design wind speeds determined from Figures 1609A, 1609B or 1609C.

Committee Reason: This proposal corrects the exceptions that are referred to in regards to nominal design wind speeds for consistency. The modification removes a proposed reference to "other standards" that is too vague.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Bonnie Manley, American Iron and Steel Institute, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1609.1.1 Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapters 26 to 30 of ASCE 7 or provisions of the alternate all-heights method in Section 1609.6. The type of opening protection required, the ultimate design wind speed, V_{ult}, and the exposure category for a site is permitted to be determined in accordance with Section 1609 or ASCE 7. Wind shall be assumed to come from any horizontal direction and wind pressures shall be assumed to act normal to the surface considered.

Exceptions:

1. Subject to the limitations of Section 1609.1.1.1, the provisions of ICC 600 shall be permitted for applicable Group R-2 and R-3 buildings.
2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AF&PA WFCM.
3. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AISI S230.
4. Designs using NAAMM FP 1001.
5. Designs using TIA-222 for antenna-supporting structures and antennas, provided the horizontal extent of Topographic Category 2 escarpments in Section 2.6.6.2 of TIA-222 shall be 16 times the height of the escarpment.
6. Wind tunnel tests in accordance with Chapter 31 of ASCE 7.

The wind speeds in Figures 1609A, 1609B and 1609C are ultimate design wind speeds, V_{ult}, and shall be converted in accordance with Section 1609.3.1 to nominal design wind speeds, V_{asd}, when the provisions of the standards referenced in Exceptions ~~3 through~~ 4 and 5 are used.

1609.3.1 Wind speed conversion. When required, the ultimate design wind speeds of Figures 1609A, 1609B and 1609C shall be converted to nominal design wind speeds, V_{asd}, using Table 1609.3.1 or Equation 16-33.

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

where:

V_{asd}= nominal design wind speed applicable to methods specified in Exceptions ~~3 through~~ 4 and 5 of Section 1609.1.1
V_{ult}= ultimate design wind speeds determined from Figures 1609A, 1609B or 1609C.

Commenter's Reason: AISI has recently completed the development of Supplement 3-12 for AISI S230-07, which converts the standard to the Ultimate Wind Speed, V_{ult}, basis. Therefore, using it no longer requires conversion of the ultimate wind speed to the nominal wind speed, V_{as}, as specified in Section 1609.3.1. The modifications recommended in this public comment reflect this change.

AISI S230-07 w/S3-12 will be recommended for adoption during the ICC Group B Administrative update process in 2013. It can be downloaded for review from the AISI website: www.steel.org.

S98-12

Final Action: AS AM AMPC____ D

S102-12

202 (New), 1403.7, 1603.1.7, 1612.4, 1612.5, G103.7, G301.2, G401.2; IPC 309.3; IMC 301.16.1

Proposed Change as Submitted

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net).

Add new text as follows:

SECTION 202
DEFINITIONS

COASTAL A ZONE. Area within a special flood hazard area, landward of a V zone or landward of an open coast without mapped V Zones. In a coastal A zone, the principal source of flooding must be astronomical tides, storm surges, seiches, or tsunamis, not riverine flooding. During the base flood conditions, the potential for breaking wave height shall be greater than or equal to 1.5 ft. The inland limit of the coastal A zone is (a) the Limit of Moderate Wave Action if delineated on a FIRM, or (b) designated by the authority having jurisdiction.

LIMIT OF MODERATE WAVE ACTION. Line that may be shown on FIRMs to indicate the inland limit of the 1.5-foot wave height during the base flood.

Revise as follows:

1403.7 Flood resistance for high-velocity wave action areas and coastal A zones. For buildings in flood hazard areas subject to high-velocity wave action and coastal A zones as established in Section 1612.3, electrical, mechanical and plumbing system components shall not be mounted on or penetrate through exterior walls that are designed to break away under flood loads.

Revise as follows:

1603.1.7 Flood design data. For buildings located in whole or in part in *flood hazard areas* as established in Section 1612.3, the documentation pertaining to design, if required in Section 1612.5, shall be included and the following information, referenced to the datum on the community's Flood Insurance Rate Map (FIRM), shall be shown, regardless of whether flood loads govern the design of the building:

1. In *flood hazard areas* not subject to high-velocity wave action or coastal A zones, the elevation of the proposed lowest floor, including the basement.
2. In *flood hazard areas* not subject to high-velocity wave action or coastal A zones, the elevation to which any nonresidential building will be dry flood proofed.
3. In *flood hazard areas* subject to high-velocity wave action or coastal A zones, the proposed elevation of the bottom of the lowest horizontal structural member of the lowest floor, including the basement.

1612.4 Design and construction. The design and construction of buildings and structures located in *flood hazard areas*, including flood hazard areas subject to high-velocity wave action and coastal A zones, shall be in accordance with Chapter 5 of ASCE 7 and with ASCE 24.

1612.5 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* not subject to high-velocity wave action 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.
   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 2.6.2.1 of ASCE 24, c*onstruction documents* shall include a statement that the design will provide for equalization of hydrostatic flood forces in accordance with Section 2.6.2.2 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.
2. For construction in flood hazard areas subject to high-velocity wave action 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.
   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/m^2) determined using allowable stress design, *construction documents* shall include a statement that the breakaway wall is designed in accordance with ASCE 24.

Revise as follows:

G103.7 Alterations in coastal areas. Prior to issuing a permit for any alteration of sand dunes and mangrove stands in flood hazard areas subject to high velocity wave action and coastal A zones, the *building official* shall require submission of an engineering analysis which demonstrates that the proposed alteration will not increase the potential for flood damage.

G301.2 Subdivision requirements. The following requirements shall apply in the case of any proposed subdivision, including proposals for manufactured home parks and subdivisions, any portion of which lies within a flood hazard area:

1. The flood hazard area, including floodways, ~~and~~ areas subject to high velocity wave action, and coastal A zones, as appropriate, shall be delineated on tentative and final subdivision plats;
2. Design flood elevations shall be shown on tentative and final subdivision plats;
3. Residential building lots shall be provided with adequate buildable area outside the floodway; and
4. The design criteria for utilities and facilities set forth in this appendix and appropriate *International Codes* shall be met.

G401.2 Flood hazard areas subject to high-velocity wave action and coastal A zones. In *flood hazard areas* subject to high-velocity wave action and coastal A zones:

1. New buildings and buildings that are substantially improved shall only be authorized landward of the reach of mean high tide.
2. The use of fill for structural support of buildings is prohibited.

[B] 309.3 Flood hazard areas subject to high-velocity wave action and coastal A zones. Structures located in flood hazard areas subject to high-velocity wave action and coastal A zones shall meet the requirements of Section 309.2. The plumbing systems, pipes and fixtures shall not be mounted on or penetrate through walls intended to break away under flood loads.

[B] 301.16.1 High-velocity wave action and coastal A zones. In flood hazard areas subject to high-velocity wave action and coastal A zones, mechanical systems and *equipment* shall not be mounted on or penetrate walls intended to break away under flood loads.

Reason: The IBC achieves compliance with the NFIP in Sec. 1612, by reference to ASCE 24 for the specific design and construction requirements. This proposal is to insert the term "coastal A zone" wherever the term "flood hazard area subject to high velocity wave action" appears, to be consistent with ASCE 24. Because of the way the term is defined, only if the Limit of Moderate Wave Action is delineated (or otherwise designated by the AHJ), is the area to be regulated as coastal A zone. ASCE 24-05 has provisions that apply in all Coastal High Hazard Areas (Zone V) and coastal A zones, essentially treating them the same (there are some slight differences because coastal A zones are shown as "Zone A" on Flood Insurance Rate Maps). When 1612.4 refers the user to ASCE 24, one of the first determinations is which flood hazard zone affects the building site. Currently, ASCE 24-05 requires the designer to determine whether conditions landward of Zone V meet the characteristics necessary for coastal A zone conditions. The proposed definition is consistent with the next edition of ASCE 24 that will specify that only if the Limit of Moderate Wave Action (LiMWA) is delineated on the FIRM (or otherwise designated by the AHJ) will the requirements for CAZ apply. FEMA uses the LiMWA to delineate the inland extend of CAZ.

A separate proposal was submitted to change the term "flood hazard area subject to high velocity wave action" to be "coastal high hazard area," which is the term used in the IRC and ASCE 24.

ASCE began the process of updating ASCE 24-05 in early 2011 and the next edition is expected to be published late 2012 or early 2013. The ASCE committee expects to have the near-final draft prepared and available at least a month before the Group A hearings and copies will be provided to the ICC committee.

Cost Impact: Costs will be lower because the RDP and the building official will not have to made independent determinations as to whether a site landward of a Zone V does or does not have coastal A zone conditions. For areas that are subject to coastal A zone conditions there is no change in construction costs because ASCE 24 already has specifications based on whether a building site is or is not subject to coastal A zone conditions.

1403.7-S-INGARGIOLA-WILSON.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The proposed definitions included questionable code wording. The committee felt it was difficult to approve language for consistency with the next edition of ASCE 24 when that standard update was not available to the committee.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

John Ingargiola, Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency and Rebecca C. Quinn, R CQuinn Consulting, Inc., representing Department of Homeland Security, Federal Emergency Management Agency, request Approval as Modified by this Public Commnet.

Modify the proposal as follows:

SECTION 202
DEFINITIONS

COASTAL A ZONE. Area within a special flood hazard area, landward of a V zone or landward of an open coast without mapped ~~V Zones~~ *coastal high hazard areas*. In a coastal A zone, the principal source of flooding must be astronomical tides, storm surges, seiches, or tsunamis, not riverine flooding. During the base flood conditions, the potential for breaking wave height shall be greater than or equal to 1.5 ft. The inland limit of the coastal A zone is (a) the Limit of Moderate Wave Action if delineated on a FIRM, or (b) designated by the authority having jurisdiction.

LIMIT OF MODERATE WAVE ACTION. Line ~~that may be~~ shown on FIRMs to indicate the inland limit of the 1.5-foot breaking wave height during the base flood.

(Portions of proposal not shown remain unchanged)

Commenter's Reason: The committee indicated support for the proposal to define the Coastal A Zone not just by the presence of specific wave conditions, but whether the Limit of Moderate Wave Action has been delineated, or the coastal A zone is otherwise designated by the AHJ. This change mirrors the change to the revised ASCE 24 that's nearing completion. Currently, ASCE 24-05 requires designers to determine if moderate wave conditions are present, without reference to a source of that information. The committee commented on “questionable” wording that was in the proposed definitions (appearing permissive); that wording is proposed to be removed – and the same deletions were included in the third ballot for ASCE 24. The committee also commented that the term “V Zone” should be replaced with the “coastal high hazard area,” which is now defined and used in the IBC.

NOTE: The original S102-12 proposal modified everywhere the term “flood hazard areas subject to high velocity wave action” appears to add “and coastal A zones” in the following sections: 1403.7, 1603.1.7, 1612.4, 1612.5, G103.7, G301.2, G401.2, P309.3 and M301.16.1. Code change S103-12 was Approved as Submitted to replace the phrase “flood hazard areas subject to high velocity wave action” with “coastal high hazard areas.”

S102-12

Final Action: AS AM AMPC____ D

S107-12
1613.1

Proposed Change as Submitted

Proponent: James Bela, Oregon Earthquake Awareness, representing self

Revise as follows:

1613.1 Scope. Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions ~~in accordance with ASCE 7,~~ excluding Chapter 14 and Appendix 11A. The *seismic design category* for a structure ~~is permitted to~~ shall be determined in accordance with Section 1613 ~~or ASCE 7~~.

Exceptions:

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

Reason: (1) ASCE 7 adopted the NEHRP Provisions (developed at the public's expense) as its "standard, then proceeded to charge the engineering community (and the public) for its "commandeering" of those Provisions as its standard.

(a) NEHRP Provisions previously have been adopted into model building codes, as in the Southern Building Code, with no problems (and, particularly, with no "added expense."
ASCE 7 carries a "disclaimer" for its use.

(2) ASCE 7 contains no "references" to justify its legitimacy.
(3) ASCE 7 was the instigator of so-called:) RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) ~~(MCE)~~ GROUND MOTION RESPONSE ACCELERATIONS FOR 0.2- and 1SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B.

(a) this is based on fatally flawed "applied mathematics" assumed in probabilistic seismic hazard assessment, or psha: see discussions under Code Change: FIGURES 1613.3.1 (1)(2)(3)(4)(5)(6)

(4) ASCE 7 is "codifying everything," and is becoming a de-facto code. Code provisions need to remain in a public consensus arena; their "disclaimer" perhaps absolves them from the problems they are creating – but they are creating "unintended consequences" for professional practice.
(5) ASCE 7 is full of errata, which casts substantial questions about the quality of effort and rigor that is going into its formulation.

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

1613.1-S-BELA.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: By deleting the reference to ASCE 7 in Section 1613.1, this proposal would remove all seismic provisions from the code without a replacement. The ASCE 7 provisions which are maintained through a consensus process are preferable.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

James Bela, Oregon Earthquake Awareness, representing self, requests Approval as Submitted.

Commenter's Reason: The Committee Action for Disapproval incorrectly asserts that deleting the reference to ASCE 7 in Section 1613.1 "would remove all seismic provisions from the code without a replacement." This is not the case, as the present seismic provisions could simply be transferred back into the body of the IBC Structural Code, where they rightfully belong (and where they historically have always been).

This is also correctly an ICC Staff function, which always has (and must have) a current and working knowledge of what is actually in both the approved building code (and also that code's referenced standards). To require this level of effort on the part of proponents would provide an insurmountable barrier to addressing (at the fundamental conceptual level) truly important public safety issues with regard to seismic design provisions. And therefore this is, in fact, an appropriate use of the Code Change submittal process; and it is the first step in returning the seismic design provisions of the IBC Structural to their appropriate docket location and format, where scrutiny and future development changes can be more clearly stated, tracked, implemented and finally enforced.

I believe the ASCE 7 so-called "consensus process" is very questionable at best, because: (a) too much of it is conducted in secret; (b) too much of it is made difficult to access or follow on the ASCE web site by interested parties and the public; and (c) the credentials, knowledge base, and biases of those participating in the ASCE 7 process are clouded and opaque; and finally (d) this process disclaims any accountability or responsibility for the use of this (unfortunately, errata-riddled) document.

So, to protect public safety . . .

"PAY NO ATTENTION TO THAT MAN BEHIND THE CURTAIN!"

http://www.youtube.com/watch?v=YWyCCJ6B2WE

S107-12

Final Action: AS AM AMPC____ D

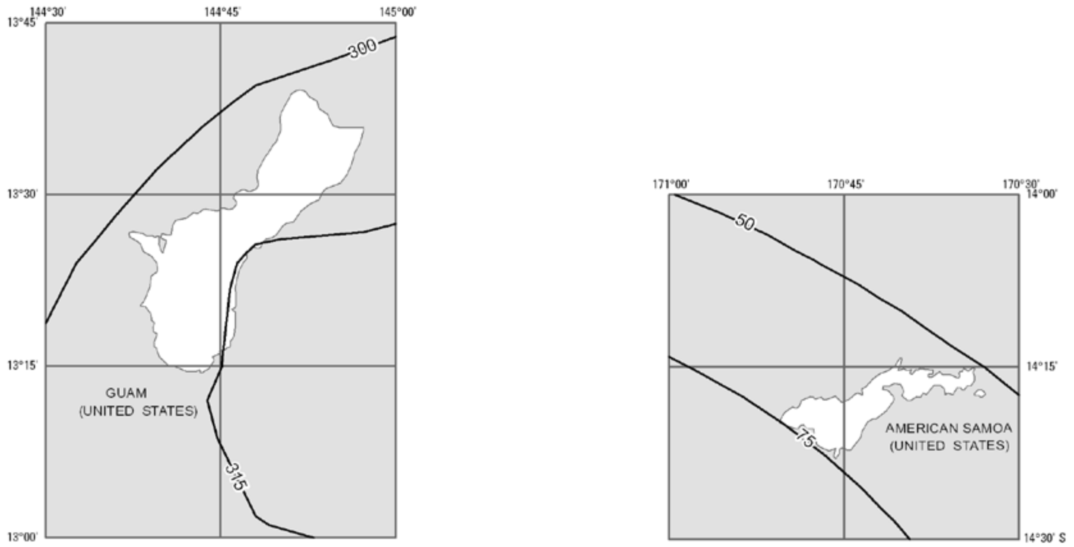
S109-12
1613.3.1

Proposed Change as Submitted

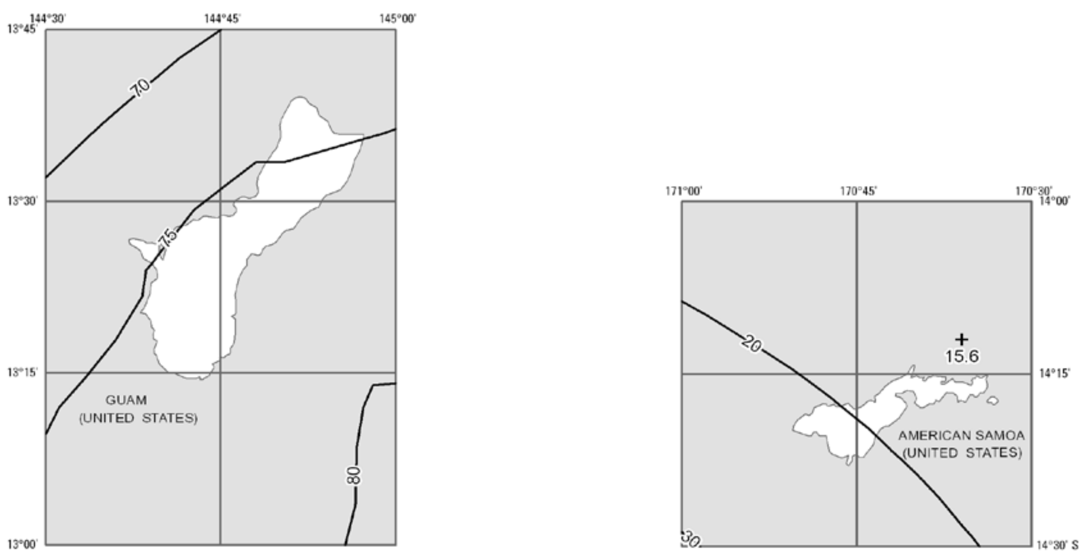
Proponent: Nicolas Luco, US Geological Survey (USGS), representing National Earthquake Hazards Reduction Program (nluco@usgs.gov), Michael Mahoney, Federal Emergency Management Agency (FEMA), representing National Earthquake Hazards Reduction Program

Revise as follows:

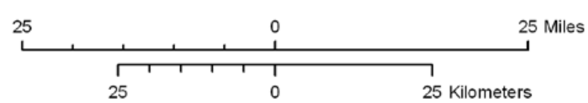
1613.3.1 Mapped acceleration parameters. The parameters S_S and S_1 shall be determined from the 0.2 and 1-second spectral response accelerations shown on Figures 1613.3.1(1) through 1613.3.1(~~6~~7) Where S_1 is less than or equal to 0.04 and S_S is less than or equal to 0.15, the structure is permitted to be assigned *Seismic Design Category* A~~. The parameters Ss and S1 shall be, respectively, 1.5 and 0.6 for Guam and 1.0 and 0.4 for American Samoa.~~



0.2-Second Spectral Response Acceleration (5% of Critical Damping)



1.0-Second Spectral Response Acceleration (5% of Critical Damping)



Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Response Accelerations for Guam and American Samoa of 0.2-Second Spectral Response Acceleration (5% of Critical Damping), Site Class B

FIGURE 1613.3.1(7) RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS FOR GUAM AND AMERICAN SAMOA OF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

Reason: The US Geological Survey (USGS) has the responsibility under the National Earthquake Hazards Reduction Program to develop and maintain seismic hazard maps that are the basis of the Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion maps in the nation's model building codes. As part of that responsibility, the USGS recently developed seismic hazard and MCE_R ground motion maps for Guam and American Samoa, using the same methodology as for the conterminous US, Hawaii, Alaska, and Puerto Rico and the US Virgin Islands. The MCE_R ground motion maps developed are being proposed as an addition to the existing maps in Figure 1613.3.1.

Cost Impact: The code change proposal will increase or decrease the cost of construction, depending on the geographic location.

1613.3.1-S-LUCO-MAHONEY.doc

Public Hearing Results

Committee Action: Disapproved

Committee Reason: The committee supports the addition of the ground motion maps for Guam and American Samoa. Their disapproval is in accordance with the proponent testimony that the maps still need work.

Assembly Action: None

Individual Consideration Agenda

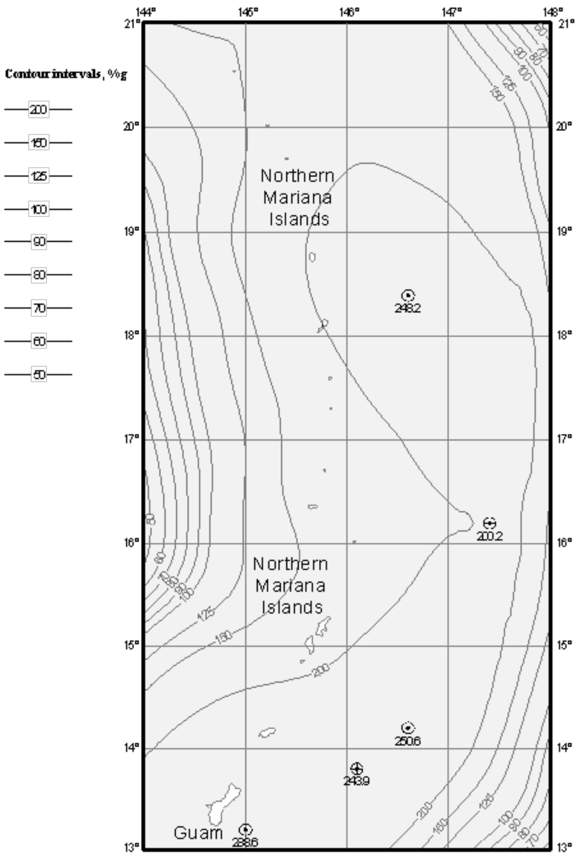
This item is on the agenda for individual consideration because a public comment was submitted.

Public Comment:

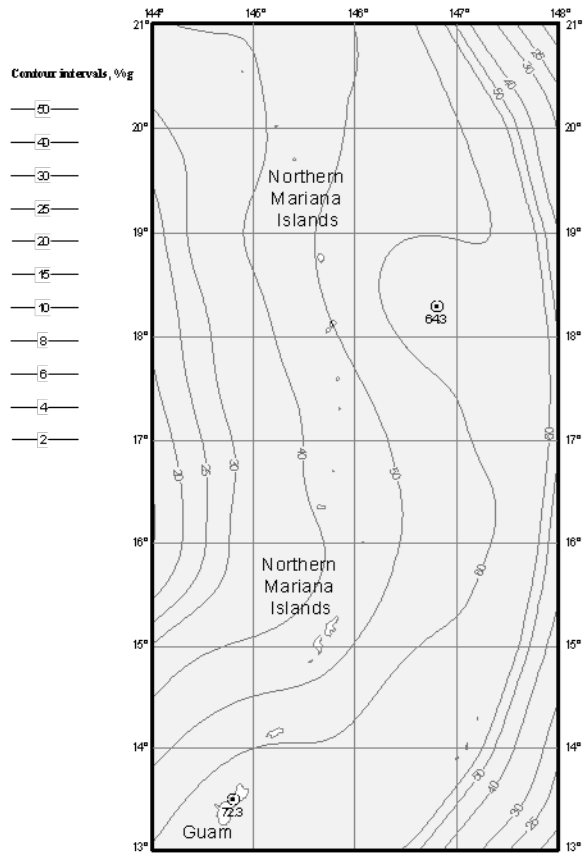
Nicolas Luco, U.S. Geological Survey (USGS), representing National Earthquake Hazards Reduction Program (NEHRP) and Michael Mahoney, Federal Emergency Management Agency, representing National Earthquake Hazards Reduction Program (NEHRP), request Approval as Modified by this Public Comment.

Replace the proposal as follows:

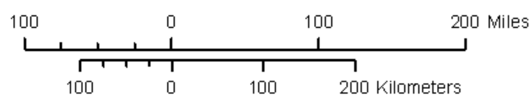
1613.3.1 Mapped acceleration parameters. The parameters S_S and S_1 shall be determined from the 0.2 and 1-second spectral response accelerations shown on Figures 1613.3.1(1) through 1613.3.1(~~7~~ 8) Where S_1 is less than or equal to 0.04 and S_S is less than or equal to 0.15, the structure is permitted to be assigned *Seismic Design Category* A.~~ The parameters Ss and S1 shall be, respectively, 1.5 and 0.6 for Guam and 1.0 and 0.4 for American Samoa.~~



0.2 Second Spectral Response Acceleration
(5% of Critical Damping)



1.0 Second Spectral Response Acceleration
(5% of Critical Damping)



Explanation

Contours of spectral response acceleration expressed as a percent of gravity.

—10—

—10—

Point values of spectral response acceleration expressed as a percent of gravity.

200.2 Local minimum

250.6 Local maximum

243.9 Saddle point

DISCUSSION

Maps prepared by United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency (FEMA)-funded Building Seismic Safety Council (BSSC). The basis is explained in commentary prepared by BSSC and in the references.

Ground motion values contoured on these maps incorporate:

- a target risk of structural collapse equal to 1% in 50 years based upon a generic structural fragility
- a factor of 1.1 and 1.3 for 0.2 and 1.0 sec, respectively, to adjust from a geometric mean to the maximum response regardless of direction
- deterministic upper limits imposed near large, active faults, which are taken as 1.8 times the estimated median response to the characteristic earthquake for the fault (1.8 is used to represent the 84th percentile response), but not less than 150% and 60% g for 0.2 and 1.0 sec, respectively.

As such, the values are different from those on the uniform-hazard 2012 USGS National Seismic Hazard Maps for Guam and the Northern Mariana Islands posted at http://earthquake.usgs.gov/hazmaps.

Larger, more detailed versions of these maps are not provided because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/designmaps) be used to determine the mapped value for a specified location.

REFERENCES

Building Seismic Safety Council, 2009, NEHRP Recommended Seismic Provisions for New Buildings and Other Structures: FEMA P-750/2009 Edition, Federal Emergency Management Agency, Washington, DC.

Huang, Yin-Nan, Whittaker, A.S., and Luco, Nicolas, 2008, Maximum spectral demands in the near-fault region, Earthquake Spectra, Volume 24, Issue 1, pp. 319-341.

Luco, Nicolas, Ellingwood, B.R., Hamburger, R.O., Hooper, J.D., Kimball, J.K., and Kircher, C.A., 2007, Risk-Targeted versus Current Seismic Design Maps for the Conterminous United States, Structural Engineers Association of California 2007 Convention Proceedings, pp. 163-175.

Mueller, C.S., Haller, K.M., Luco, Nicolas, Petersen, M.D., and Frankel, A.D., 2012, Seismic Hazard Assessment for Guam and the Northern Mariana Islands: U.S. Geological Survey Open-File Report 2012–1015.

Figure 1613.3.1(7) Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Response Accelerations for Guam and the Northern Mariana Islands of 0.2- and 1-Second Spectral Response Acceleration (5% of Critical Damping), Site Class B

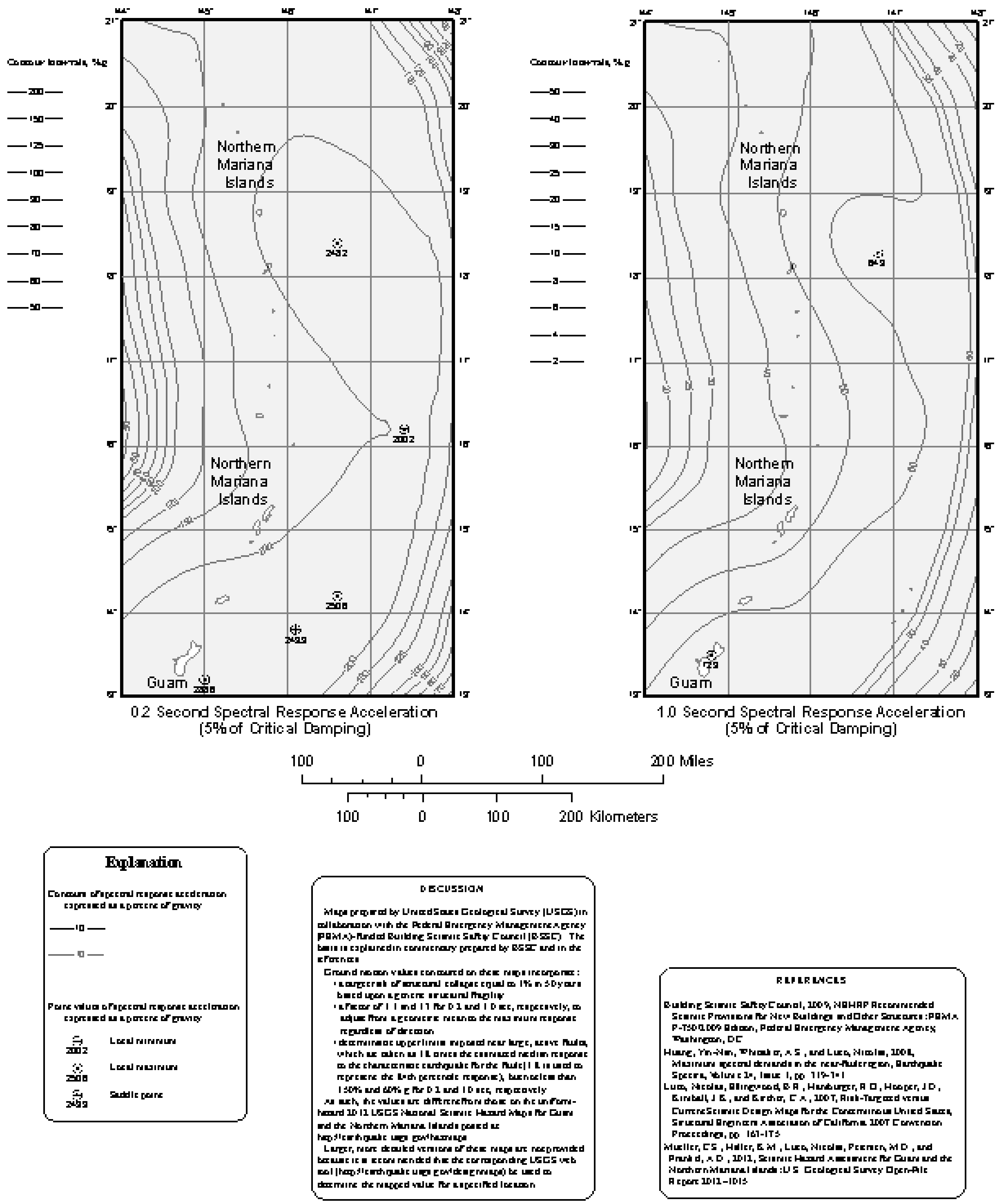


Figure 1613.3.1(7) Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Response Accelerations for Guam and the Northern Mariana Islands of 0.2- and 1-Second Spectral Response Acceleration (5% of Critical Damping), Site Class B

<u>**FIGURE 1613.3.1(7) Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Response Accelerations for Guam and the Northern Mariana Islands 0f 0.2- and 1-Second Spectral Response Acceleration (5% of Critical Damping), Site Class B**</u>

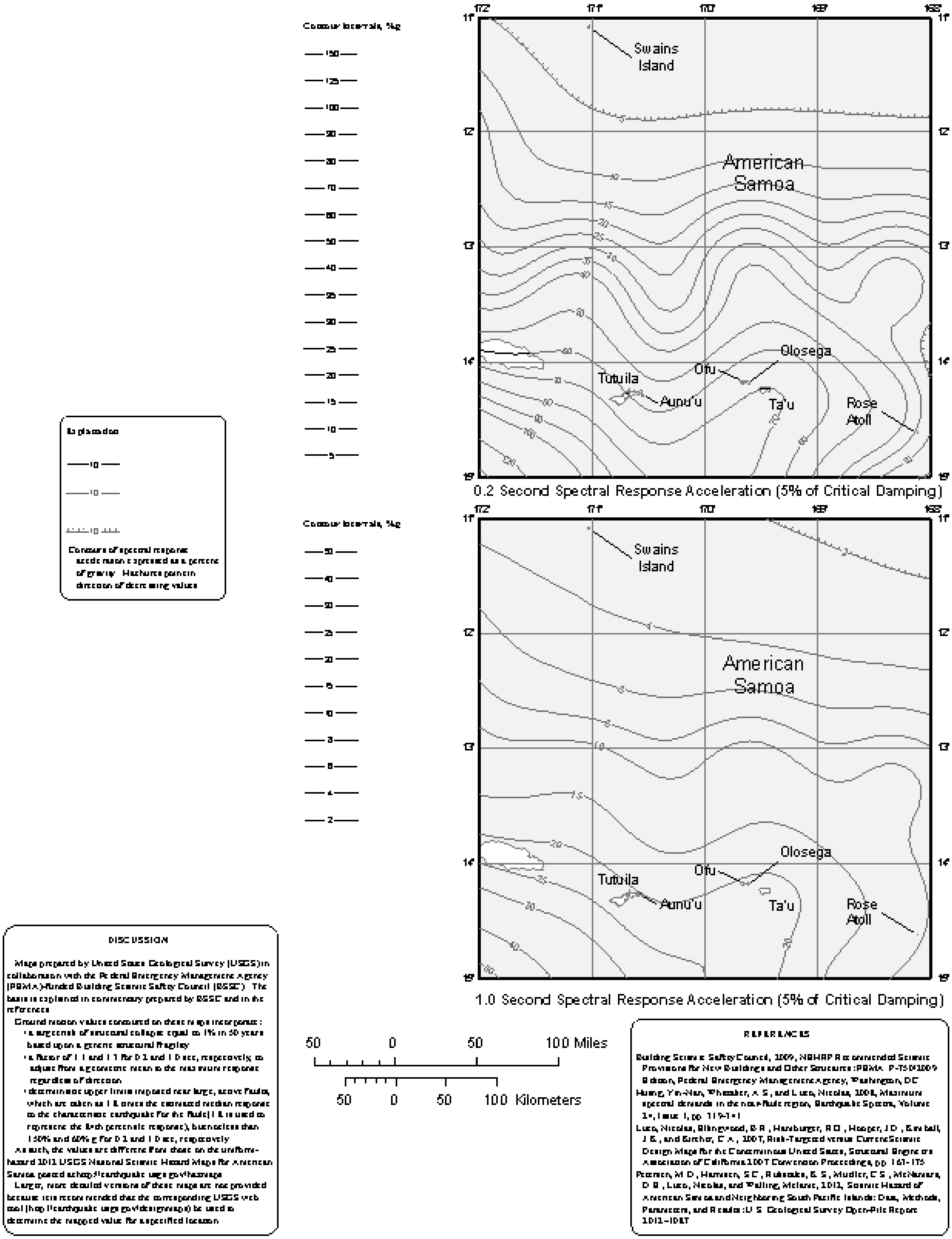


Figure 1613.3.1(8) Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Response Accelerations for American Samoa of 0.2- and 1-Second Spectral Response Acceleration (5% of Critical Damping), Site Class B

<u>**Figure 1613.3.1(8) Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Response Accelerations for American Samoa of 0.2- and 1-Second Spectral Response Acceleration (5% of Critical Damping), Site Class B**</u>

Commenter's Reason: The 2012 ICC Public Hearing Results explain that "the [code development] committee supports the addition of ground motion maps for Guam and American Samoa." As we testified at the hearing, however, at that time the proposed maps still needed work. Since then, the USGS has finalized the maps, via further internal and external review, including a public review workshop. Now, in this public comment, we provide the final maps. With respect to the previously proposed maps, the final values herein are roughly 10% smaller for Guam and 0-15% larger for American Samoa, reflecting relatively minor changes. Before the Final Action Hearing (more specifically, by October 10, 2012), these final maps (which now include the Northern Mariana Islands with Guam) will also have been balloted by the Building Seismic Safety Council (BSSC) Provisions Update Committee.

As stated in the proposal, "the US Geological Survey (USGS) has the responsibility under the National Earthquake Hazards Reduction Program to develop and maintain seismic hazard maps that are the basis of the Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion maps in the nation's model building codes. As part of that responsibility, the USGS has recently developed seismic hazard and MCE_R ground motion maps for Guam and American Samoa, using the same methodology as for the conterminous US, Hawaii, Alaska, and Puerto Rico and the US Virgin Islands. The MCE_R ground motion maps developed are being proposed as an addition to the existing maps in Figure 1613.3.1."

In comparing the proposed MCE_R ground motion maps (as modified herein) to the geographically-constant ground motion values stipulated in the 2012 IBC, it is important to bear in mind that the latter values are not based on seismic hazard analyses. According to the commentary of the 1997 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA 303), the values in the 2012 IBC are merely conversions, via rough approximations, from values on the 1994 *NEHRP Recommended Provisions* maps that had been in use for nearly 20 years. As such, they do not take into account the 1993 Guam earthquake that was the largest ever recorded in the region and caused considerable damage, the 2009 earthquake near American Samoa that caused a tsunami, nor the 2008 "Next Generation Attenuation (NGA)" and another 2006 empirical ground motion prediction equations that can be used for both Guam and American Samoa. This and other such information is directly used in the seismic hazard analyses that are the basis for the proposed MCE_R ground motion maps, as documented in the USGS Open-File Reports referenced on the maps. This same type of information is already the basis for the MCE_R ground motions maps for the conterminous US, Hawaii, Alaska, and Puerto Rico and the US Virgin Islands that are in the 2012 IBC.

S109-12

Final Action: AS AM AMPC____ D

S110-12

Figures 1613.3.1(1) (New), 1613.3.1(2) (New), 1613.3.1(3) (New), 1613.3.1(4) (New), 1613.3.1(5) (New), 1613.3.1(6) (New)

Proposed Change as Submitted

Proponent: James Bela, Oregon Earthquake Awareness, representing self

Delete and substitute as follows:

~~FIGURE 1613.3.1(1)~~
~~RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCER) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B~~

~~FIGURE 1613.3.1(2)~~
~~RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCER) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B~~

~~FIGURE 1613.3.1(3)~~
~~RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCER) GROUND MOTION RESPONSE ACCELERATIONS FOR HAWAII OF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B~~

~~FIGURE 1613.3.1(4)~~
~~RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCER) GROUND MOTION RESPONSE ACCELERATIONS FOR ALASKA OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS~~

~~FIGURE 1613.3.1(5)~~
~~RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCER) GROUND MOTION RESPONSE ACCELERATIONS FOR ALASKA OF 1.0-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B~~

~~FIGURE 1613.3.1(6)~~
~~RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCER) GROUND MOTION RESPONSE ACCELERATIONS FOR PUERTO RICO AND THE UNITED STATES VIRGIN ISLANDS OF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5%OF CRITICAL DAMPING), SITE CLASS B~~

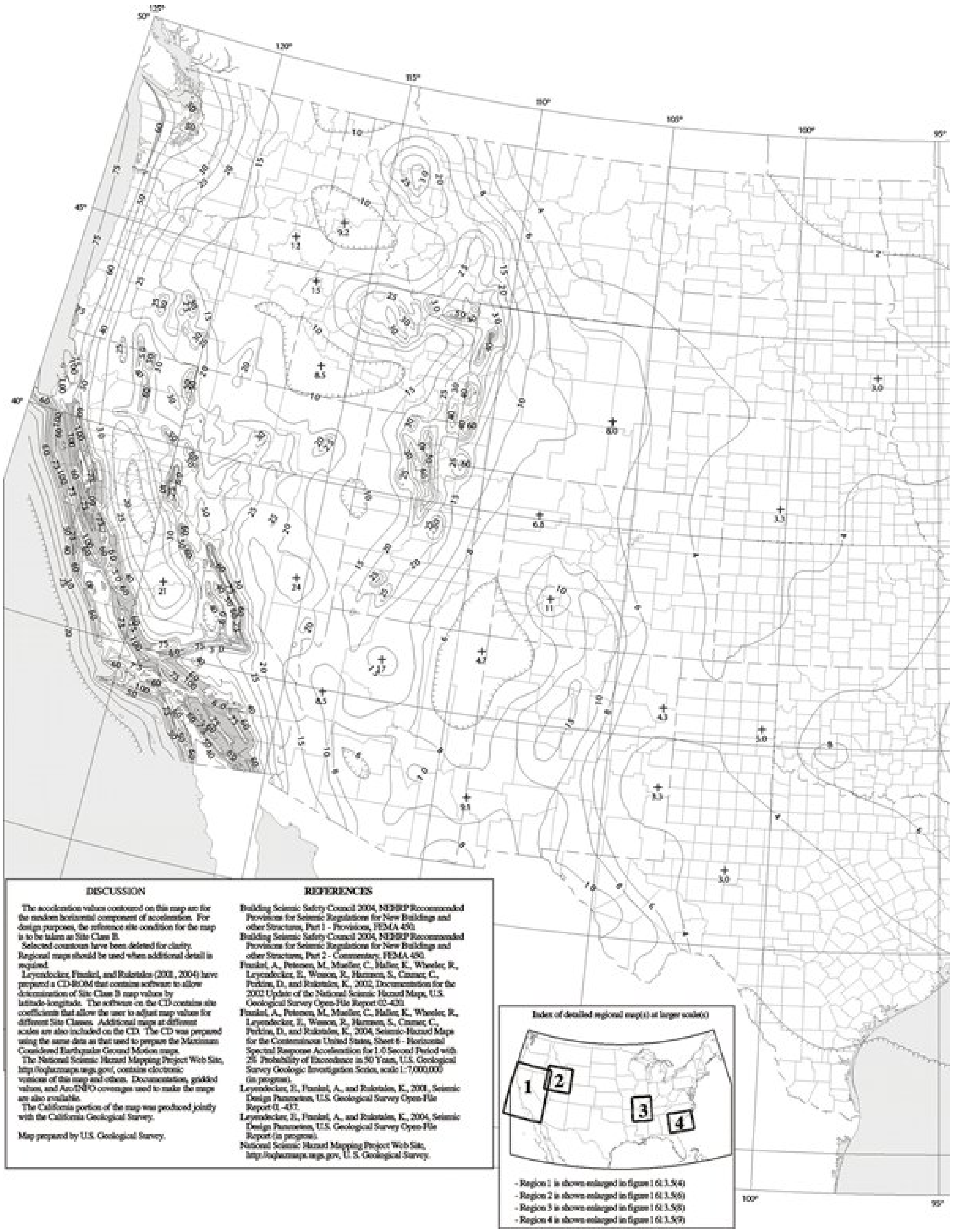


FIGURE 1613.3.1(1)
MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

FIGURE 1613.3.1(1) - continued
MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

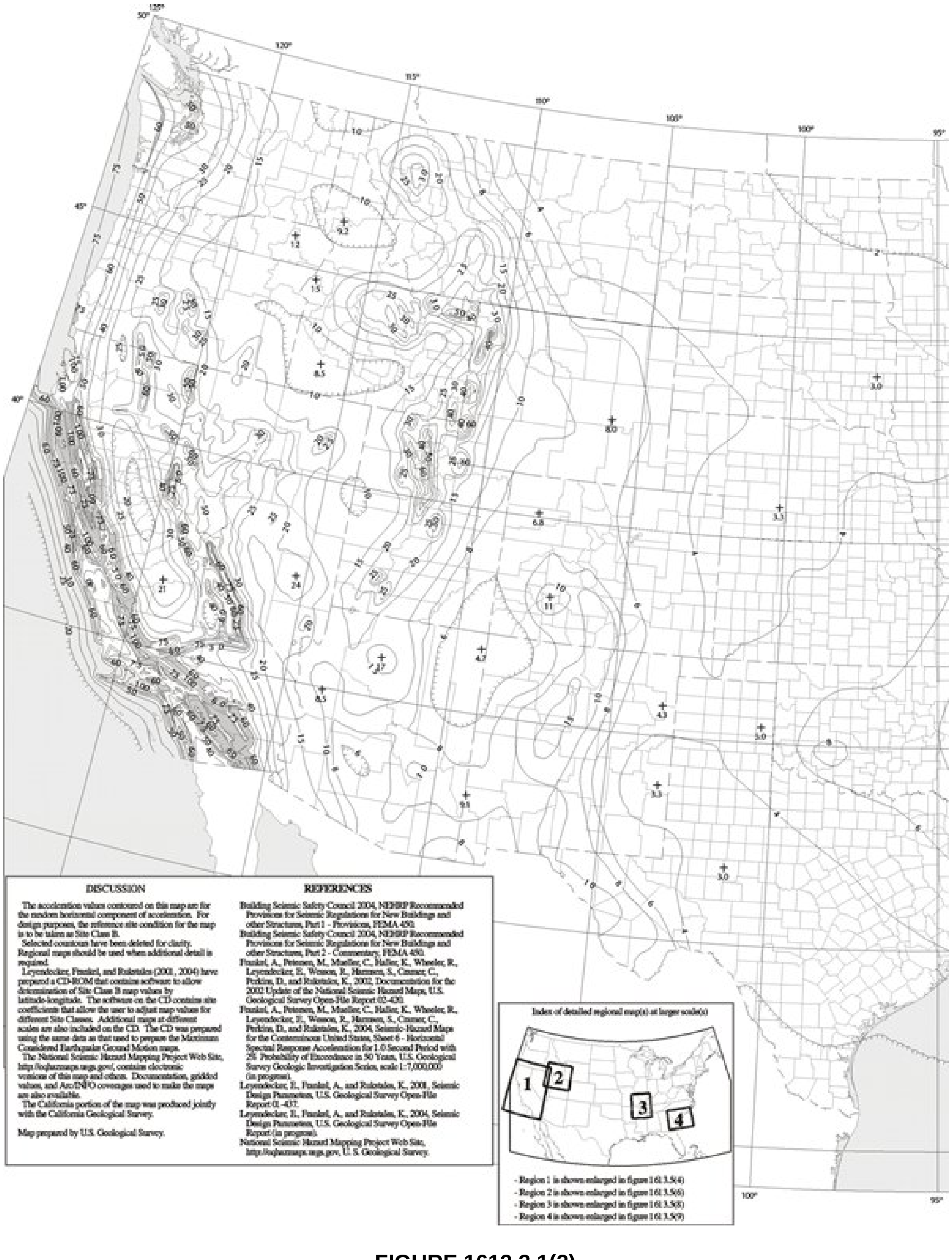


FIGURE 1613.3.1(2)
MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

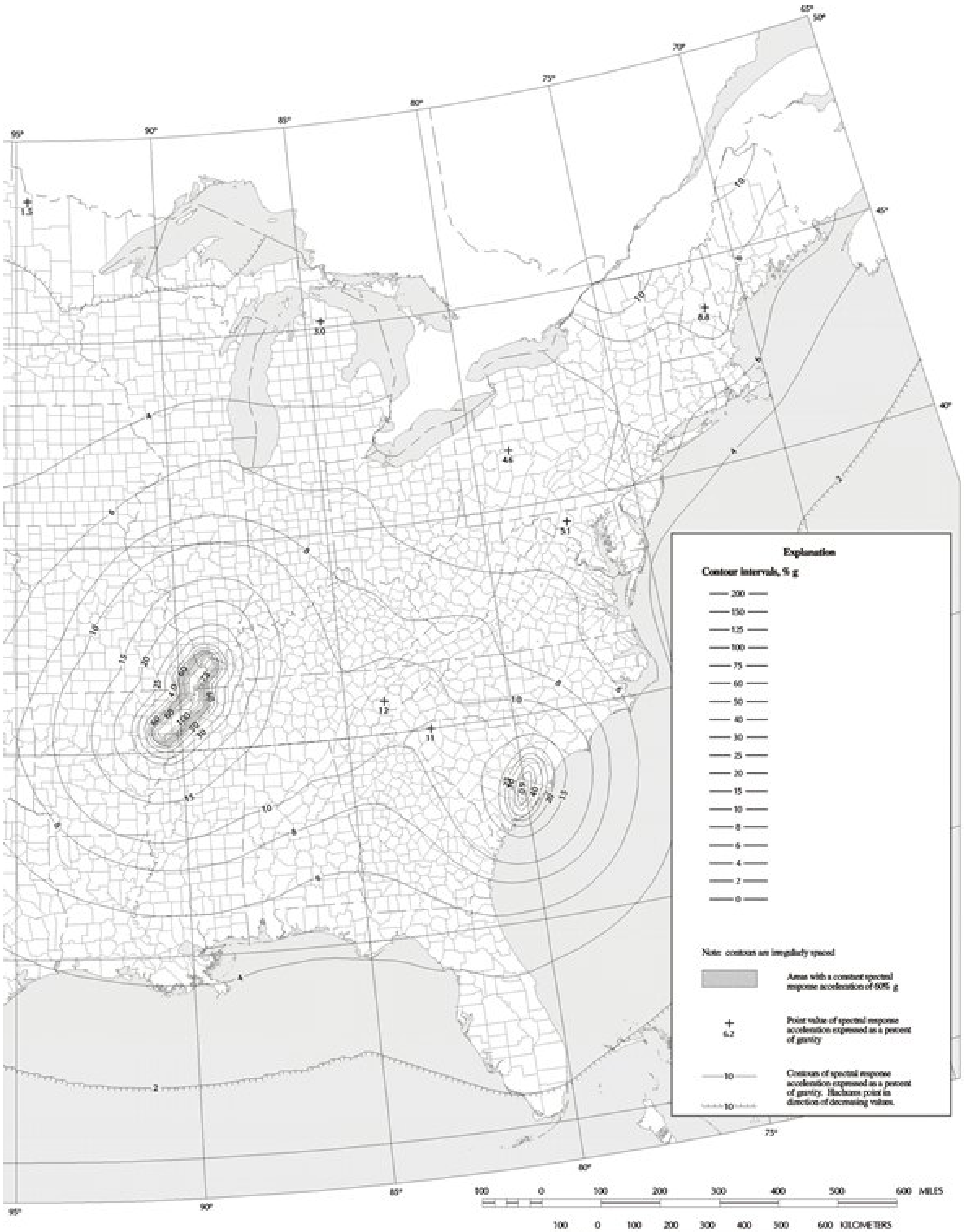
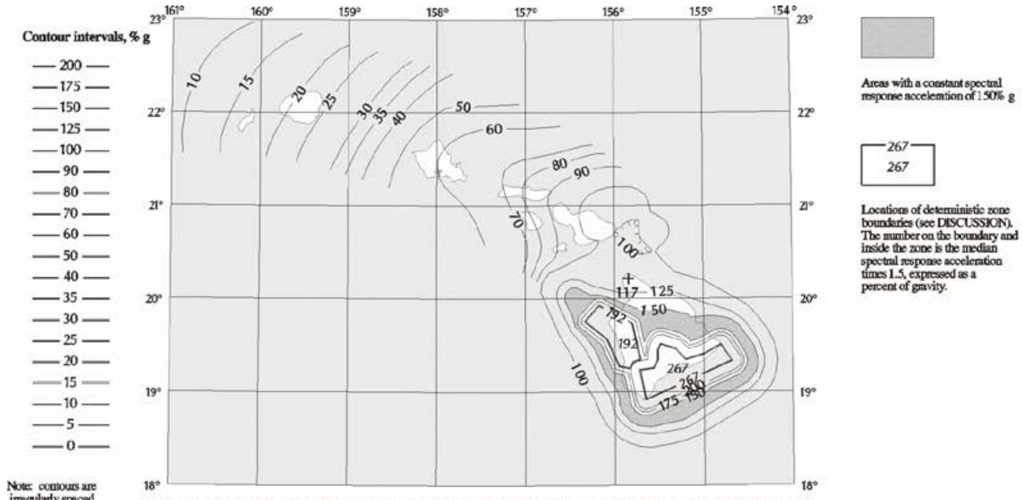
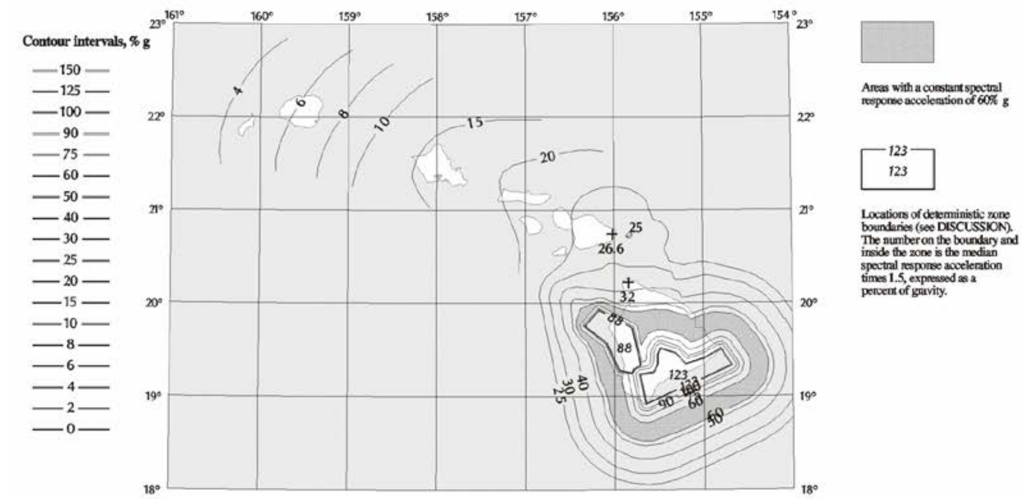


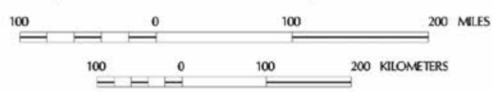
FIGURE 1613.3.1(2) - continued
MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS FOR THE CONTERMINOUS UNITED STATES OF 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING)



1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING)



Explanation

+ 6.2 — Point value of spectral response acceleration expressed as a percent of gravity

10 — Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.

DISCUSSION

The acceleration values contoured on this map are for the random horizontal component of acceleration. For design purposes, the reference site condition for the map is to be taken as Site Class B.

The two areas shown as zone boundaries are the projection to the earth's surface of horizontal rupture planes at 9 km depth. Spectral accelerations are constant within the boundaries of the zones. The number on the boundary and inside the zone is the median spectral response acceleration times 1.5.

Leyendecker, Frankel, and Rukstales (2001, 2004) have prepared a CD-ROM that contains software to allow determination of Site Class B map values by latitude-longitude. The software on the CD contains site coefficients that allow the user to adjust map values for different Site Classes. Additional maps at different scales are also included on the CD. The CD was prepared using the same data as that used to prepare the Maximum Considered Earthquake Ground Motion maps.

The National Seismic Hazard Mapping Project Web Site, http://eqhazmaps.usgs.gov, contains electronic versions of this map and others. Documentation, gridded values, and Arc/INFO coverages used to make the maps are also available.

Map prepared by U.S. Geological Survey.

REFERENCES

Building Seismic Safety Council 2004, NEHRP Recommended Provisions for Seismic Regulations for New Buildings and other Structures, Part 1 - Provisions, FEMA 450.

Building Seismic Safety Council 2004, NEHRP Recommended Provisions for Seismic Regulations for New Buildings and other Structures, Part 2 - Commentary, FEMA 450.

Klein, F., Frankel, A., Mueller, C., Wesson, R. and Okubo, P., 2001, Seismic hazard in Hawaii: high rate of large earthquakes and probabilistic ground-motion maps, Bull. Seism. Soc. Am., v. 91, pp. 479-498.

Klein, F., Frankel, A., Mueller, C., Wesson, R. and Okubo, P., 1998, Seismic-Hazard Maps for Hawaii, Sheet 2 - 2% Probability of Exceedance in 50 Years for Peak Horizontal Acceleration and Horizontal Spectral Response Acceleration for 0.2, 0.3, and 1.0 Second Periods U.S. Geological Survey Geologic Investigation Series I-2724, scale 1:2,000,000.

Leyendecker, E., Frankel, A., and Rukstales, K., 2001, Seismic Design Parameters, U.S. Geological Survey Open-File Report 01-437.

Leyendecker, E., Frankel, A., and Rukstales, K., 2004, Seismic Design Parameters, U.S. Geological Survey Open-File Report (in progress).

National Seismic Hazard Mapping Project Web Site, http://eqhazmaps.usgs.gov, U. S. Geological Survey.

FIGURE 1613.3.1(3)
MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS FOR HAWAII OF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

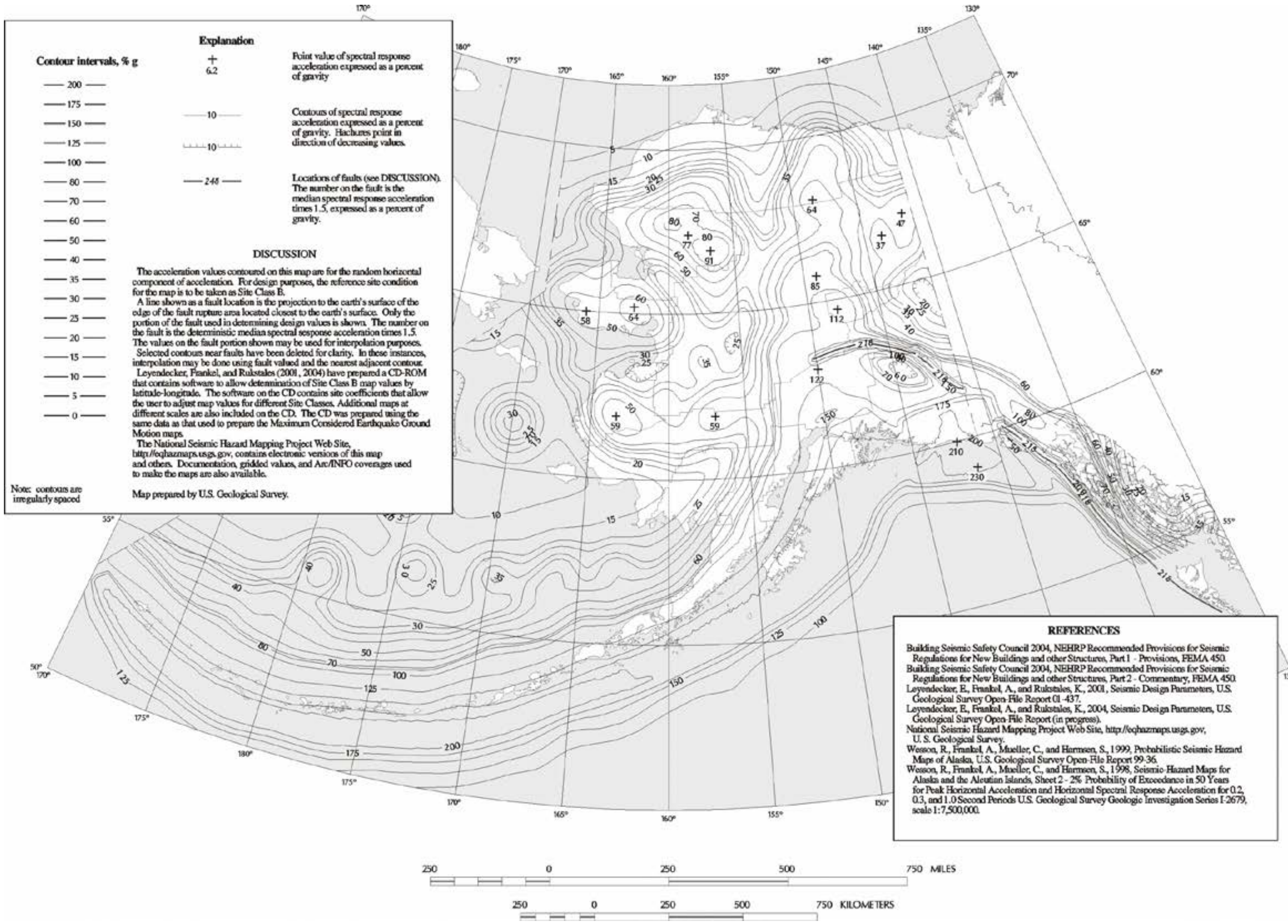


FIGURE 1613.3.1(4)
MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS FOR ALASKA OF 0.2-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

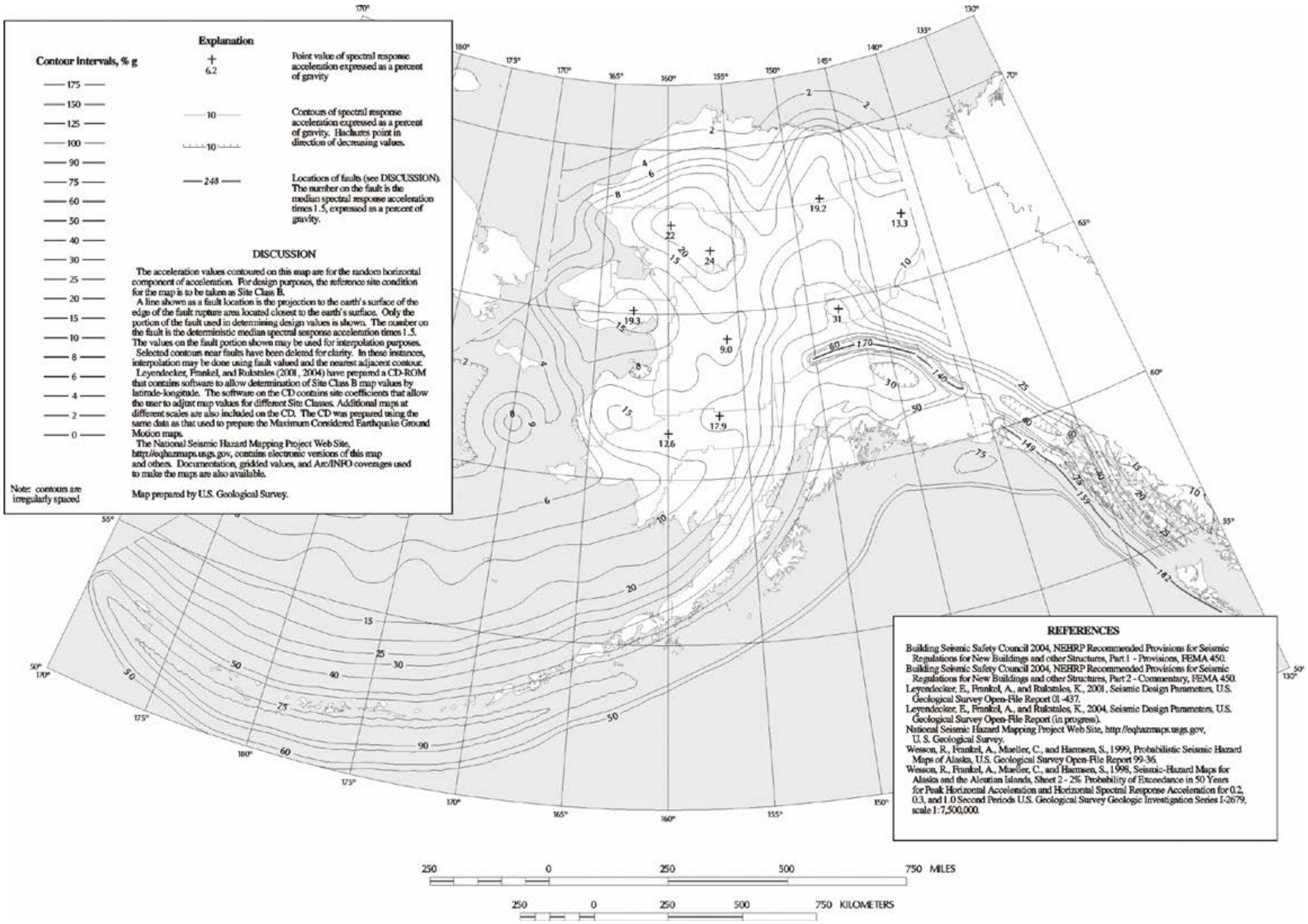
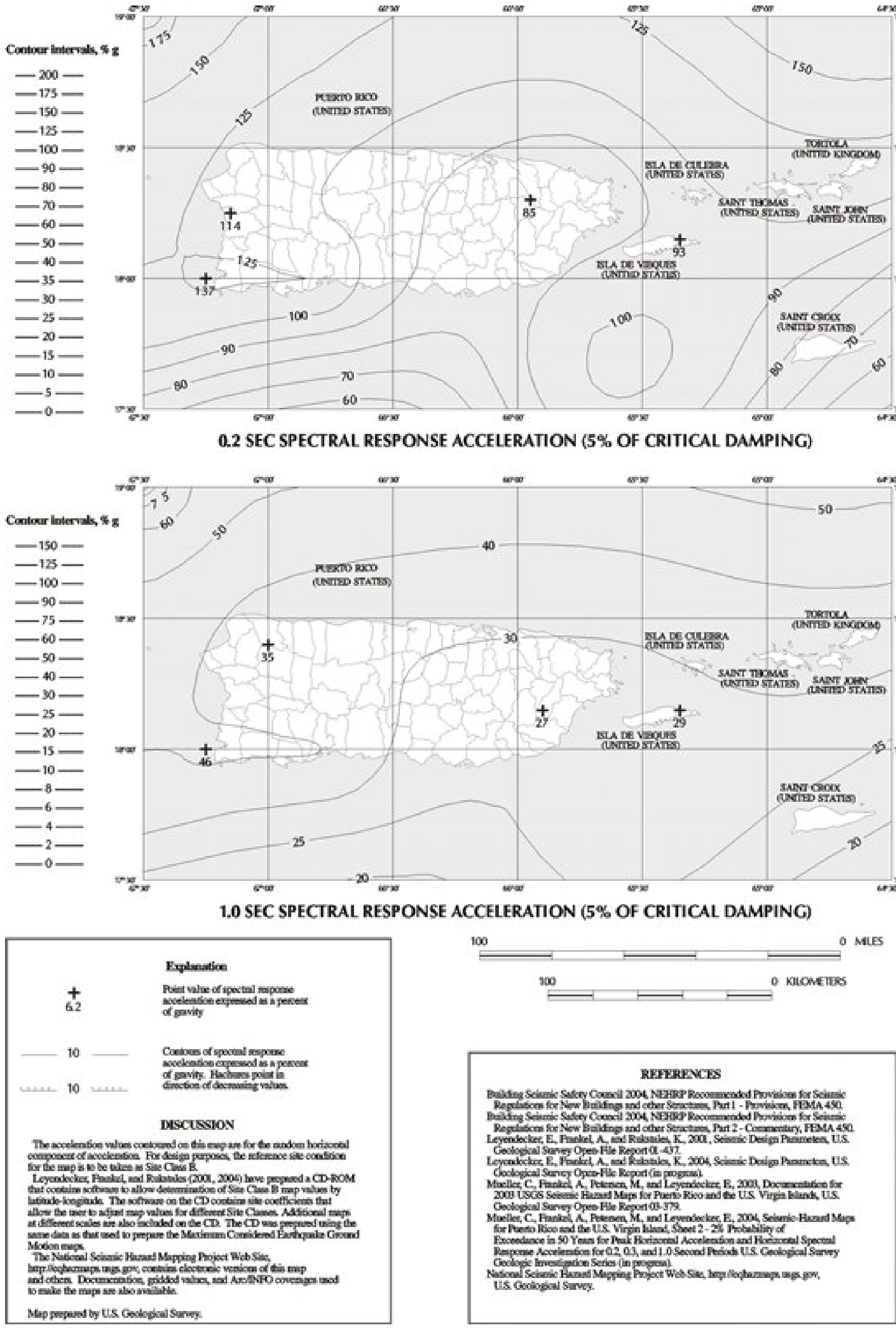


FIGURE 1613.3.1(5)
MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS FOR ALASKA OF 1.0-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



<u>FIGURE 1613.3.1(6)</u>
<u>MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION RESPONSE ACCELERATIONS FOR PUERTO RICO AND THE UNITED STATES VIRGIN ISLANDS OF 0.2- AND 1-SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B</u>

Reason: (1) Constantly changing the USGS National Seismic Hazard Maps' "ground motion response accelerations contours" is **destabilizing** to design practice, plan review requirements, and code enforcement provisions, because such changes are:

(a) creating **yo-yo earthquake design standards** – "high" one code cycle and "low" the next; or vice-versa; making it, as a result,

ever more difficult to develop, practice and apply "professional engineering judgment" in the design process.
(b) creating serious and perplexing problems for addressing seismic hazards for **existing buildings** – which must then **"benchmark"** to a specific year and to a specific version (year & edition) of seismic hazard map (for any specific public policy mandate/requirements for earthquake retrofit/mitigation ordinances or measures. These required "benchmark" seismic hazard maps will then be different (sometimes a lot different) from the current (and ever-changing and ever-evolving) USGS National Seismic Hazard Maps. This is, and will continue to be, a big source of confusion.

(2) RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS contours in the IBC 2012 / ASCE 7-10 are sometimes **30% lower** than previous map values of just a decade ago:

(a) the recent 08-23-2011 M 5.8 Mineral VA (Cuckoo) earthquake had 30% lower design values (with these new maps) than a decade ago – making the earthquake's epicentral region **Seismic Design Category A-B**; yet the **actual intensity of earthquake ground shaking** experienced there was the "stated intensity" that could be expected for the IBC/ASCE 7-10 designation **SDC D!**.(Bela 2011)
(b) when the seismic hazard maps **depict such low hazard ground motion response accelerations** and their corresponding **low** Seismic Design Categories, they both foster and create the "circumstances" for **"comfortable inaction;"** and, unfortunately, this feeling of "comfortable inaction" easily transfers to the arena of public policy.
(c) The condition of "comfortable inaction" (due to perceived low hazard - depicted on the seismic hazard map) was cited as perhaps the main culprit in Christ Church, New Zealand's lack of adequate preparedness during its recent hammering by a "pair" of earthquakes – which killed around 200 people in unsafe **"Killer Buildings."**.
(3) The basic underlying methodology for preparing the USGS National Seismic Hazard Maps (and their derivative so-called Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Response Accelerations contours); i.e., probabilistic seismic hazard assessment (or psha) is fatally **"flawed"** – due to systemic "errors" in the applied mathematics which both create and define it. And it is, unfortunately, these same flawed "mathematics" that are prescribing how these psha-determined ground motion contours are ultimately derived, computed . . . and then finally codified.
(4) Errors in its methodology aside, the basic problems, difficulties and really insurmountable obstacles to performing a psha seismic hazard assessment *(Mualchin, 2010; Bela and Mualchin 2011)* have **never** actually been "solved." And they still remain unsolved! These problems involve data-driven earth-science requirements for a knowledge and understanding of:

(a) fault slip rates;
(b) frequency of occurrence of earthquakes (and their known magnitudes); and
(c) earthquake source mechanisms – specifically, (i) the style of faulting: and (ii) the hypocentral depth (or where exactly the earthquake rupture process begins).

(5) The psha methodology is easily "manipulated," particularly in the sense that: (i) selecting the probabilistic hazard level is a totally **arbitrary** process; and (ii) changing the hazard level (higher hazard or lower hazard) gives a completely different ground motion response acceleration contour – and consequently, then, different code requirements!
(6) These very real and insurmountable problems with psha's methodology have been swept away by its proponents: by convoluted (and mostly unintelligible) efforts and preoccupations with "logic trees," "quantifying uncertainties," etc. These efforts proceed busily ahead; but, meanwhile, they are "neglecting baseline principles" (of "what" the earthquake can do to you – and "how" it can do it – and the maximum Magnitude it could be). All that mathematical busywork, logic-tree accounting, and so-called "expert opinion" built a the "better model" (or -- so the proponents believe). Unfortunately, that "better model" then:

(a) has become **"substituted"** for "reality" by its creators;
(b) has dismissed criticisms of it -- by claiming (itself) to be "best available science;" and
(c) has become ultimately so **"complicated" --** that not even its proponents now can logically and successfully explain how it came to be (Hamburger et. al., 2010; Bela, 2011); nor can they effectively explain how to apply it to the real world of earthquake engineering, public safety, and socioeconomic issues of community resiliency.

(7) The ground motion accelerations, and their probabilities for exceeding them, are combined and co-mingled in such a way that the actual sources (or **earthquake magnitudes**, **frequency content of earthquake ground motions**, and **duration** of strong ground shaking) are treated more-or-less equally—and they are most certainly not!
(8) The **"Maximum Credible Earthquake" (MCE)** or "Maximum Capable Earthquake" or "Maximum Possible Earthquake" (within ¼ unit of Magnitude, M) is never explicitly stated. And it's really "Magnitude, Magnitude, Magnitude!" (and for the same reasons previously stated in (4)) – that has everything to do with building performance (damage and repair costs) and, more importantly, public safety and community resilience.
(9) R-Factors, or Response Modification Factors, that are used in design become **less reliable** in ascertaining/predicting the **"end result"** (or the building's actual performance in an earthquake). And, **"an earthquake"** really needs to explicitly consider the full suite of earthquake possibilities that the regional tectonics forewarn us can occur (including MCE = Maximum Credible Earthquake, or Maximum Possible Earthquake). "R-Factors" have become less reliable primarily because:
(a) quite a lot of the "ductility" or building "toughness" that the code relies upon to: (i) ride out the earthquake (by bending, not breaking, and absorbing energy); and (ii) remain standing (without killing the occupants) -- is due to "over-strength;" and.
(b) when the code design "strength" is systematically diminished (weakened) or reduced (over several-to-many iterations of seismic hazard mapping --by lowering (yo-yo effect) the "numerator" quantity in the design strength

equation; then when dividing this numerator (now smaller number) by the same "large" number (R-Factor in denominator) – we have now "lost" perhaps a good portion of our "over-strength" – that was implicit in selecting the weights of the various R-Factors in the first place!

Basically, with RISK-TARGETED (MCE_R) , the code is now dividing an ever-decreasing and now smaller number (perhaps by 30%) by the same "large" number (R-Factor denominator) -- with the result that the buildings' performances and outcomes are really now much less certain . . . and also now much more problematical.

(10) The psha methodology has been shown in dramatic and tragic fashion to be not only "misleading", but also deadly, in the last decade or so of the "Eleven of the World's Deadliest Earthquakes." (Panza et. al. 2011, Table 1) In example after example, and all across the globe (where now more than 700, 000 people have perished); the psha-methodology "prescribed" seismic hazard: was determined to be either low or very low – but was "disproved" in these many cases by earthquakes that were "surprises" from what psha had determined could be expected. In too many of these deadly "surprises", the actual intensities of ground shaking experienced were greater by factors of 2X to 4X – than what psha had predicted. (Bela 2010; Bela and Mualchin, 2011;

Kossobokov and Nekrasova, 2010;)

It is clear that this is an unsafe situation (to general public) that must **not** continue; but it does continue for some of these following main reasons:

(a) the psha methodology is "anonymous," so when there is clear evidence (> 700,000 casualties) that it is "not working;" no one is accountable for its: (i) external failures (mass casualties); and/or (ii) internal failures (very real errors in its "applied mathematics" derivations).
(b) the psha methodology has a hierarchial and powerful elite behind its influence and continued use.
(c) the psha methodology has a pedigree of high sounding terms (like "quantifying uncertainty," "logic-tree", "expert opinion," "best science," etc.) -- all purporting to increase the method's **"precision."** But the end result, as these Eleven Deadliest Earthquakes" have shown us, is, unfortunately, still too **"inaccurate"** and "too deadly" for protecting the public safety. And in this regard, it is clearly missing its target!

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Panza, G.F.; Irikura, K.; Kouteva-Guentcheva, M.; Peresan, A.; Wang, Z.; Saragoni, R. (Eds.) (2011). Advanced Seismic Hazard Assessment, 320 p.
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Table 1 List of the top eleven deadliest earthquakes occurred during the period 2000-2011 and the corresponding intensity differences (ΔI) among the observed values and those predicted by the Global Seismic Hazard Assessment Program, or GSHAP.

Allesandro Martelli, Paolo Clemente, Massimo Forni, Giuliano F. Panza,
Antonello Salvatori (2011).

RECENT DEVELOPMENT AND APPLICATION OF SEISMIC ISOLATION AND ENERGY DISSIPATION SYSTEMS, IN PARTICULAR IN ITALY, CONDITIONS FOR THEIR CORRECT USE AND RECOMMENTATIONS FOR CODE IMPROVEMENTS, in 12TH WORLD CONFERENCE ON SEISMIC ISOLATION, ENERGY DISSIPATION AND ACTIVE CONTROL OF STRUCTURES Sept. 20-23, 2011 Sochi-city, Russia

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

F1613-S-BELA.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: Retaining the current risk-targeted ground motion maps for seismic design is preferred. The best available technology ought to be used and it would be wrong to ignore what's been developed and vetted for twenty years.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

James Bela, Oregon Earthquake Awareness, representing self, requests Approval as Submitted.

Commenter's Reason: The Committee Action for Disapproval incorrectly: (a) substitutes the committee's so-called "preference" for "retaining the current risk-targeted ground motion maps for seismic design" without stating specific and defensible objections against the Proposed Change S110-12, which simply restored what already previously was "OK" in the IBC Structural Building Code; (b) misunderstands and misconstrues the fact that so-called "current risk-targeted ground motion maps" are, to the contrary, not the best available technology; and (c) most incorrectly asserts that these current risk-targeted ground motion maps have "been developed and vetted for twenty years."

Regarding (c) above, these current risk-targeted ground motion maps for seismic design have only first appeared in the 2012 Edition IBC Structural building code. And so they most assuredly and categorically have in no way whatsoever been "vetted for twenty years." To the contrary, in reality they were first approved by the Structural Code Committee (despite opposing testimony); at a time when they actually had not even been vetted within the ASCE 7 balloting and voting process, much less even published in a printed format (ASCE 7-10).

The best available technology for seismic hazard assessment and its derivative product: seismic design procedures and requirements – is Deterministic Seismic Hazard Assessment (DSHA and neoDSHA or NDSHA); not what historically has been dominant (and later imposed) since the 1977 National Earthquake Hazards Reduction Program (NEHRP). Under NEHRP, the U.S. Geological Survey (one of the four main NEHRP agencies) pursued exclusively an "applied mathematics" model for depicting seismic hazards, known as Probabilistic Seismic Hazard Assessment, or PSHA. In effect, the USGS substituted a "mathematical model" (something which could not be directly observed – the traditional criterion for scientific fact) for "scientific fact."

These new risk-targeted ground motion maps sometimes reduce seismic design requirements by as much as 30% from previous code requirements, notably within western Oregon (with its location and hazard within the Pacific Northwest defined by the potential for giant Magnitude 9 subduction zone earthquakes and tsunamis). This "yo-yoing" of seismic design requirements between adjacent IBC code editions has to stop! And saying NO! to this latest offending and irritating iteration (risk-targeted ground motion maps or MCE_R) is the necessary place to call a halt.

I believe it is true to say: The complexity and convoluted methodologies behind these ever-evolving USGS driven seismic hazard maps have long since exceeded the abilities of (a) code committees to fully comprehend their derivations and usefulness in the seismic design process; (b) of practicing structural engineers to hone and apply judgment in producing better and safer buildings in support of both resilient buildings and resilient regional and community economies; and (c) sadly, even exceeded the abilities of those same individuals who continue to promote and reformat ever more complexity into a flawed seismic hazard model.

The data show that designing for what is "probable" does not protect public safety from what is "possible." The most recently deadly examples of this fact have been Haiti (2010), New Zealand (2010 and 2011) http://www.youtube.com/watch?v=dazS3LhTHo , and Japan (2011). And even the quite recent 23 August 2011 M 5.8 Central Virginia earthquake near Washington D.C. demonstrated the inadequacies and public safety liabilities of depicting seismic hazard using PSHA instead of DSHA methodologies

TAKE ME HOME . . . *SEISMIC LOADS!*

I haven't seen anything regarding Site Class, for Mineral or Louisa VA, as well as the estimated epicentral region of Central Virginia's Piedmont? Cuckoo seems to be the closest built environment to the epicenter (with still an uncertainty: horizontal +/- 2.3 km (1.4 miles); depth +/- 3.1 km (1.9 miles)). No one has officially designated this as the CUCKOO Earthquake. But read below and see if, perhaps, that term might be better reserved for USGS seismic hazard mapping and U.S. Building Code requirements in both the Central Virginia Seismic Zone and in other known and active seismic zones throughout the Central and Eastern U.S. (CEUS)?

Also, the MMI intensity of earthquake ground shaking (VII - VIII at the estimated epicentral location) was more correctly indicative of SDC D. [http://www.nibs.org/client/assets/files/bssc/P749/P-749_Chapter5.pdf]
Since 2000, the USGS Seismic Hazard Maps have continued to lower the hazard [SDS = S_{DS} design earthquake spectral response accelerations:
$\mathbf{S_S}$ = 0.31g (1997) (2000); 0.26g (2003); 0.22g (2009)

SCB: S_{DS} = 0.21g (1997) (2000); 0.17g (2003); 0.15g (2009).
SCC: S_{DS} = 0.25g (1997) (2000); 0.20g (2003); 0.17g (2009).

SCD: S_{DS} = 0.32g (1997) (2000); 0.27g (2003); 0.23g (2009).],
making building code earthquake provisions less safe regarding both public safety and economic well-being.

These numbers translate to about a 30% decline in design strength (from a low number to an even lower number) in just the last decade! (for the S_{DS} "Design Earthquake Spectral Response Acceleration Parameter"). A 33% increase in design strength used to be the difference between Seismic Zone 3 and Seismic Zone 4 requirements!
For Site Class B, this now makes the epicentral region of this M 5.8 Virginia (Cuckoo) earthquake Seismic Design Category A (SDC A) - the same as Florida and Michigan (which have no active seismic zones or geologic evidence of mountain building).
http://www.dmme.virginia.gov/DMR3/Va_5.8_earthquake.shtml

This "minor" earthquake now seems to be amongst the most widely felt earthquakes in U.S. history.
(i.e., "ever!") -- "Felt strongly in much of central Virginia and southern Maryland. Felt throughout the eastern US from central Georgia to central Maine and west to Detroit, Michigan and Chicago, Illinois. Felt in many parts of southeastern Canada from Montreal to Windsor." Source USGS
Clearly we are no longer in Florida, Michigan . . . or even in Kansas any more!
Too many (a) unsafe conditions and (b) brittle-failure-mode susceptible building products are allowed in the low SDC's A, B, and C - and it defies both logic, engineering judgment, common sense, as well as the professional responsibility of our combined professions. I doubt if any of the brick veneer that separated during this M 5.8 Virginia earthquake would have even been required to be adequately attached for earthquake (lateral force) resistance in these SDC's of A,B and C?

Remember: "The buck stops shear!"

West Virginia, Mountain Mama . . . Take Me Home . . . Seismic Loads!" . . . . because
http://www.youtube.com/watch?v=oN86d0CdgHQ

"We have nothing to fear but veneer itself!"

"NATURE, TO BE COMMANDED, MUST BE OBEYED"

-- Francis Bacon

The huge and tragic losses in recent years from very large and even giant earthquakes and tsunamis . . . compels us to incorporate code requirements for greater public safety measures: measures that would more realistically both anticipate and deal with "what is possible;" not just with what is probable.

When buildings cannot withstand strong earthquake shaking, insufficient code requirements are simply leaving it "up-to-chance" . . . whether people live or die, and many of us who have witnessed the evolving weakening of earthquake design requirements now more than ever believe this is both not only improper but also entirely unreasonable for a civilized society.

When hazards are minimized, greater risks are made to seem somehow "acceptable." And with that we have become lulled into a false sense about of our earthquake security. Furthermore, in too many of these cases, we have been left with only "comfortable inaction" -- as our only preparation and defense against what so many earthquake professionals assured us were only rare or very unlikely events.

It is now clear, having witnessed so many recent and tragic occurrences, that public safety from future earthquakes and other so-called "extreme events" must be protected by more realistically assessing and designing for "what is possible," and not just for what is probable.

This proposed code change paves the way for: (a) performing seismic hazard assessment with the traditional, simpler and more realistic deterministic seismic hazard assessment dsha methodology, which fully considers the complete range of earthquake magnitudes that may be generated on any active earthquake fault -- up to and including the largest possible size event, which always is the most impactful to modern society;

(b) insuring that engineering design loads and building standards for all critical facilities and buildings can adequately withstand all these so-defined seismic hazards; and

(c) communicating fully (in clear and understandable language) such seismic hazards and seismic risks (including so-called "operational short term warnings") to government, stakeholders, and particularly to the public; so that not only personal safety but also community resilience shall be more reliably protected.

"Reading maketh a full man; conference a ready man; [week-long code hearings an exhausted man]; and writing an exact man." [Correctly considering the potential from all "possible" earthquakes, makes a safe man] -- Francis Bacon

So, to protect public safety . . . let's use our brains!

"PAY NO ATTENTION TO THAT MAN BEHIND THE CURTAIN!"

http://www.youtube.com/watch?v=fO9EU0w3CrY&featured=related

S110-12
Final Action: AS AM AMPC____ D

S111-12
1613.5 (New), 1613.5.1 (New)

Proposed Change as Submitted

Proponent: Kelly Cobeen, representing self; Dana Deke Smith and Steve Winkel, Building Seismic Safety Council, representing FEMA/Code Resource Support Committee (dsmith@nibs.org) (swinkel@preview-group.com)

Add new text as follows:

1613.5 Amendments to ASCE 7. The provisions of Section 1613.5 shall be permitted as an amendment to the relevant provisions of ASCE 7.

1613.5.1 Transfer of anchorage forces into diaphragm. Modify ASCE 7 Section 12.11.2.2.1 as follows:

12.11.2.2.1 Transfer of anchorage forces into diaphragm. Diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorages forces into the diaphragms. Diaphragm connections shall be positive, mechanical, or welded. Added chords are permitted to be used to form subdiaphragms to transmit the anchorage forces to the main continuous cross-ties. The maximum length-to-width ratio of a wood, wood structural panel, or untopped steel deck sheathed structural subdiaphragm that serves as part of the continuous tie system shall be 2.5 to 1. Connections and anchorages capable of resisting the prescribed forces shall be provided between the diaphragm and the attached components. Connections shall extend into the diaphragm a sufficient distance to develop the force transferred into the diaphragm.

Reason: The subdiaphragm aspect ratio is indicated in this proposal as only applying to wood sheathed diaphragms, wood structural panel sheathed diaphragms, and untopped metal deck diaphragm. When limitation of subdiaphragms was first submitted as a proposed change to the 1997 UBC by Kariotis [code change proposal 1631.2.8-95-1 K.A.S.E.] in the form of an allowable shear limitation, the reason focused on tilt-up buildings with nailed diaphragms and contemporary designs not meeting the intent of provisions written after observed poor performance in the 1973 Sylmar Earthquake. When approved for inclusion in the 1997 UBC [code change proposal 16-96-2 SEAOC/ Seismology] the approved wording for the aspect ratio limitation specifically applied only to wood structural subdiaphragms. In the process of being included in the IBC and ASCE 7, the wording designating wood subdiaphragms was dropped, making the requirement applicable to all subdiaphragms. This code change proposes to reintroduce the limit to wood subdiaphragms because they are the original system of concerns and observed poor performance, and include untopped steel deck diaphragms due to the similarities in construction and perceived structural behavior. This aspect ratio limit is not perceived to be necessary for good performance for other diaphragm types; once this aspect ratio limit is removed for concrete, composite deck, and other diaphragm types, other diaphragm limitations within the referenced material standards will govern design.

Cost Impact: The code change proposal will not increase the cost of construction and may reduce cost for some structural systems.

1613.5.1-S-COBEEN-SMITH-WINKEL.doc

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This code change corrects a mistake by amending the ASCE 7 provision for diaphragm anchorage forces. This clarifies that the subdiaphragm aspect ratio limit applies only to specific types of diaphragms.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Daniel J. Walker, P.E., Thomas Associates, Inc., representing Metal Building Manufacturers Association, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1613.5.1 Transfer of anchorage forces into diaphragm. Modify ASCE 7 Section 12.11.2.2.1 as follows:

12.11.2.2.1 Transfer of anchorage forces into diaphragm. Diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorages forces into the diaphragms. Diaphragm connections shall be positive, mechanical, or welded. Added chords are permitted to be used to form subdiaphragms to transmit the anchorage forces to the main continuous cross-ties. The maximum length-to-width ratio of a wood <u>or</u>~~,~~ wood structural panel~~, or untopped steel deck~~ sheathed structural subdiaphragm that serves as part of the continuous tie system shall be 2.5 to 1. Connections and anchorages capable of resisting the prescribed forces shall be provided between the diaphragm and the attached components. Connections shall extend into the diaphragm a sufficient distance to develop the force transferred into the diaphragm.

(Portions of proposal not shown remain unchanged)

Commenter's Reason: We were surprised to see the untopped steel deck included with this proposed requirement that was based on wood diaphragm performance observations and only addressed wood diaphragms in the 1997 UBC as stated in the original reason statement. As the reason stated, untopped steel decks were included in the proposal "due to similarities in construction and perceived structural behavior". Other than these construction types being lightweight, the link between their behavior is not very strong. We think this is not well supported, and that a new requirement shouldn't be based on a perception only.

S111-12
Final Action: AS AM AMPC____ D

S114-12
1703.1, 1703.1.1, 1703.3

<u>Proposed Change as Submitted</u>

Proponent: Phillip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1703.1 Approved agency. An *approved agency* shall provide all information as necessary for the *building official* to determine that the agency meets the applicable requirements <u>specified in Sections 1703.1.1 through 1703.1.4</u>.

1703.1.1 Independence. An *approved agency* shall be objective, competent and independent from the contractor responsible for the work being inspected. The agency shall also disclose <u>to the *building official* and the *registered design professional in responsible charge*</u> possible conflicts of interest so that objectivity can be confirmed.

1703.3 ~~Approved~~ Record <u>of approval</u>. For any material, appliance, equipment, system or method of construction that has been *approved*, a record of such approval, including the conditions and limitations of the approval, shall be kept on file in the *building official's* office and shall be ~~open to~~ <u>available for</u> public ~~inspection~~ <u>review</u> at appropriate times.

Reason: Section 1703.1 requires approved agencies to provide the information necessary for the building official to verify that the agency meets the applicable requirements but these requirements are not identified. The proposal specifies the sections containing the requirements.

Section 1703.1.1 requires approved agencies to disclose possible conflicts of interest so that objectivity can be confirmed but the recipient of the disclosure is not identified. The proposal specifies the building official and the registered design professional in responsible charge as the recipients.

Section 1703.3 is clarifies the requirement of the building official to provide access to the public for records of approval.

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

1703.1-S-BRAZIL.doc

<u>Public Hearing Results</u>

Committee Action: **Approved as Modified**

Modify proposal as follows:

1703.1 Approved agency. An *approved agency* shall provide all information as necessary for the *building official* to determine that the agency meets the applicable requirements specified in Sections 1703.1.1 through ~~1703.1.4~~ <u>1703.1.3</u>.

(Portions of proposal not shown are unchanged)

Committee Reason: The committee supports clarifying to whom an approved agency must disclose conflicts of interest and including the registered design professional in addition to the building official in a good idea. The floor modification corrects a section reference.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Mark K. Gilligan, S.E., representing self, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1703.1 Approved agency. An *approved agency* shall provide all information as necessary for the *building official* to determine that the agency meets the applicable requirements specified in Sections 1703.1.1 ~~through 1703.1.3~~ 1703.1.4.

1703.1.1 Independence. An *approved agency* shall be objective, competent and independent from the contractor responsible for the work being inspected. The agency shall also disclose to the *building official* ~~and the *registered design professional in responsible charge*~~ possible conflicts of interest so that objectivity can be confirmed.

1703.3 Record of approval. For any material, appliance, equipment, system or method of construction that has been *approved*, a record of such approval, including the conditions and limitations of the approval, shall be kept on file in the *building official's* office and shall be available for public review at appropriate times.

Commenter's Reason: Naming the design professional as a recipient of the information of potential conflicts of interest by the Approved Agency will change the design professional's scope of services with his client and increase the design professional's liability exposure. The Owner of the project who hires the Approved Agency is not required to be notified. This provision would make the design professional responsible for passing the information on to his client who is not listed. It is suggested that this provision would effectively create an obligation for the design professional to proactively inquiring whether the approved agency has any potential conflicts of interest to report so that the design professional could be assured that he had passed along the information to his client.

In the vast majority of situations the design professional has no contractual relationship with the approved agency and has no management responsibility with respect to the approved agency. The proposed provision would change this situation by placing the design professional between the approved agency and the Owner in a role where he has responsibility but no authority.

The design professional's right to rely on information provided by his/her Client or the Client's consultants or contractors is adequately covered by contract and existing case law. It is suggested that it is not the role of building codes to define the contractual relationship between the design professional and his client.

While the Building Official may have an interest in understanding potential conflicts of the agency, that it approved, it is not appropriate for the building code to change the contractual relationship between the design professional and his client. The building code should focus on the compliance of the project and not on how the Owner arranges to comply with the regulations. Thus reference to the registered design professional in responsible charge should be deleted from the proposed code change.

S114-12

Final Action: AS AM AMPC____ D

S118-12

1704.1, 1704.2.5.2, 1704.5 (New), 1705.12.3, 1910.5, 2207.5

Proposed Change as Submitted

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1704.1 General. This section provides minimum requirements for special inspections, the statement of special inspections, contractor responsibility, submittals to the *building official* and structural observations.

1704.2.5.2 Fabricator approval. *Special inspections* required by Section 1705 are not required where the work is done on the premises of a fabricator registered and *approved* to perform such work without *special inspection*. Approval shall be based upon review of the fabricator's written procedural and quality control manuals and periodic auditing of fabrication practices by an *approved special inspection* agency. At completion of fabrication, the *approved* fabricator shall submit a *certificate of compliance* to the owner or the owner's authorized agent for submittal to the *building official* as specified in Section 1704.5 stating that the work was performed in accordance with the *approved construction documents*.

1704.5 Submittals to the building official. In addition to the submittal of reports of *special inspections* and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner's authorized agent to the *building official* after review and acceptance by a *registered design professional* and prior to the construction or work being performed 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 1704.2.5.2
2. *Certificates of compliance* for the seismic qualification of nonstructural components, supports and attachments in accordance with Section 1705.12.3
3. *Certificates of compliance* for *designated seismic systems* in accordance with Section 1705.12.4
4. Reports of preconstruction tests for shotcrete in accordance with Section 1910.5
5. *Certificates of compliance* for open web steel joists and joist girders in accordance with Section 2207.5

(Renumber subsequent sections)

1705.12.3 Seismic certification of nonstructural components. The *registered design professional* shall specify on the construction documents the requirements for certification by analysis, testing or experience data for nonstructural components and designated seismic systems in accordance with Section 13.2 of ASCE 7, where such certification is required by Section 1705.12. *Certificates of compliance* shall be submitted to the *building official* as specified in Section 1704.5.

Revise as follows:

1910.5 Preconstruction tests. ~~When~~ Where preconstruction tests are required by ~~the *building official*~~ Section 1910.4, 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.

Revise as follows:

2207.5 Certification. At completion of manufacture, the steel joist manufacturer shall submit a *certificate of compliance* ~~in accordance with~~ to the owner or the owner's authorized agent for submittal to the *building official* as specified in Section ~~1704.2.5.2~~ 1704.5 stating that work was performed in accordance with *approved construction documents* and with SJI standard specifications.

Reason: The purpose for the proposal is to provide a new section (Section 1704.5) in the building code that comprehensively specifies the requirements for the submittal of reports and certificates related to construction that is subject to special inspections and tests required by Chapter 17 of the building code. Typically, these documents certify or otherwise verify that a material or product meets certain special requirements, or are alternatives to the general requirements, of the building code.

The items in new Section 1704.5 are typically references to provisions elsewhere in the building code or a referenced standard. The charging language of the new section specifies the requirements for submittal to the building official (e.g., by whom, after review and acceptance, and before the work begins) and the requirements apply equally to each listed submittal. The referenced provisions, however, contain additional requirements unique to each situation. The proposal modifies these provisions to be consistent with the submittal requirements in new Section 1704.5. For example, Item 2 requires submittal of the certificate of conformance "in accordance with Section 1705.12.3." Section 1705.12.3, in turn, requires submittal of the certificate of conformance "to the building official as specified in Section 1704.5." Similar language is found in Item 4 and corresponding Section 1910.5.

Item 1 is similar to Item 2 in that it requires submittal of the certificate of conformance "in accordance with Section 1704.2.5.2." Section 1704.2.5.2, however, requires submittal of the certificate of conformance to "the owner or the owner's authorized agent for submittal to the building official as specified in Section 1704.5...". This is because of the requirement in Section 1704.2.5.2 for submittal of the certificate of compliance by the approved fabricator and is done to avoid a conflict with new Section 1704.5. Similar language is found in Item 5 of new Section 1704.5 and corresponding Section 2207.5.

The charging statement in new Section 1704.5 states that the submittals are in addition to the submittal of reports of special inspections and tests because also listing them in the new section is not needed since this activity is already covered in Section 1704.2.4. It is also not advisable because the submittal of reports of special inspections and tests is the responsibility of approved agencies but the submittals listed in this new section are the responsibility of the owner or owner's authorized agent. Examples of reports of special inspections and tests submitted by approved agencies are: tests of concrete for strength, slump and air content (see Table 1705.3); tests of masonry units, grout and mortar (see Section 1705.4); and strength tests of shotcrete (see Table 1705.3).

Item 4 is included in new Section 1704.5 because the preconstruction tests required by Section 1910.4 are not also a requirement in Chapter 17 of the building code and requiring the submittal of test reports to the building official will enable the building official to verify, before construction begins, the validity of structural design assumptions based on the success of the preconstruction tests. Text requiring the submittal of the test reports to the building official is added to Section 1910.5 in conjunction with Item 4.

For Items 2 and 3 of new Section 1704.5, a separate proposal places the provisions of Section 1705.12.3 into two subsections (Sections 1705.12.3 and 1705.12.4) to provide effective charging language for the corresponding provisions in ASCE 7-10. In that proposal, requirements for the submittal of certificates of compliance to the building official are added to each subsection. This proposal for a new Section 1704.5 also adds a similar requirement to Section 1705.12.3 but the only purpose for doing so is to specify Section 1704.5. Should both proposals be approved by the ICC membership, our intent is that Section 1705.12.3 reads: "Certificates of compliance for the seismic qualification shall be submitted to the building official as specified in Section 1704.5;" and Section 1705.12.4 reads: "Certificates of compliance documenting that the requirements are met shall be submitted to the building official as specified in Section 1704.5."

Note that separate proposals:

1. Transfer the requirements of Section 1705.12.1 to new Section 1704.5;
2. Add additional requirements for submittals that are related to structural steel ;
3. Correlate the language in Section 1704.2.5 with the definition of "fabricated item" in Section 202;
4. Add additional requirements for submittals that are related to the welding of concrete reinforcement and anchor bolts;
5. Add additional requirements for submittals that are related to masonry;
6. Change "the owner" to "the owner or the owner's authorized agent";
7. Add a new Section 107.1.1 that correlates with this proposal; and
8. Add "responsible" before "registered design professional".

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

1704.1 #1-S-BRAZIL.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee feels the compilation of required submittals is a good idea, but there apparent confusion over the proposed wording. There's concern with requiring these before the start of construction could delay the construction process. There is also some concern with contractual issues being introduced into the code as well as with the registered design professional's acceptance of submittals.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Philip Brazil, P.E., S.E., representing self, and Lee Kranz, City of Bellevue, representing Washington Association of Building Officials, Technical Code Development Committee, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1704.5 Submittals to the building official. In addition to the submittal of reports of *special inspections* and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner's authorized agent to the *building official* ~~after review and acceptance by a *registered design professional* and prior to the construction or work being performed~~ for each of the following:

1. *Certificates of compliance* for the fabrication of structural, load-bearing or lateral load-resisting members or assemblies on the premises of a registered and *approved fabricator* in accordance with Section 1704.2.5.2
2. *Certificates of compliance* for the seismic qualification of nonstructural components, supports and attachments in accordance with Section 1705.12.3
3. *Certificates of compliance* for *designated seismic systems* in accordance with Section 1705.12.4
4. Reports of preconstruction tests for shotcrete in accordance with Section 1910.5
5. *Certificates of compliance* for open web steel joists and joist girders in accordance with Section 2207.5

(Portions of code change proposal not shown remain unchanged)

Commenter's Reason: In response to the Committee Reason and the testimony at the Dallas Code Development Hearing, the language for review and acceptance by a registered design professional and submittal prior to the construction or work being performed is deleted.

Note that separate proposals:

a. Change "the owner" to "the owner or the owner's authorized agent" throughout the IBC (S90-12-AS); and
b. Place the provisions of Section 1705.12.3 into two subsections (1705.12.3 and 1705.12.4) to provide effective charging language for the corresponding provisions in ASCE 7-10 (S129-12-AS).

S118-12

Final Action: AS AM AMPC____ D

S121-12
1704.2, 1704.2.1, 1704.2.4

Proposed Change as Submitted

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1704.2 Special inspections. Where application is made for construction as described in this section, the owner or the *registered design professional in responsible charge* acting as the owner's agent shall employ one or more *approved agencies* to ~~perform~~ provide inspections during construction on the types of work listed under Section 1705 and identify them to the *building official*. These inspections are in addition to the inspections identified in Section 110.

Exceptions:

1. *Special inspections* 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* 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 are not required for portions of structures designed and constructed in accordance with the cold-formed steel light-frame construction provisions of Section 2211.7 or the conventional light-frame construction provisions of Section 2308.

1704.2.1 Special inspector qualifications. Prior to the start of the construction, **t**he ~~special inspector~~ *approved agencies* shall provide written documentation to the building official demonstrating ~~his or her~~ the competence and relevant experience or training of the *special inspectors* who will perform the *special inspections* and tests during construction. Experience or training shall be considered relevant when the documented experience or training is related in complexity to the same type of *special inspection* activities for projects of similar complexity and material qualities. These qualifications are in addition to qualifications specified in other sections of this code. The *registered design professional in responsible charge* and engineers of record involved in the design of the project are permitted to act as the *approved agency* and their personnel are permitted to act as the special inspector for the work designed by them, provided they qualify as special inspectors.

1704.2.4 Report requirement. ~~Special inspectors~~ *Approved agencies* shall keep records of inspections. The ~~special inspector~~ *approved agency* shall furnish inspection reports to the *building official*, and to the *registered design professional in responsible charge*. Reports shall indicate that work inspected was or was not completed in conformance to *approved construction documents*. Discrepancies shall be brought to the immediate attention of the contractor for correction. If they are not corrected, the discrepancies shall be brought to the attention of the *building official* and to the *registered design professional in responsible charge* prior to the completion of that phase of the work. A final report documenting required *special inspections* and correction of any discrepancies noted in the inspections shall be submitted at a point in time agreed upon prior to the start of work by the applicant and the *building official*.

Reason: Section 1704.2 requires the owner or owner's agent to employ approved agencies to perform special inspections and tests required by Section 1705. The act of an owner or owner's agent to employ an approved agency for this purpose, however, is a private matter (typically contractual) and not an appropriate subject for a building code that requires compliance with its provisions. The proposal revises the language to require the owner or owner's agent to identify to the building official the approved agencies who will provide the special inspections and tests required by Section 1705 that will be performed by special inspectors and others (e.g., testing lab personnel) employed or retained by the approved agency.

Section 1704.2.1 requires special inspectors to provide documentation of their qualifications to the building official but it does not specify when this is required to occur. Being a subsection of Section 1704.2, Section 1704.2.1 also does not specify the relationship between the special inspector providing documentation of qualifications and the owner or owner's agent employing an approved agency. Special inspectors are employed or retained by an approved agency to perform special inspections (see definition of "special inspector" in Section 202). The proposal revises the language to require the approved agency to provide to the building official prior to the start of construction documentation of the qualifications for the special inspectors who will perform the special inspections and tests during construction.

An example of written documentation demonstrating the competence and relevant experience of an approved agency would be evidence of accreditation as an approved agency by the International Accreditation Service (IAS), Inc. The requirements for obtaining and maintaining such accreditation from the IAS are in the Accreditation Criteria for Special inspection Agencies, AC291. Notable provisions in AC291 are definitions, many of which are from 2012 IBC Section 202 (Section 2); information required to be submitted by the agency for accreditation (Section 3); requirements for inspection reports issued by the agency, including compliance with the reporting requirements of IBC Chapter 17 (Section 4); requirements for training, supervision and monitoring of special inspectors (Section 5); and minimum qualifications of special inspectors for specific classes of construction, including those in 2012 IBC Section 1705 (Section 6).

Section 1704.2.4 requires special inspectors to keep records of inspections and furnish inspection reports to the building official and the registered design professional in responsible charge. Special inspectors do generate records of their actions but these are typically kept for submittal by the approved agency that employs or retains them. Section 1704.2.4 is changed to require approved agencies to keep records of special inspections and tests and to submit the reports to the building official and the registered design professional in responsible charge.

Note that separate proposals also revise Section 1704.2 to:

1. Distinguish between special inspections and tests by approved agencies and inspections by the building official;
2. Clarify that the application is made to the building official as specified in Section 105 ; and
3. Update references to "approved agency" throughout the building code, including instances of "special inspection agency".

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

1704.2 #2-S-BRAZIL.doc

Public Hearing Results

Committee Action: Approved as Modified

Modify proposal as follows:

1704.2 Special inspections. Where application is made for construction as described in this section, the owner or the *registered design professional in responsible charge* acting as the owner's agent shall employ one or more *approved agencies* to provide inspections during construction on the types of work listed under Section 1705 and identify ~~them~~ <u>the inspections</u> to the *building official*. These inspections are in addition to the inspections identified in Section 110.

Exceptions:

1. *Special inspections* 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* 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 are not required for portions of structures designed and constructed in accordance with the cold-formed steel light-frame construction provisions of Section 2211.7 or the conventional light-frame construction provisions of Section 2308.

(Portions of proposal not shown are unchanged)

Committee Reason: This code clarifies when the documentation of special inspector qualification must be submitted to the building official. It also clears up who keeps the inspection records and furnishes them to the building official. The modification makes it clear that the inspections are to be identified.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Philip Brazil, P.E., S.E., representing self; and Lee Kranz, City of Bellevue, representing Washington Association of Building Officials, Technical Code Development Committee, request Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1704.2 Special Inspections. Where application is made for construction as described in this section, the owner or the *registered design professional in responsible charge* acting as the owner's agent shall employ one or more *approved agencies* to provide inspections during construction on the types of work listed under Section 1705 and identify the ~~inspections~~ approved agencies to the *building official*. These inspections are in addition to the inspections specified in Section 110.

Exceptions:

1. *Special inspections* 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* 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 are not required for portions of structures designed and constructed in accordance with the cold-formed steel light-frame construction provisions of Section 2211.7 or the conventional light-frame construction provisions of Section 2308

(Portions of proposal not shown remain unchanged)

Commenter's Reason: The purpose for the public comments is to correct an inadvertent error in the approved proposal. In the originally submitted proposal, "them" meant the approved agencies, not the inspections. This was also noted in the first paragraph of the reason statement. The public comment makes the necessary adjustment to the language.

S121-12
Final Action: AS AM AMPC____ D

S123-12
1704.2.5, 1704.2.5.1, 1704.2.5.2, 1705.10 (New)

Proposed Change as Submitted

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing self (pbrazil@reidmiddleton.com)

Revise as follows:

1704.2.5 Special inspection of ~~fabricators~~ fabricated items. Where fabrication of structural load-bearing members and assemblies is being ~~performed~~ conducted on the premises of a fabricator's shop, *special inspections* of the fabricated items shall be ~~required by this section and as required elsewhere in this code~~ performed during fabrication.

Exceptions:

1. ~~**Fabrication and implementation procedures.**~~ *Special inspections* during fabrication are not required where the special inspector ~~shall verify~~ verifies that the fabricator maintains detailed fabrication and quality control procedures that provide a basis for inspection control of the workmanship and the fabricator's ability to conform to *approved construction documents* and referenced standards. The special inspector shall review the procedures for completeness and adequacy relative to the code requirements for the fabricator's scope of work.

2. *Special inspections* ~~as required by Section 1704.2.5 shall~~ are not ~~be~~ required where the fabricator is registered and *approved* in accordance with Section 1704.2.5.2.

~~**1704.2.5.2**~~ **1704.2.5.1 Fabricator approval.** *Special inspections* ~~required by Section 1705~~ during fabrication are not required where the work is done on the premises of a fabricator registered and *approved* to perform such work without *special inspection*. Approval shall be based upon review of the fabricator's written procedural and quality control manuals and periodic auditing of fabrication practices by an *approved special inspection* agency. At completion of fabrication, the *approved* fabricator shall submit a *certificate of compliance* to the *building official* stating that the work was performed in accordance with the *approved construction documents*.

1705.10 Fabricated items. Special inspections of fabricated items shall be performed in accordance with Section 1704.2.5.

(Renumber subsequent sections)

Reason: Section 1704.2.5 requires special inspections to be performed for all structural load-bearing members and assemblies that are fabricated on the premises of a fabricator's shop (e.g., not at the construction site) as specified in the section and elsewhere in the building code. One example of this is the fabrication of metal-plate-connected wood trusses, which is subject to the special inspections required by Section 1704.2.5. Special inspections of the installation of the trusses at the construction site is not required except for trusses spanning 60 feet or greater (Section 1705.5.2).

A second example is the fabrication of precast, prestressed, concrete members (e.g., hollow-core slabs), which is also subject to the special inspections required by Section 1704.2.5 as well as those of Section 1705.3 for concrete construction. Note that Item 9 of Table 1705.3 specifies inspection of prestressed concrete.

Section 1704.2.5 requires special inspections of the fabricated items. Section 1704.2.5.1 specifies duties of the special inspector but these duties are not directly related to special inspections of the fabricated items. Instead, the specified duties are typical of what is conducted by an approved agency for the accreditation of a fabricator by a nationally recognized accreditation service such as the International Accreditation Service. Based on Section 1704.2.5, these duties are required in addition to special inspections of the fabricated items that are required elsewhere in the building code, such as for precast, prestressed, concrete members.

The proposal modifies the provisions in Section 1704.2.5 by requiring special inspections of fabricated items during fabrication. Section 1704.2.5.1 is changed to an exception making it an alternative to the basic requirement for special inspection in Section 1704.2.5.

The other changes in the proposal are made to clarify the language. Section 1705.10 is added because Section 1704.2.5 requires special inspections except where the work is done on the premises of an approved fabricator (Section 1704.2.5.2) and should be included in Section 1705, which specifies required special inspection and tests.

The current provisions in Section 1704.2.5.2 (renumbered to Section 1704.2.5.1 are an acknowledgement that there are fabricators who (1) fabricate products or assemblies with sufficient quality and through the application of documented procedures (e.g., quality management systems), and (2) and are recognized for this through certification, accreditation or qualification by a national recognized organization providing such services, that they should be exempt from further requirements for special inspection of fabrication. Examples are:

1. The certification program of steel fabricators and erectors by the American Institute of Steel Construction (AISC), which is audited by the Quality Management Company;
2. The accreditation of the fabrication inspection programs for reinforced concrete and precast/prestressed concrete, structural steel and wood wall panels by the International Accreditation Service (IAS) (see AC157, AC172 and AC196, respectively, for accreditation criteria);
3. The accreditation of the inspection programs for manufacturers of metal building systems by the International Accreditation Service (IAS) (see AC472 for accreditation criteria); and
4. Qualification of prefabricated items such as prefabricated wood shear panels, cold-formed, pin-connected open-web trusses with wood chords and tubular or angular steel webs, and steel lateral-force-resisting vertical assemblies, as alternatives to applicable requirements in the IBC or other codes by the ICC Evaluation Service (ICC-ES) (see AC130, AC306 and AC322, respectively, for acceptance criteria).
5. The certification of structural and architectural concrete products by the Precast, Prestressed Concrete Institute (PCI).
6. The certification of precast concrete products by the National Precast Concrete Association (NPCA).

Note that separate proposals:

1. Revise Section 1704.2.5.2 to specify that the approved fabricator is required to submit the certificate of compliance to the owner or the owner's authorized agent in conjunction with the requirement in proposed Section 1704.5 for submittal of the certificate to the building official;
2. Revise Sections 1704.2.5 and 1704.2.5.1 for consistency with and to correlate with the definition of "fabricated item" in Section 202; and
3. Revise Section 1704.2.5.2 and other sections to update references to "approved agency" throughout the building code.

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

1704.2.5 #1-S-BRAZIL.doc

Public Hearing Results

Committee Action: Approved as Submitted

Committee Reason: This code change properly identifies conditions under which special inspections of fabricators are required.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment 1:

Philip Brazil, P.E., S.E., representing self; and Lee Kranz, City of Bellevue, representing Washington Association of Building Officials, Technical Code Development Committee; and Constadino (Gus) Sirakis, PE, representing New York City Department of Buildings, request Approval as Modified by this Public Comment.

Modify the proposal as follows:

1704.2.5 Special inspection of fabricated items. Where fabrication of structural load-bearing members and assemblies is being conducted on the premises of a fabricator's shop, *special inspections* of the fabricated items shall be performed during fabrication.

Exceptions:

1. *Special inspections* during fabrication are not required where the ~~special inspector verifies that the~~ fabricator maintains *approved* detailed fabrication and quality control procedures that provide a basis for inspection control of the workmanship and the fabricator's ability to conform to *approved construction documents* and referenced standards. ~~The special inspector shall review the procedures for completeness and adequacy relative to the code requirements for the fabricator's scope of work.~~ Approval shall be based upon review of fabrication and quality control procedures and periodic inspection of fabrication practices by the building official.

2. *Special inspections* are not required where the fabricator is registered and *approved* in accordance with Section 1704.2.5.2.

1704.2.5.1 Fabricator approval. *Special inspections* during fabrication are not required where the work is done on the premises of a fabricator registered and *approved* to perform such work without *special inspection*. Approval shall be based upon review of the fabricator's written procedural and quality control manuals and periodic auditing of fabrication practices by an *approved special inspection* agency. At completion of fabrication, the *approved* fabricator shall submit a *certificate of compliance* to the *building official* stating that the work was performed in accordance with the *approved construction documents*.

(Portions of proposal not shown remain unchanged)

Commenter's Reason: The purpose for the public comment is to clarify that the exemption from required special inspections in Exception #1 is permitted only when approved by the building official. The language is revised to require the fabricator to maintain *approved* detailed fabrication and quality control procedures and *approved* is defined in IBC Section 202 as "acceptable to the *building official* or authority having jurisdiction." The added language for approval to be based upon review of fabrication and quality control procedures and periodic inspection by the building official is for consistency with language in Section 1704.2.5.1 for similar actions by the approved agency.

Note that separate proposals:

a. Revise Section 1704.2.5.2 and other sections to update references to "approved agency" throughout the building code (e.g., change from "approved special inspection agency" to "approved agency," S117-12-AM); and
b. Revise Sections 1704.2.5 and 1704.2.5.1 for consistency with and to correlate with the definition of "fabricated item" in Section 202 (e.g., change from "referenced standards" to "this code," S124-12-AS).

Public Comment 2:

Bonnie E. Manley, American Iron and Steel Institute, representing American Institute of Steel Construction, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1704.2.5 Special inspection of fabricated items. Where fabrication of structural load-bearing members and assemblies is being conducted on the premises of a fabricator's shop, *special inspections* of the fabricated items shall be performed during fabrication.

Exceptions:

1. *Special inspections* as specified by Section 1705, excluding Sections 1705.10, 1705.11, and 1705.12, during fabrication are not required where the special inspector verifies that the fabricator maintains detailed fabrication and quality control procedures that provide a basis for inspection control of the workmanship and the fabricator's ability to conform to *approved construction documents* and referenced standards. The special inspector shall review the procedures for completeness and adequacy relative to the code requirements for the fabricator's scope of work.

2. *Special inspections* as specified by Section 1705, excluding Sections 1705.10, 1705.11, and 1705.12, are not required where the fabricator is registered and *approved* in accordance with Section 1704.2.5.1~~2~~.

1704.2.5.1 Fabricator approval. *Special inspections* during fabrication as specified by Section 1705, excluding Sections 1705.10, 1705.11, and 1705.12, are not required where the work is done on the premises of a fabricator registered and *approved* to perform such work without *special inspection*. Approval shall be based upon review of the fabricator's written procedural and quality control manuals and periodic auditing of fabrication practices by an *approved special inspection* agency. At completion of fabrication, the *approved* fabricator shall submit a *certificate of compliance* to the *building official* stating that the work was performed in accordance with the *approved construction documents*.

1705.10 Fabricated items. Special inspections of fabricated items shall be performed in accordance with Section 1704.2.5.

Commenter's Reason: This public comment builds upon the approved changes in Proposal S123-12 by reintroducing the primary modification recommended in Proposal 126-12. That proposal was disapproved by the ICC Structural Code Committee because the "Action taken on S123-12 was preferred" – hence, this public comment, which folds the changes recommended in Proposal S126-12 in on top of the changes approved in Proposal S123-12. Specifically, this comment corrects the unintended consequences of modifications made by Proposal S116-09/10, effective with IBC 2012. That proposal reorganized Chapter 17 and combined all special inspections and tests into Section 1705, including requirements for additional special inspection and testing for wind resistance and seismic resistance. Previously, special inspections for wind resistance and seismic resistance were not permitted to be waived from special inspections under the approved fabricators provisions, as demonstrated by the modifications successfully made under Proposal S109-07/08 for the 2009 IBC. Proposal S109–07/08 added the specific reference to Section 1704 into 1704.2.2, with the reason stated as follows:

> "This modification attempts to clarify exactly which inspections are permitted to be waived when work is done by a registered and approved fabricator. As written now, it could be interpreted to mean that the special inspections for seismic resistance required by Section 1707.2 could be waived. This is not appropriate and needs to be corrected."

We believe that the community inadvertently took a step back in the 2012 IBC with the success of Proposal S116-09/10 and remain committed to the belief that special inspections identified for seismic and wind resistance should not be waived even for approved fabricators. The systems addressed by these special inspections are critical to the performance of the building in a wind or seismic event and therefore warrant the higher level of attention.

S123-12

Final Action: AS AM AMPC____ D

S128-12
1704.3.1

Proposed Change as Submitted

Proponent: Stephen Kerr, S.E., Josephson Werdowatz and Associates, representing Structural Engineers Association of California (SEAOC) (skerr@jwa-se.com)

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 inspection* or testing 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 inspection* or testing for seismic or wind resistance as specified in Sections 1705.10, 1705.11 and 1705.12.
5. For each type of *special inspection*, ~~identification as to~~ whether it will be continuous *special inspection*, ~~or~~ periodic *special inspection*, or performed at a frequency in accordance with the notation used in the reference standard where the inspections are defined.

Reason: The quality assurance requirements of AISC 360 and AISC 341, which are referenced as the standard for special inspections and testing for structural steel, do not describe the frequency of the inspections as "periodic" or "continuous." Rather, detailed inspection tasks are defined, and the level of effort for each task is described by the terms "Observe" and "Perform". This proposal accommodates this alternate approach to the frequency of special inspection.

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

1704.3.1-S-KERR.doc

Public Hearing Results

Committee Action: Disapproved

Committee Reason: The committee prefers that special inspections be referred to strictly as continuous or periodic. There is no requirement to add wording frequencies according to reference standards.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Stephen Kerr, representing Structural Engineers Association of California, requests Approval as Modified by this Public Comment.

Modify the proposal 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 inspection* or testing 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 inspection* or testing for seismic or wind resistance as specified in Sections 1705.10, 1705.11 and 1705.12.
5. For each type of *special inspection*, identification as to whether it will be continuous special inspection, periodic special inspection, or performed ~~at a frequency~~ in accordance with the notation used in the reference standard where the inspections are defined.

Commenter's Reason: The quality assurance requirements of AISC 360 and AISC 341, which are referenced as the standard for special inspections and testing for structural steel, do not describe the frequency of the inspections as "periodic" or "continuous." Rather, detailed inspection tasks are defined, and the level of effort for each task is described by the terms "Observe" and "Perform". Whereas inspection frequency "periodic or continuous" is time dependent, interval to "observe or perform" is project dependent based on design. Neither the building official nor the design professional of record can control the work of the contractor or that of the special inspector, except to identify the critical elements which need special inspection. This proposal accommodates this alternate approach to the frequency of special inspection in accordance with the commentary on section N of AISC 360.

S128-12

Final Action: AS AM AMPC____ D

S133-12
1704.5 (New), Chapter 35 (New)

Proposed Change as Submitted

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing self (pbrazil@reidmiddleton.com)

Add new text as follows:

1704.5 Submittals to the building official. In addition to the submittal of reports of *special inspections* and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner's authorized agent to the *building official* after review and acceptance by a *registered design professional* and prior to the construction or work being performed for each of the following:

1. Welding procedure specifications in accordance with Section 6.1.2 of AWS D1.4 for the welding of concrete reinforcement other than by fillet welds.
2. Test reports for Grade 55 anchor bolts verifying compliance with Supplementary Requirement S1 of ASTM F 1554 for weldability.
3. Test reports for Grade A and B anchor bolts verifying compliance with Supplementary Requirement S1 of ASTM A 307 for weldability.

Add new standard to Chapter 35 as follows:

ASTM

F1554-07a Standard Specification for Anchor Bolts, Steel, 36, 55 and 105-ksi Yield Strength

Reason: This proposal is a continuation of a separate proposal that adds a new Section 1704.5 specifying submittals to the building official. This proposal adds three items to those in the separate proposal and the charging language in new Section 1704.5 is identical in both proposals.

Item 1 is added to new Section 1704.5 because Section 6.1.2 of AWS D1.4 requires qualification testing for the welding procedure specifications (WPS) of all types of welded joints that include reinforcing bars except for those consisting of fillet welds, which are deemed to be prequalified and, thus, exempt from testing. Section 6.1.2.3 of the standard requires the WPS to be made available to those authorized to examine them. The requirement for availability means that welding procedure specifications are available for submittal to the building official. Requiring their submittal to the building official will enable the building official to verify whether the welded joints are adequately designed to meet applicable requirements. Note that the 1998 edition of AWS D1.4 is a referenced standard of the 2012 IBC (see Chapter 35) but the 2011 edition is the current edition.

Item 2 is added to new Section 1704.5 because Grade 55 anchor bolts complying with ASTM F 1554-07a are not suitable for welding but weldable steel is possible, provided the material for the bolts meets Supplementary Requirement S1 of the standard. In ASTM F 1554-07a, Section 4.2 classifies Grade 55 anchor bolts complying with Supplementary Requirement S1 as weldable, Section 5.1 requires orders for anchor bolts to include required test reports (Section 5.1.13), and Section 17.1 requires the purchaser to be furnished with a test report that includes the carbon equivalent in accordance with Supplementary Requirement S1 (Section 17.1.1). The requirement that the purchaser be furnished with the test reports means that they are available for submittal to the building official. Requiring their submittal to the building official will enable the building official to verify whether the anchor bolts meet the applicable requirements for weldability.

Grade 36 bolts complying with ASTM F 1554-07a are weldable because of the limits on carbon in Table 1 ("Chemical Requirements for Grade 36") of the standard, which are 0.26%-0.28% by heat analysis and 0.29%-0.31% by product analysis depending on the bolt diameter. Grade 55 anchor bolts not complying with Supplementary Requirement S1 are not weldable because of the lack of limits on carbon in Table 2 ("Chemical Requirements for Grades 55 and 105") of the standard. In Supplementary Requirement S1, Section S1.2 assumes that suitable welding procedures for the steel being welded and the intended service will be selected, Section S1.5.1 specifies limits on carbon of 0.30% by heat analysis and 0.33% by product analysis, Section S1.5.2 requires an analysis of the carbon equivalent (CE) verifying that limits on CE are met (0.45% for alloy and low-alloy steel and 0.40% for carbon steel), and Section S1.6 requires the anchor bolts to be designated by a white paint mark on the side of the bar to be encased in concrete.

Of the ASTM standards applicable to other commonly used anchor bolts, Table 2 ("Chemical Requirements") of ASTM A 36 for carbon steel shapes, plates and bars of structural quality limits carbon in bars to 0.26%-0.29% depending on nominal diameter; and Table 1 ("Chemical Requirements for Grades A and B Bolts and Studs") of ASTM A 307 for carbon steel bolts and studs limits carbon in Grade A and B bolts and studs to 0.29% by heat analysis and 0.33% by product analysis. ASTM A 307 Grade C bolts and studs are specified as having properties complying with ASTM A 36 (Section 1.1). The effect of these provisions is that anchor bolts with properties complying with ASTM A 36 (e.g., ASTM A 307, Grade C) are weldable but anchor bolts complying with ASTM A 307, Grade A or B, may not be weldable and the standard specifies additional requirements (Section 1.5) to ensure weldability

(Supplementary Requirement S1) that are similar to those in ASTM F 1554-07a. Item 3 is added to new Section 1704.5 because of this.

Note that separate proposals:

1. Transfer the requirements of Section 1705.12.1 to new Section 1704.5 ;
2. Add additional requirements for submittals that are related to structural steel;
3. Add additional requirements for submittals that are related to masonry ; and
4. Add a new Section 107.1.1 that correlates with this proposal.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] 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, 2012.

1704.5 (NEW) #1-S-BRAZIL.doc

Public Hearing Results

Note: For staff analysis of the content of ASTM F 1554 relative to CP#28, Section 3.6, please visit:
http://www.iccsafe.org:8888/cs/codes/Documents/2012-13cycle/Proposed-A/00a_updates.pdf

Committee Action: Disapproved

Committee Reason: Disapproval is consistent with the committee's action on S118-12. There's concern that requiring these submittals before the start of construction could delay the construction process.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Philip Brazil, P.E., S.E., representing self, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1704.5 Submittals to the building official. In addition to the submittal of reports of *special inspections* and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner's authorized agent to the *building official* ~~after review and acceptance by a *registered design professional* and prior to the construction or work being performed~~ for each of the following:

1. Welding procedure specifications in accordance with ~~Section 6.1.2 of~~ AWS D1.4 for the welding of concrete reinforcement other than by fillet welds.
2. Test reports for Grade 55 anchor bolts verifying compliance with Supplementary Requirement S1 of ASTM F 1554 for weldability.
3. Test reports for Grade A and B anchor bolts verifying compliance with Supplementary Requirement S1 of ASTM A 307 for weldability.

(Portions or proposal not shown remain unchanged)

Commenter's Reason: In response to the Committee Reason and the testimony at the Dallas Code Development Hearing, the language for review and acceptance by a registered design professional and submittal prior to the construction or work being performed is deleted from the charging text. Also, the section reference in Item #1 is deleted to eliminate the need to correlate the standard with the IBC in the future. The language in Item #1 is sufficiently descriptive to make the section reference unnecessary.

Note that a separate proposal changes "the owner" to "the owner or the owner's authorized agent" throughout the IBC (S90-12-AS).

S133-12
Final Action: AS AM AMPC____ D

S134-12
1704.5 (New), Chapter 35 (New)

Proposed Change as Submitted

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, representing self (pbrazil@reidmiddleton.com)

Add new text as follows:

1704.5 Submittals to the building official. In addition to the submittal of reports of *special inspections* and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner's authorized agent to the *building official* after review and acceptance by a *registered design professional* and prior to the construction or work being performed for each of the following:

1. Test reports verifying compliance with Supplementary Requirement S30 of ASTM A6 for W-shaped and WT-shaped elements of structural steel with flange thicknesses of 1-1/2 inches (38 mm) or greater that are required to have a Charpy V-notch toughness as specified in Section A3.3 of AISC 341;
2. Test reports verifying compliance with Supplementary Requirement S5 of ASTM A6 for structural steel plates of 2 inches (51 mm) in thickness or greater that are required to have a Charpy V-notch toughness as specified in Section A3.3 of AISC 341;
3. *Certificates of compliance* for verification that welds at elements of structural steel and their connections that are in the *seismic force-resisting system* are made with filler metal having a Charpy V-notch toughness as specified in Section A3.3a of AISC 341;
4. *Certificates of compliance* for verification that demand critical welds are made with filler metal having a Charpy V-notch toughness as specified in Section A3.3b of AISC 341;
5. Test reports verifying compliance with Supplementary Requirement S30 of ASTM A6 for hot-rolled shapes of structural steel with flange thicknesses greater than 2 inches (51 mm) that are required to have a Charpy V-notch toughness as specified in Section A3.1c of AISC 360;
6. *Certificates of compliance* for the fabrication of steel buckling-restrained braces on the premises of an *approved fabricator* in accordance with Section 1704.2.5.2.

Add new standard to Chapter 35 as follows:

ASTM

A 6-11 Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes and Sheet Piling

Reason: This proposal is a continuation of a separate proposal that adds a new Section 1704.5 specifying submittals to the building official. This proposal adds six items to those in the separate proposal and the charging language in new Section 1704.5 is identical in both proposals. The parenthetic references to AISC 341-05 below are provided for reference and correspond to the referenced provisions of AISC 341-10. Similarly, there are parenthetic references to AISC 360-05 that correspond to the referenced provisions of AISC 360-10.

Items 1 and 2 are added to new Section 1704.5 because of the requirements in Section A3.3 of AISC 341-10 (Section 6.3 of AISC 341-05) for minimum Charpy V-notch (CVN) toughness in (1) hot rolled shapes of structural steel with flange thicknesses of 1-1/2 inches or greater, and (2) structural steel plates 2 inches in thickness or greater and meeting the condition specified therein, where they are elements of the seismic force-resisting system in structures within the scope of AISC 341. However, there are no provisions in AISC 341-10 (or AISC 341-05) for verification by the building official (authority having jurisdiction) that the requirements are met.

The condition specified in Section A3.3 of AISC 341-10 for steel plates is that Charpy V-notch (CVN) toughness is limited for (1) members built up from plate, (2) connection plates where inelastic strain under seismic loading is expected, and (3) the steel core of buckling-restrained braces. Note that there is apparently an error in Section A3.3 of AISC 341-10 for hot-rolled shapes in that the minimum flange thickness is specified as 1/2 inch (38 mm) but, given the stated thickness in millimeters, 1-1/2 inches is intended.

Section A3.3 of AISC 341-10 (Section 6.3 of AISC 341-05) requires the structural steel to comply with Section A3.1c of AISC 360-10 (Section A3.1c of AISC 360-05). For hot rolled shapes of structural steel with flange thicknesses greater than 2 inches and meeting the conditions specified therein, Section A3.1c of AISC 360-10 requires the construction documents (structural design documents) to specify that such shapes shall be supplied with CVN impact test results in accordance with ASTM A6, Supplementary

Requirement S30. Assuming that it is not the intent for the shapes to supply the test results, it is assumed that the intent is for tests in accordance with ASTM A6, Supplementary Requirement S30 to be conducted on the shapes.

Section A3.3 of AISC 341-10 also requires the structural steel to be tested for CVN toughness as specified in ASTM A6, Supplementary Requirement S30, for hot-rolled shapes and in accordance with ASTM A 673 for steel plate. This has the effect of modifying the requirement in Section A3.1c of AISC 360-10 to lower the threshold for CVN impact testing of hot-rolled shapes of structural steel to those with flange thicknesses of 1-1/2 inches or greater and to also require CVN impact testing for structural steel plates that are 2 inches in thickness or greater. The requirement for test results means that test reports are available for submittal to the building official. Requiring their submittal to the building official will enable the building official to verify whether the structural steel meets the applicable requirements for CVN toughness.

In ASTM A 6-11, Section 1.8 indicates that the supplementary requirements therein are for use where additional testing or restrictions are required by the purchaser in the purchase order, Section 14.1 requires test reports for each heat supplied, and Section 14.1.6 requires the test reports to report the results of tests required by the purchase order. As for Section A3.1c of AISC 360-10 (discussed above), the requirement for test reports means that they are available for submittal to the building official, and requiring their submittal to the building official will enable the building official to verify whether the structural steel meets the applicable requirements for CVN toughness.

Supplementary Requirement S5 of ASTM A 6-11 requires CVN impact tests to be conducted in accordance with ASTM A 673 (Section S5.1). Supplementary Requirement S30 of ASTM A 6-11 requires CVN impact tests to be conducted in accordance with ASTM A 673 using specimens taken from the alternate core location (Section S30.1). This means that the supplementary requirements are identical in that both require impact testing in accordance with ASTM A 673 to determine CVN toughness except that Supplementary Requirement S30 imposes an additional condition on the testing, which is to take specimens from the alternate core location. Section A3.3 of AISC 341-10 references ASTM A 673 for steel plate but the proposal references Supplementary Requirement S5 of ASTM A 6-11 for consistency with the reference to Supplementary Requirement S30 of ASTM A 6-11 for hot-rolled shapes of structural steel.

Item 1 is limited in scope to W-shaped and WT-shaped structural members because the requirement in Section A3.3 of AISC 341-10 (Section 6.3 of AISC 341-05) for minimum CVN toughness is limited to hot-rolled shapes of structural steel with flange thicknesses of 1-1/2 inches or greater, which occur only in W-shaped and WT-shaped elements of structural steel. Section 3.1.2 of ASTM A 6-11 defines "shapes" as including "W" shapes, "HP" shapes, "S" shapes, "M" shapes, "C" shapes, "MC" shapes and "L" shapes. Of these shapes, the *AISC Steel Construction Manual* (thirteenth edition) only lists W-shaped and WT-shaped elements of structural steel with flange thicknesses of 1-1/2 inches or greater (Tables 1-1 and 1-8). Note that the *Manual* also does not list any "MT" shapes or "ST" shapes with flange thicknesses of 1-1/2 inches or greater.

The provisions in Section A3.3 of AISC 341-10 (Section 6.3 of AISC 341-05) and Section A3.1c of AISC 360-10 (Section A3.1c of AISC 360-05) are limited to hot-rolled shapes of structural steel but are not limited by type of shape. In Items 1 and 2 of this proposal, however, the requirement for submittal of test reports is limited by type of shape but is not limited to hot-rolled shapes of structural steel. The type of shape is limited to eliminate extraneous shapes for which the requirement for submittal does not apply. Limiting the requirement for submittal to shapes that are hot-rolled is not included because "hot-rolled" is a manufacturing process and is not relevant to the requirement for submittal. The "hot-rolled" limit is also not included for consistency with ASTM A 6-11 whose scope specifies the standard as applying to "rolled structural steel bars, plates, shapes and sheet piling" (Section 1.1).

Section A3.3 of AISC 341-10 and Section A3.1c of AISC 360-10 do specify hot-rolled shapes and the same is true of Section 6.3 of AISC 341-05 and Section A3.1c of AISC 360-05. None of these standards, however, define "hot-rolled" nor, to my knowledge, does any referenced standard of the 2012 IBC or any other standard referenced in the AISC standards listed above.

Items 3 and 4 are added to new Section 1704.5 because of the requirements in Sections A4.4a and A4.4b of AISC 341-10 (Sections 7.3a and 7.3b of AISC 341-05) for minimum CVN toughness of welds that are used in elements of structural steel and their connections that are in the seismic force-resisting system of structures within the scope of AISC 341. AISC 341-05 directly specifies the requirements. AISC 341-10 indirectly specifies them by referencing the requirements in Section (Clause) 6.3 of AWS D1.8. As for Items 1 and 2 of the proposal (discussed above), there are no provisions in AISC 341-10 (or AISC 341-05) for verification by the building official (authority having jurisdiction) that the requirements are met.

Section (Clause) 6.3 of AWS D1.8 (2009 edition) contains requirements for filler and weld metal of welds, including demand critical welds, that are within the scope of the standard. Among those requirements, Sections 6.3.1 and 6.3.5 specify mechanical properties for filler metals, including minimum CVN toughness, of welds and demand critical welds, respectively, which are listed in corresponding Tables 6.1 and 6.2. Note that AWS D1.8 is not a referenced standard of the 2012 IBC.

Section (Clause) 6.1.1 of AWS D1.8 requires welding procedure specifications to be prequalified, or to be qualified by testing in accordance with applicable AWS D1.1 requirements. Note that Section 1.1 of AWS D1.8 (1) establishes the applicability of AWS D1.8 as supplementing AWS D1.1 and (2) states that the provisions in AWS D1.1 apply to the welds governed by the provisions AWS D1.1 except where modified in AWS D1.8.

Section (Clause) 4.0 of AWS D1.1 (2008 edition) contains requirements for qualification testing of welding procedure specifications (WPS's). Section 3.1, however, exempts prequalified welding procedure specifications from requirements for qualification testing. A WPS is required to meet the provisions of Chapter 3 of AWS D1.1 in order to be prequalified. However, there are no provisions in Chapter 3 for minimum CVN toughness. Section 4.1.1.3 requires CVN tests to be included in the WPS qualification where required by the construction (contract) documents. Section 1.4.1(5) requires the Engineer to specify in the construction (contract) documents the CVN toughness criteria for weld metal (and base metal). Where notch toughness of welds used in elements of structural steel or their connections (welded joints) is required, Section 2.2.2 requires the Engineer to specify in the construction (contract) documents the minimum absorbed energy and corresponding test temperature for the filler metal (e.g., prequalified) or to specify that the WPS shall be qualified by CVN tests.

The effect of these provisions in AWS D1.1 is that the standard specifies CVN impact testing for qualification of welded joints to meet specified requirements for minimum CVN toughness. The standard does not prevent a prequalified WPS from being qualified to meet requirements for minimum CVN toughness but verification is only possible through review of the WPS. Section 3.1 of the standard requires all prequalified welding procedure specifications to be written. This requirement means that prequalified welding procedure specifications are available for submittal to the building official. Where there are requirements for minimum CVN toughness, requiring the submittal of welding procedure specifications or equivalent documents (see below) to the building official will enable the building official to verify whether the welded joints meet the applicable requirements for CVN toughness.

Given the discussion above on the provisions in AWS D1.8 and D1.1, it would appear that the submittal of welding procedure specifications is needed to verify CVN toughness where required by Section A4.4a or A4.4b of AISC 341-10. AISC 341-10, however, presents another approach. Section J2 contains requirements for documents to be submitted or made available to the engineer of record. Section J2.1 requires the submittal of welding procedure specifications (Item 1); certificates of conformance from the manufacturer for electrodes, fluxes and shielding gases (Item 2); and, for demand critical welds, applicable manufacturer's certifications that the filler metal meets supplemental notch toughness requirements (Item 3). Given these requirements and for consistency with Section 1704.2.5.2 and other sections of the 2012 IBC, the submittal of certificates of compliance instead of welding procedure specifications is specified in Items 3 and 4. Note that Section J2 does not specify that the documents required to be submitted or made available to the engineer of record are also required to be submitted or made available to the authority having jurisdiction (building official).

Item 5 is added to new Section 1704.5 because of the requirement in Section A3.1c of AISC 360-10 (Section A3.1c of AISC 360-05) for minimum Charpy V-notch (CVN) toughness of heavy structural steel shapes (e.g., with flange thicknesses greater than 2 inches) and meeting several conditions specified therein. Section A3.1c requires the construction documents (structural design documents) to specify that such shapes shall be supplied with CVN impact test results in accordance with ASTM A6, Supplementary Requirement S30. The requirement for test results means that test reports are available for submittal to the building official. Requiring their submittal to the building official will enable the building official to verify whether the structural steel meets applicable requirements for CVN toughness.

Item 6 is added to new Section 1704.5 to enable the building official to verify that fabrication of the steel buckling-restrained braces, where it is conducted at a location other than the construction site, was performed in accordance with the building code, its referenced standards (e.g., AISC 341) and the approved construction documents. Otherwise, special inspection at the fabricator's shop should be conducted (see IBC Section 1704.2.5).

Note that separate proposals:

1. Transfer the requirements of Section 1705.12.1 to new Section 1704.5;
2. Add additional requirements for submittals that are related to the welding of concrete reinforcement and anchor bolts;
3. Add additional requirements for submittals that are related to masonry; and
4. Add a new Section 107.1.1 that correlates with this proposal.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] 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, 2012.

1704.5 (NEW) #2-S-BRAZIL.doc

Public Hearing Results

Note: For staff analysis of the content of ASTM A 6 relative to CP#28, Section 3.6, please visit:
http://www.iccsafe.org:8888/cs/codes/Documents/2012-13cycle/Proposed-A/00a_updates.pdf

Committee Action: Disapproved

Committee Reason: Disapproval is consistent with the committee's action on S118-12 and S133-12. There's concern that requiring these submittals before the start of construction could delay the construction process.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Philip Brazil, P.E., S.E., representing self, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1704.5 Submittals to the building official. In addition to the submittal of reports of *special inspections* and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner's authorized agent to the *building official* ~~after review and acceptance by a *registered design professional* and prior to the construction or work being performed~~ for each of the following:

1. Test reports verifying compliance with Supplementary Requirement S30 of ASTM A6 for ~~W-shaped and WT-shaped elements~~ <u>hot-rolled shapes</u> of structural steel with flange thicknesses of 1-1/2 inches (38 mm) or greater that are required to have a Charpy V-notch toughness as specified in ~~Section A3.3 of~~ AISC 341;

2. Test reports verifying compliance with Supplementary Requirement S5 of ASTM A6 for structural steel plates of 2 inches (51 mm) in thickness or greater that are required to have a Charpy V-notch toughness as specified in ~~Section A3.3 of~~ AISC 341;
3. *Certificates of compliance* for verification that welds at elements of structural steel and their connections that are in the *seismic force-resisting system* are made with filler metal having a Charpy V-notch toughness as specified in ~~Section A3.3a of~~ AISC 341;
4. *Certificates of compliance* for verification that demand critical welds are made with filler metal having a Charpy V-notch toughness as specified in ~~Section A3.3b of~~ AISC 341;
5. Test reports verifying compliance with Supplementary Requirement S30 of ASTM A6 for hot-rolled shapes of structural steel with flange thicknesses greater than 2 inches (51 mm) that are required to have a Charpy V-notch toughness as specified in ~~Section A3.1c of~~ AISC 360;
6. *Certificates of compliance* for the fabrication of steel buckling-restrained braces on the premises of an *approved fabricator* in accordance with Section 1704.2.5.2.

(Portions of proposal not shown remains unchanged)

Commenter's Reason: In response to the Committee Reason and the testimony at the Dallas Code Development Hearing, the language for review and acceptance by a registered design professional and submittal prior to the construction or work being performed is deleted from the charging text. Also, the section references in the items are deleted to eliminate the need to correlate the standards with the IBC in the future. The language in each item is sufficiently descriptive to make the section reference unnecessary.

In Item #1, "W-shaped and WT-shaped" is changed to "hot-rolled" for consistency with Item #5 and with the corresponding sections in the standards (Section A3.3 of AISC 341-10 and Section A3.1c of AISC 360-10).

The section references in Items #3 and #4 of the original proposal were incorrect but are being deleted in the public comment. They should have been Sections A3.4a and A3.4b, respectively.

Note that a separate proposal changes "the owner" to "the owner or the owner's authorized agent" throughout the IBC (S90-12-AS).

S134-12

Final Action: AS AM AMPC____ D

S135-12
1704.5 (New)

Proposed Change as Submitted

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing self (pbrazil@reidmiddleton.com)

Add new text as follows:

1704.5 Submittals to the building official. In addition to the submittal of reports of *special inspections* and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner's authorized agent to the *building official* after review and acceptance by a *registered design professional* and prior to the construction or work being performed for each of the following:

1. Reports of preconstruction tests for masonry where the prism test method of Section 2105.2.2 is used to determine the *compressive strength of masonry* in accordance with Section 1.19.3 of TMS 402/ACI 530/ASCE 5.
2. Reports of preconstruction tests of grout where the unit strength method of Section 2105.2.2 is used to determine the *compressive strength of masonry* in accordance with Section 1.19.3 of TMS 402/ACI 530/ASCE 5.

Reason: This proposal is a continuation of a separate proposal that adds a new Section 1704.5 specifying submittals to the building official. This proposal adds two items to those in the separate proposal and the charging language in new Section 1704.5 is identical in both proposals.

The items are added to new Section 1704.5 because Section 1.19.3 of TMS 402/ACI 530/ASCE 5 requires compliance with a Level C quality assurance program for engineered masonry in structures classified as Risk Category IV. Table 1.19.3 for Level C quality assurance requires the verification of the specified compressive strength of masonry, f_m, prior to construction. Section 1.19.6.2 requires the compressive strength of masonry to be determined in accordance with TMS 602/ACI 530.1/ASCE 6. Article 1.4.B.1 of TMS 602/ACI 530.1/ASCE 6 requires the determination to be done by the unit strength method or the prism test method. Determination by the prism test method is, therefore, not required but when it is chosen for the verification of f_m prior to construction it requires testing of compressive strength in accordance with ASTM C 1314 (Article 1.4.B.3), which becomes a preconstruction test. Item 1 is added because of this. When the unit strength method is chosen for the same purpose, the grout is required to be tested for compressive strength in accordance with ASTM C 1019 (Article 1.4.B.2b (3b), which also becomes a preconstruction test. Item 2 is added because of this. In each case, requiring the submittal of test reports to the building official will enable the building official to verify, before construction begins, the validity of structural design assumptions based on the success of the preconstruction tests.

Neither TMS 402/ACI 530/ASCE 5 nor TMS 602/ACI 530.1/ASCE 6 specifies submittals to applicable regulatory officials (e.g., building official or authority having jurisdiction). In TMS 402/ACI 530/ASCE 5, Section 1.19.4 requires the quality assurance program to set forth the procedures for reporting and review, and Item 1 in Tables 1.19.2 (Level B Quality Assurance) and 1.19.3 (Level C Quality Assurance) specifies verification of compliance with the approved submittals ("approved" is not defined in Section 1.6, Definitions). In TMS 602/ACI 530.1/ASCE 6, (1) Section 1.5.A specifies that written acceptance of submittals be obtained prior to use of the materials or methods requiring acceptance; (2) Section 1.5.B specifies the submittals; (3) Section 1.2 defines "acceptable/accepted" as being done by the architect/engineer and "architect/engineer" as the individual or firm that issues, or administers the work under, the drawings and specifications ("approved" is not defined); and (4) Sections 1.6.A and 1.6.B specify the services and duties of testing agencies and inspection agencies, respectively, including requirements for the owner to retain the agencies and the agencies to report results and submit final reports to the architect/engineer and contractor.

Note that separate proposals:

1. Transfer the requirements of Section 1705.12.1 to new Section 1704.5 ;
2. Add additional requirements for submittals that are related to structural steel ;
3. Add additional requirements for submittals that are related to the welding of concrete reinforcement and anchor bolts and
4. Add a new Section 107.1.1 that correlates with this proposal.

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

1704.5 (NEW) #3-S-BRAZIL.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: Disapproval is consistent with the committee's action on S118-12, 133-12 and 134-12. There's concern that requiring these submittals before the start of construction could delay the construction process.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Philip Brazil, P.E., S.E., representing self, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1704.5 Submittals to the building official. In addition to the submittal of reports of *special inspections* and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner's authorized agent to the *building official* ~~after review and acceptance by a *registered design professional* and prior to the construction or work being performed~~ for each of the following:

~~1.~~ Reports of preconstruction tests ~~for masonry where the prism test method of Section 2105.2.2 is~~ <u>in accordance with Section 1.19.3 of TMS 402/ACI 530/ASCE 5</u> used to determine the *compressive strength of masonry* ~~in accordance with Section 1.19.3 of TMS 402/ACI 530/ASCE 5~~.

~~2. Reports of preconstruction tests of grout where the unit strength method of Section 2105.2.2 is used to determine the *compressive strength of masonry* in accordance with Section 1.19.3 of TMS 402/ACI 530/ASCE 5.~~

Commenter's Reason: In response to the Committee Reason and the testimony at the Dallas Code Development Hearing, the language for review and acceptance by a registered design professional and submittal prior to the construction or work being performed is deleted from the charging text. Also, the section references in the items are deleted to eliminate the need to correlate the standards with the IBC in the future. The language in each item is sufficiently descriptive to make the section reference unnecessary.

The public comment also consolidates the items from the original proposal into a single item. The instances where preconstruction testing is used to determine compressive strength of masonry may involve masonry prisms, the masonry units and grout, or only the masonry units. This depends upon the type of masonry, the design methodology and whether the unit strength method or the prism test method is selected.

The reference to Section 1.19.3 of TMS 402/ACI 530/ASCE 5 is retained rather than replacing it with language sufficiently descriptive to make the section reference unnecessary. Section 1.19.3 of TMS 402/ACI 530/ASCE 5 is applicable to Level C quality assurance programs for masonry in structures assigned to Risk Category IV and designed in accordance with chapters of TMS 402/ACI 530/ASCE 5 other than Chapter 5, 6 or 7, which is too cumbersome.

Note that a separate proposal changes "the owner" to "the owner or the owner's authorized agent" throughout the IBC (S90-12-AS).

S135-12
Final Action: AS AM AMPC____ D

S136-12
1704.5 (New), 1705.3.1, 1705.12.1

Proposed Change as Submitted

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1704.5 Submittals to the building official. In addition to the submittal of reports of *special inspections* and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner's authorized agent to the *building official* after review and acceptance by a *registered design professional* and prior to the construction or work being performed for each of the following:

1. Reports of material properties verifying compliance with the requirements of AWS D1.4 for weldability as specified in Section 3.5.2 of ACI 318 for reinforcing bars in concrete complying with a standard other than ASTM A 706 that are to be welded; and
2. Reports of mill tests in accordance with Section 21.1.5.2 of ACI 318 for reinforcing bars complying with ASTM A 615 and used to resist earthquake-induced flexural or axial forces in the special moment frames, special structural walls, or coupling beams connecting special structural walls, of *seismic force-resisting systems* in *structures* assigned to *Seismic Design Category* B, C, D, E or F.

1705.3.1 Materials. In the absence of sufficient data or documentation providing evidence of conformance to quality standards for materials in Chapter 3 of ACI 318, the building official shall require testing of materials in accordance with the appropriate standards and criteria for the material in Chapter 3 of ACI 318. ~~Weldability of reinforcement, except that which conforms to ASTM A 706, shall be determined in accordance with the requirements of Section 3.5.2 of ACI 318.~~

~~**1705.12.1 Concrete reinforcement.** Where reinforcement complying with ASTM A 615 is used to resist earthquake induced flexural and axial forces in special moment frames, special structural walls and coupling beams connecting special structural walls, in structures assigned to *Seismic Design Category* B, C, D, E or F, the reinforcement shall comply with Section 21.1.5.2 of ACI 318. Certified mill test reports shall be provided for each shipment of such reinforcement. Where reinforcement complying with ASTM A 615 is to be welded, chemical tests shall be performed to determine weldability in accordance with Section 3.5.2 of ACI 318.~~

Reason: This proposal is a continuation of a separate proposal that adds a new Section 1704.5 specifying submittals to the building official. This proposal adds two items to those in the separate proposal and the charging language in new Section 1704.5 is identical in both proposals.

The requirement in Section 1705.12.1 to provide certified mill test reports for reinforcement in special moment frames, special structural walls and coupling beams is relocated to Item 2 of new Section 1704.5 because the subject of Section 1705.12 is testing and qualification for seismic resistance but there is no testing specified in Section 1705.12.1. The submittal of certified mill test reports is specified but there is no corresponding requirement in ACI 318-11 that the reports be certified or that the act of submittal amounts to a "qualification." Also ACI 318 has consistently specified "mill tests" since the alternative to reinforcement complying with ASTM A 706 first appeared in the 1983 edition. The limitation in Section 1705.12.1 to reinforcement complying with ASTM A 615 is retained in Item 2 for consistency with the same limitation in the referenced section of ACI 318-11 (Section 21.1.5.2).

Relocating the requirement in Section 1705.12.1 to Item 2 of new Section 1704.5 has an additional benefit that is provided by the charging language in the new section. Section 1705.12.1 requires mill test reports to be provided with each shipment of reinforcement but that does not ensure the reports will be available to the owner, design team, construction team or building official. New Section 1704.5, however, requires the owner or authorized agent to submit the reports to the building official after review and acceptance by a registered design professional and prior to the construction or work begin performed. Also, the current requirement in Section 1705.12.1 that the reports be provided for each shipment means that they are available for submittal to the building official.

The charging language in Section 21.1.5.2 of ACI 318-11 specifies deformed reinforcement but Item 2 specifies reinforcing bars for consistency with (1) the basic requirement in Section 21.1.5.2 for compliance with ASTM A 706, which is limited in scope to "deformed and plain low-alloy steel bars…for concrete reinforcement" (Section 1.1), and (2) the alternative of compliance with ASTM

A 615, which is limited in scope to "deformed and plain carbon steel bars for concrete reinforcement," provided the special requirements of Section 21.1.5.2 are also met.

The source document for some of the language in Section 1705.12.1 is the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (Section 3.4.1.2 of FEMA 368 and Section 2.4.1.2 of FEMA 450-1).

In Item 1 of new Section 1704.5, the requirement in the last sentence of Section 1705.1.2.1 for chemical tests of reinforcement complying with ASTM A 615 that is to be welded is replaced with a requirement to submit reports of material properties for reinforcing bars complying with a standard other than ASTM A 706 that verify compliance with the requirements of AWS D1.4 for weldability. These changes correct several errors. First, the current language in Section 1705.1.2.1 is limited in scope to Seismic Design Categories B through F by that section, and to Seismic Design Categories C through F by the charging language in Section 1705.12 (Item 1), but verification of weldability is not a seismic issue. Verifying weldability is important for concrete reinforcement designed to resist all load effects, not merely seismic load effects.

Second, the current language in Section 1705.1.2.1 requires chemical tests of reinforcement be performed to determine weldability in accordance with Section 3.5.2 of ACI 318 but Section 3.5.2 of ACI 318 does not require chemical tests to be performed. Instead, it requires the ASTM specification to be supplemented by specifying a "report of material properties."

Third, Section 1705.12.1 requires the chemical tests for reinforcement complying with ASTM A 615 but Section 3.5.2 of ACI 318 specifies the report of material properties for reinforcement complying with a standard other than ASTM A 706. In ACI 318-11, specified standards other than ASTM A 615 and A 706 include A 955, A 996 and A 1035 (see Section 3.5.3.1).

Fourth, Section 1705.12.1 specifies concrete reinforcement but Section 3.5.2 of ACI 318 specifies reinforcing bars, which is done to exclude other types of concrete reinforcement such as plain reinforcement, headed shear studs, structural steel, steel pipe and steel tubing. Refer to Section 3.5, and the definition of "reinforcement" in Section 2.2, in ACI 318-11 for further information.

The language in Item 1 of new Section 1704.5 is consistent with the provisions in Section 3.5.2 of ACI 318 as discussed above. Section 3.5.2 of ACI 318 has consistently specified (1) a report of material properties, (2) a standard other than ASTM A 706 and (3) reinforcing bars, ever since the section first appeared in the 1977 edition. Section 3.5.2 also requires the applicable ASTM specifications for reinforcing bars to be "supplemented to require a report of material properties necessary to conform to the requirements in AWS D1.4." The requirement means that reports of material properties are available for submittal to the building official. Requiring their submittal to the building official will enable the building official to verify whether the reinforcing bars meet the applicable requirements for weldability.

For Items 1 and 2, neither ACI 318-11 nor ACI 301 ("Specifications for Structural Concrete," not an IBC referenced standard) specifies submittals to applicable regulatory officials (e.g., building official or authority having jurisdiction). In ACI 318, (1) Section 1.2.2 specifies the filing of calculations pertinent to the design with the contract documents when required by the building official, (2) Section 1.3.1 specifies inspection as required by the legally adopted general building code, and (3) Sections 1.3.2 through 1.3.4 specify requirements for the keeping and retention of inspection records, but (4) reports of mill tests and material properties are not included. In ACI 301-05, (1) Section 1.5.1 specifies that submittals required by the standard be submitted for review and acceptance; (2) Section 1.2 defines "submitted" as being provided to the architect/engineer for review **or** acceptance and "architect/engineer" as the individual or firm that issues the project drawings and specifications **or** administers the work under the contract documents ("approved" is not defined); (3) Section 1.5.2 specifies reporting by the testing agency of test results to the owner, architect/engineer and contractor; and (4) Section 1.6.2 specifies requirements for testing agencies, including acceptance by the architect/engineer before performing any work.

Note that Section 1.3.4 of AWS D1.4-98 requires the calculation of carbon equivalent for all reinforcing bars, including those complying with ASTM A 706. If mill test reports are not available to enable the calculation, chemical analysis is permitted to be performed. If the chemical composition is not known, special preheat temperatures are required (see Section 1.3.4.3).

Also, the likely source document for the current requirement to perform chemical tests, the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (Section 3.4.1.3 of FEMA 368 and Section 2.4.1.3 of FEMA 450-1) did not require chemical tests to be performed. It required verification "that chemical tests have been performed to determine weldability in accordance with Section 3.5.2 of ACI 318."

Note that separate proposals:

1. Add additional requirements for submittals that are related to structural steel (Sxx-12/13);
2. Add additional requirements for submittals that are related to the welding of concrete reinforcement and anchor bolts (Sxx-12/13);
3. Add additional requirements for submittals that are related to masonry (Sxx-12/13); and
4. Add a new Section 107.1.1 that correlates with this proposal (Sxx-12/13).

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

1704.5 (NEW) #4-S-BRAZIL.doc

Public Hearing Results

Committee Action: Disapproved

Committee Reason: This proposal would add inspection requirements that could delay the construction process.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Philip Brazil, P.E., S.E. representing self; and Lee Kranz, City of Bellevue, representing Washington Association of Building Officials, Technical Code Development Committee, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1704.5 Submittals to the building official. In addition to the submittal of reports of *special inspections* and tests in accordance with Section 1704.2.4, reports and certificates shall be submitted by the owner or the owner's authorized agent to the *building official* ~~after review and acceptance by a *registered design professional* and prior to the construction or work being performed~~ for each of the following:

1. Reports of material properties verifying compliance with the requirements of AWS D1.4 for weldability as specified in Section 3.5.2 of ACI 318 for reinforcing bars in concrete complying with a standard other than ASTM A 706 that are to be welded; and
2. Reports of mill tests in accordance with Section 21.1.5.2 of ACI 318 for reinforcing bars complying with ASTM A 615 and used to resist earthquake-induced flexural or axial forces in the special moment frames, special structural walls, or coupling beams connecting special structural walls, of *seismic force-resisting systems* in *structures* assigned to *Seismic Design Category* B, C, D, E or F.

(Portions of proposal not shown remain unchanged)

Commenter's Reason: In response to the Committee Reason and the testimony at the Dallas Code Development Hearing, the language for review and acceptance by a registered design professional and submittal prior to the construction or work being performed is deleted from the charging text.

In contrast with the public comments on Proposals S133-12, S134-12 and S135-12, the section references in the items are not deleted to be consistent with the current language in IBC Sections 1705.3.1 and 1705.12.1, which specify the section references.

Note that a separate proposal changes "the owner" to "the owner or the owner's authorized agent" throughout the IBC (S90-12-AS).

S136-12

Final Action:	AS	AM	AMPC____	D

S137-12
1704.5.1, 1705.11, 1705.11.7, 1905.1.8, 2209.1

Proposed Change as Submitted

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing Washington Association of Building Officials, Technical Code Development Committee (pbrazil@reidmiddleton.com)

Revise as follows:

1704.5.1 Structural observations for seismic resistance. Structural observations shall be provided for those structures assigned to *Seismic Design Category* D, E or F where one or more of the following conditions exist:

1. The structure is classified as *Risk Category* III or IV in accordance with Table 1604.5.
2. The height of the structure is greater than 75 feet (22 860 mm) above the base as defined in Section 11.2 of ASCE 7.
3. The structure is assigned to *Seismic Design Category* E, is classified as *Risk Category* I or II in Accordance with Table 1604.5, and is greater than two *stories above grade plane*.
4. When so designated by the *registered design professional* responsible for the structural design.
5. When such observation is specifically required by the *building official*.

1705.11 Special inspections for seismic resistance. *Special inspections* itemized in Sections 1705.11.1 through 1705.11.8, unless exempted by the exceptions of Section 1704.2, are required for the following:

1. The seismic force-resisting systems in structures assigned to *Seismic Design Category* C, D, E or F in accordance with Sections 1705.11.1 through 1705.11.3, as applicable.
2. Designated seismic systems in structures assigned to *Seismic Design Category* C, D, E or F in accordance with Section 1705.11.4.
3. Architectural, mechanical and electrical components in accordance with Sections 1705.11.5 and 1705.11.6.
4. Storage racks as defined in Section 11.2 of ASCE 7 that are in structures assigned to *Seismic Design Category* D, E or F in accordance with Section 1705.11.7.
5. Seismic isolation systems in accordance with Section 1705.11.8.

   Exception: Special inspections itemized in Sections 1705.11.1 through 1705.11.8 are not required for structures designed and constructed in accordance with one of the following:

   1. The structure consists of light-frame construction; the design spectral response acceleration at short periods, *SDS*, as determined in Section 1613.3.4, does not exceed 0.5; and the building height of the structure does not exceed 35 feet (10 668 mm).
   2. The seismic force-resisting system of the structure consists of reinforced masonry or reinforced concrete; the design spectral response acceleration at short periods, *SDS*, as determined in Section
      1613.3.4, does not exceed 0.5; and the building height of the structure does not exceed 25 feet (7620 mm).
   3. The structure is a detached one- or two-family dwelling not exceeding two *stories above grade plane* and does not have any of the following horizontal or vertical irregularities in accordance with Section 12.3 of ASCE 7:
      3.1. Torsional or extreme torsional irregularity.
      3.2. Nonparallel systems irregularity.
      3.3. Stiffness-soft story or stiffness-extreme soft story irregularity.

3.4. Discontinuity in lateral strength-weak story irregularity.

1705.11.7 Storage racks. Periodic *special inspection* is required during the anchorage of storage racks as defined in Section 11.2 of ASCE 7 that are 8 feet (2438 mm) or greater in height in structures assigned to *Seismic Design Category* D, E or F.

Revise as follows:

1905.1.8 ACI 318, Section 22.10. Delete ACI 318, Section 22.10, and replace with the following:

22.10 - Plain concrete in structures assigned to Seismic Design Category C, D, E or F.

22.10.1 - Structures assigned to Seismic Design Category C, D, E or F shall not have elements of structural plain concrete, except as follows:

(a) *Structural plain concrete basement, foundation or other walls below the base as defined in Section 11.2 of ASCE 7 are permitted in detached one- and two-family dwellings three stories or less in height constructed with stud-bearing walls. In dwellings assigned to Seismic Design Category D or E, the height of the wall shall not exceed 8 feet (2438 mm), the thickness shall not be less than 71/2 inches (190 mm), and the wall shall retain no more than 4 feet (1219 mm) of unbalanced fill. Walls shall have reinforcement in accordance with 22.6.6.5.*

(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.*

Revise as follows:

2209.1 Storage racks. The design, testing and utilization of ~~industrial steel~~ storage racks as defined in Section 11.2 of ASCE 7 and 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 the provisions of Section 15.5.3 of ASCE 7, except that the mapped acceleration parameters, S_s and S_1, shall be determined in accordance with Section 1613.3.1.

Reason: The purpose for the proposal is to clarify the meaning of "base" and "storage rack," which are defined in ASCE 7-10 but are not also defined in the building code. Both of these terms have meanings that necessitate knowing their definitions to fully understand the technical provisions related to them. Therefore, the proposal adds references to Section 11.2 of ASCE 7-10 for their definitions. The only instances of these terms in the 2012 IBC where they are directly related to their corresponding definitions in ASCE 7-10 are in this proposal.

For storage racks, adding a reference to the definition in ASCE 7-10 in Section 1705.11.7 also has the effect of narrowing the scope to those that are defined. Note that "storage rack" is defined in ASCE 7-10 as including "industrial pallet racks, moveable shelf racks and stacker racks made of cold-formed or hot-rolled structural members;" but excluding "other types of racks such as drive-in and drive-through racks, cantilever racks, portable racks or racks made of materials other than steel."

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

1704.5.1-S-BRAZIL.doc

Public Hearing Results

Committee Action: Approved as Modified

Modify proposal as follows:

1704.5.1 Structural observations for seismic resistance. Structural observations shall be provided for those structures assigned to *Seismic Design Category* D, E or F where one or more of the following conditions exist:

1. The structure is classified as *Risk Category* III or IV in accordance with Table 1604.5.
2. The height of the structure is greater than 75 feet (22 860 mm) above the base as defined in ~~Section 11.2 of~~ ASCE 7.
3. The structure is assigned to *Seismic Design Category* E, is classified as *Risk Category* I or II in Accordance with Table 1604.5, and is greater than two *stories above grade plane*.
4. When so designated by the *registered design professional* responsible for the structural design.
5. When such observation is specifically required by the *building official*.

1705.11 Special inspections for seismic resistance. *Special inspections* itemized in Sections 1705.11.1 through 1705.11.8, unless exempted by the exceptions of Section 1704.2, are required for the following:

1. The seismic force-resisting systems in structures assigned to *Seismic Design Category* C, D, E or F in accordance with Sections 1705.11.1 through 1705.11.3, as applicable.
2. Designated seismic systems in structures assigned to *Seismic Design Category* C, D, E or F in accordance with Section 1705.11.4.
3. Architectural, mechanical and electrical components in accordance with Sections 1705.11.5 and 1705.11.6.
4. Storage racks as defined in ~~Section 11.2 of~~ ASCE 7 that are in structures assigned to *Seismic Design Category* D, E or F in accordance with Section 1705.11.7.
5. Seismic isolation systems in accordance with Section 1705.11.8.

   Exception: Special inspections itemized in Sections 1705.11.1 through 1705.11.8 are not required for structures designed and constructed in accordance with one of the following:

   1. The structure consists of light-frame construction; the design spectral response acceleration at short periods, *SDS*, as determined in Section 1613.3.4, does not exceed 0.5; and the building height of the structure does not exceed 35 feet (10 668 mm).
   2. The seismic force-resisting system of the structure consists of reinforced masonry or reinforced concrete; the design spectral response acceleration at short periods, *SDS*, as determined in Section 1613.3.4, does not exceed 0.5; and the building height of the structure does not exceed 25 feet (7620 mm).
   3. The structure is a detached one- or two-family dwelling not exceeding two *stories above grade plane* and does not have any of the following horizontal or vertical irregularities in accordance with Section 12.3 of ASCE 7:
      3.1. Torsional or extreme torsional irregularity.
      3.2. Nonparallel systems irregularity.
      3.3. Stiffness-soft story or stiffness-extreme soft story irregularity.
      3.4. Discontinuity in lateral strength-weak story irregularity.

1705.11.7 Storage racks. Periodic *special inspection* is required during the anchorage of storage racks as defined in ~~Section 11.2 of~~ ASCE 7 that are 8 feet (2438 mm) or greater in height in structures assigned to *Seismic Design Category* D, E or F.

1905.1.8 ACI 318, Section 22.10. Delete ACI 318, Section 22.10, and replace with the following:

22.10 - Plain concrete in structures assigned to Seismic Design Category C, D, E or F.

22.10.1 - Structures assigned to Seismic Design Category C, D, E or F shall not have elements of structural plain concrete, except as follows:

(a) Structural plain concrete basement, foundation or other walls below the base as defined in ~~Section 11.2 of~~ ASCE 7 are permitted in detached one- and two-family dwellings three stories or less in height constructed with stud-bearing walls. In dwellings assigned to Seismic Design Category D or E, the height of the wall shall not exceed 8 feet (2438 mm), the thickness shall not be less than 71/2 inches (190 mm), and the wall shall retain no more than 4 feet (1219 mm) of unbalanced fill. Walls shall have reinforcement in accordance with 22.6.6.5.

(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.*

2209.1 Storage racks. The design, testing and utilization of storage racks as defined in ~~Section 11.2~~ of ASCE 7 and 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 the provisions of Section 15.5.3 of ASCE 7, except that the mapped acceleration parameters, S_s and S_1, shall be determined in accordance with Section 1613.3.1.

Committee Reason: This code change clarifies structural terms that rely on definitions in ASCE 7, The modification deletes the specific section references to make the code text easier to maintain.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Bonnie E. Manley, American Iron and Steel Institute, representing Rack Manufacturers Institute, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

<u>**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, and 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.</u>

1705.11 Special inspections for seismic resistance. *Special inspections* itemized in Sections 1705.11.1 through 1705.11.8, unless exempted by the exceptions of Section 1704.2, are required for the following:

1. The seismic force-resisting systems in structures assigned to *Seismic Design Category* C, D, E or F in accordance with Sections 1705.11.1 through 1705.11.3, as applicable.
2. Designated seismic systems in structures assigned to *Seismic Design Category* C, D, E or F in accordance with Section 1705.11.4.
3. Architectural, mechanical and electrical components in accordance with Sections 1705.11.5 and 1705.11.6.
4. <u>*Storage racks*</u> ~~Storage racks as defined in ASCE 7~~ that are in structures assigned to *Seismic Design Category* D, E or F in accordance with Section 1705.11.7.
5. Seismic isolation systems in accordance with Section 1705.11.8.

Exception: Special inspections itemized in Sections 1705.11.1 through 1705.11.8 are not required for structures designed and constructed in accordance with one of the following:

1. The structure consists of light-frame construction; the design spectral response acceleration at short periods, *SDS*, as determined in Section 1613.3.4, does not exceed 0.5; and the building height of the structure does not exceed 35 feet (10 668 mm).
2. The seismic force-resisting system of the structure consists of reinforced masonry or reinforced concrete; the design spectral response acceleration at short periods, *SDS*, as determined in Section 1613.3.4, does not exceed 0.5; and the building height of the structure does not exceed 25 feet (7620 mm).
3. The structure is a detached one- or two-family dwelling not exceeding two *stories above grade plane* and does not have any of the following horizontal or vertical irregularities in accordance with Section 12.3 of ASCE 7:
   3.1. Torsional or extreme torsional irregularity.
   3.2. Nonparallel systems irregularity.
   3.3. Stiffness-soft story or stiffness-extreme soft story irregularity.
   3.4. Discontinuity in lateral strength-weak story irregularity.

1705.11.7 Storage racks. Periodic *special inspection* is required during the anchorage of *storage racks* ~~storage racks as defined in ASCE 7~~ that are 8 feet (2438 mm) or greater in height in structures assigned to *Seismic Design Category* D, E or F.

2209.1 Storage racks. The design, testing and utilization of *storage racks* ~~storage racks as defined in ASCE 7 and~~ 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 the provisions of Section 15.5.3 of ASCE 7, except that the mapped acceleration parameters, S_s and S_1, shall be determined in accordance with Section 1613.3.1.

Commenter's Reason: It doesn't make sense to send a user to ASCE 7 to find the definition for storage racks. Currently, ASCE 7-10 includes the following definition for storage racks:

> **STORAGE RACKS:** Include industrial pallet racks, moveable shelf racks, and stacker racks made of cold-formed or hot-rolled structural members. Does not include other types of racks such as drive-in and drive-through racks, cantilever racks, portable racks, or racks made of materials other than steel.

Originally, this ASCE 7 definition was sourced from the scope of the 2008 edition of RMI/ANSI MH 16.1. Proposal S243-12, which was approved as submitted, adopts the 2012 edition of RMI/ANSI MH 16.1, which states the following in the scope:

> **1.1 SCOPE**
> This Specification and companion Commentary (hereinafter referred to as the Specification) applies to industrial steel storage racks, movable-shelf racks, rack-supported systems and automated storage and retrieval systems (stacker racks) made of cold-formed or hot-rolled steel structural members. Such rack types also include push-back rack, pallet-flow rack, case-flow rack, pick modules, and rack-supported platforms. This Specification is intended to be applied to the design of the storage rack portion of any rack structure that acts as support for the exterior walls and roof, except as noted. It does not apply to other types of racks, such as drive-in or drive-through racks, cantilever racks, portable racks, or to racks made of material other than steel.

By approving Proposal S137-12, the ICC Structural Code Committee has indicated a desire to source a clear definition for storage racks. Rather than send the user outside of the IBC, our recommendation is to bring the most up-to-date definition into the IBC. Therefore, this public comment introduces a definition to Section 202 for storage racks, which is based upon the 2012 edition of RMI/ANSI MH 16.1, and deletes the references to ASCE 7 in Sections 1705.11(4), 1705.11.7 and 2209.1.

S137-12
Final Action: AS AM AMPC____ D

S145-12

1705.2.2, Table 1705.2.2, 1705.2.2.1.1, 1705.5, Table 1705.5 (New), 1705.10.1, 1705.10.2, 1705.11.2, 1705.11.3

Proposed Change as Submitted

Proponent: D. Kirk Harman, The Harman Group, representing The National Council of Structural Engineers Associations (NCSEA) Code Advisory Committee, Quality Assurance and Special Inspection Subcommittee.

Revise as follows:

1705.2.2 Steel construction other than structural steel. Special inspection for steel construction other than structural steel shall be in accordance with Table 1705.2.2 and this section.

Exceptions:

1. *Special inspection* of cold-formed steel light-frame construction for buildings and structures in Risk Category I shall not be required.
2. *Special inspection* of cold-formed steel light-frame construction for buildings and structures in Risk Category II that are 3 stories or less in height above grade plane and that are not included in Sections 1705.10 or 1705.11, shall not be required.

TABLE 1705.2.2
REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]
1. Material verification of cold-formed steel deck and cold-formed steel light-frame construction:			
a. Identification markings to conform to ASTM standards specified in the approved construction documents.	—	X	Applicable ASTM material standards
b. Manufacturer's certified test reports.	—	X	
2. Inspection of welding:			
a. Cold-formed steel deck and cold-formed steel light-frame construction:			
1) Floor and roof deck welds.	—	X	AWS D1.3
2) Cold-formed steel light-frame construction welds.	---	X	AWS D1.3
b. Reinforcing steel:			
1) Verification of weldability of reinforcing steel other than ASTM A 706.	—	X	AWS D1.4 ACI 318: Section 3.5.2

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]
2) Reinforcing steel resisting flexural and axial forces in intermediate and special moment frames, and boundary elements of special structural walls of concrete and shear reinforcement.	X	—	
3) Shear reinforcement.	X	—	
4) Other reinforcing steel.	—	X	
3. Inspection of cold-formed steel light-frame construction including framing, shear walls, diaphragms and shear panels for conformance with the *approved construction documents:*			
a. Inspect member locations and sizes.		X	
b. Inspect bracing, strap bracing, drag strut and stiffener locations and sizes.		X	
c. Verify mechanical connectors including screws, powder actuated fasteners, bolts, anchor bolts, hold downs, anchors and other fastening components.		X	Applicable ASTM Standards
d. Inspect material thickness, grade and fastening of diaphragms, and sheathing for the lateral force resisting system.		X	
e. Inspect connections including plates and components; screw quantity, size and spacing; powder actuated fastener quantity size and location; bolt size and location; anchor bolt size, spacing and location; hold down size location and configuration; beam hangers and framing.		X	

For SI: 1 inch = 25.4 mm.

a. Where applicable, see also Section 1705.10 Special inspections for wind resistance and Section 1705.11, Special inspections for seismic resistance.

1705.2.2.1.1 Cold-formed steel. Welding inspection and welding inspector qualification for cold-formed steel floor and roof decks and cold-formed steel light-frame construction shall be in accordance with AWS D1.3.

1705.5 Wood construction. *Special inspections* of the fabrication process of prefabricated wood structural elements and assemblies shall be in accordance with Section 1704.2.5. *Special inspections* of site-built assemblies shall be in accordance with this section and Table 1705.5.

Exceptions:

1. *Special inspection* of wood construction for buildings and structures in Risk Category I shall not be required.
2. *Special inspection* of wood construction for buildings and structures in Risk Category II that are 3 stories or less in height above grade plane and that are not included in Sections 1705.10 or 1705.11 shall not be required.

TABLE 1705.5
REQUIRED VERIFICATION AND INSPECTION OF WOOD CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]
1. Inspection of wood construction including framing, shear walls, diaphragms and shear panels for conformance with the *approved construction documents:*			
a. Verify grade stamp on framing lumber, plywood and OSB.		X	
b. Inspect wood framing including layout, member sizes, blocking, bridging and bearing lengths.		X	
c. Verify mechanical connectors including screws, powder actuated fasteners, bolts, anchor bolts, hold downs, anchors and other fastening components.		X	Applicable ASTM Standards
d. Inspect diaphragms, shear walls and wood structural panel sheathing size and thickness; sizes of framing members at adjoining panel edges and nail or staple size and spacing.		X	

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]
e. Inspect wood connections including plates and components; nail quantity, size and spacing; bolt size and location; anchor bolt size, spacing and location; hold down size location and configuration; beam hangers and framing.		X	

a. Where applicable, see Section 1705.10, Special inspections for wind resistance and Section 1705.11, Special inspections for seismic resistance.

1705.10.1 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 components within the main windforce-resisting system, including wood shear walls, wood diaphragms, drag struts, braces and hold-downs.

Exception: For buildings and structures in Risk Category I or II that are 3 stories or less in height above grade plane, *special inspection* is not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other components of the main wind-force-resisting system, where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center.

1705.10.2 Cold-formed steel light-frame construction. Periodic special inspection is required during welding operations of elements of the main windforce-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the main windforce-resisting system, including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

Exception: For buildings and structures in Risk Category I or II and 3 stories or less in height above grade plane, *special inspection* is not required for cold- formed steel light-frame shear walls, braces, diaphragms, collectors (drag struts) and hold-downs where either of the following apply:

1. The sheathing is gypsum board or fiberboard.
2. The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c.).

1705.11.2 Structural wood. Continuous special inspection is required during field gluing operations of elements of the seismic force-resisting system. Periodic special inspection is required for nailing, bolting, anchoring and other fastening of components within the seismic force-resisting system, including wood shear walls, wood diaphragms, drag struts, braces, shear panels and hold-downs.

Exception: For buildings and structures in Risk Category I or II and 3 stories or less in height above grade plane *special inspection* is not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other components of the seismic force-resisting system, where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c.).

1705.11.3 Cold-formed steel light-frame construction. Periodic special inspection is required during welding operations of elements of the seismic force-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the seismic force-resisting system, including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

Exception: For buildings and structures in Risk Category I or II and 3 stories or less in height above grade plane, s*pecial inspection* is not required for cold-formed steel light-frame shear walls, braces, diaphragms, collectors (drag struts) and hold-downs where either of the following apply:

1. The sheathing is gypsum board or fiberboard.
2. The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) o.c.

Reason: : NCSEA believes that light frame construction in wood and cold formed steel have become more commonly used for load bearing applications of significant height and in regions with moderate and high seismic and wind concerns. These types of construction should be subject to Special Inspections in a similar manner and to a comparable extent as other systems such as concrete, structural steel and masonry. There is a large group of buildings constructed with light frame construction that is not subject to the same requirements for Special Inspection as the same buildings constructed with structural steel, concrete or masonry. This proposal seeks to correct this deficiency in the Code.

This proposal provides requirements to be consistent across both wood and cold-formed steel systems to avoid any competitive advantage of one system over the other. This proposal will improve the consistency of special inspections across all of the major structural materials.

Exceptions are provided to limit the applicability of these provisions to exclude single and two family dwellings, small commercial, agricultural and buildings of lesser occupancies unless these minor structures are subject to the existing requirements of 1705.10 and 1705.11.

This proposal contains provisions addressing both wood frame and cold-formed steel light-frame construction together. This is an effort to address both systems in one change therefore avoiding any perception of one system having an advantage over the other regarding special inspection.

The proposed revisions to 1705.2 and 1705.5 improve the Special Inspection requirements for both wood and cold-formed steel light-frame construction in a manner consistent with Special Inspection requirements for structural steel, concrete and masonry.

The proposed revisions to 1705.10 and 1705.11 are to coordinate between the additional requirements for Special Inspections in high seismic and high wind conditions and the proposed provisions. The proposed changes to 1705.10 and 1705.11 do not reduce the requirements of these sections they only prevent the exceptions for these sections from conflicting with the new requirements. In addition, notes are added to the tables to refer to 1705.10 and 1705.11 for additional requirements.

There will be no increase in construction cost due to the increased Special Inspection that will take place. Currently structural engineers provide for these inspections in project specifications. However, individual requirements vary greatly and there is not a consistent level of requirements. Standardization of these requirements in the Code will reduce delays and added costs due to confusion created by varying specifications. The improved field quality assurance will improve safety and reduce field errors resulting in a savings in construction cost and schedule. The improved public safety and potential reduction in construction cost support adoption of this proposal.

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

1705.2.2-SHARMAN.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee feels that the proposed expansion of special inspections for light-frame construction was not sufficiently justified as noted in numerous objections raised during testimony in opposition to the proposal.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment 1:

D. Kirk Harman, The Harman Group representing, The National Council of Structural Engineers Associations (NCSEA) Code Advisory Committee, Quality Assurance and Special Inspection Subcommittee, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1705.5 Wood construction. *Special inspections* of the fabrication process of prefabricated wood structural elements and assemblies shall be in accordance with Section 1704.2.5. *Special inspections* of site-built assemblies shall be in accordance with this section ~~and Table 1705.5~~.

~~**Exceptions:**~~

~~1. *Special inspection* of wood construction for buildings and structures in Risk Category I shall not be required.~~
~~2. *Special inspection* of wood construction for buildings and structures in Risk Category II that are 3 stories or less in height above grade plane and that are not included in Sections 1705.10 or 1705.11 shall not be required.~~

~~**TABLE 1705.5**~~
~~**REQUIRED VERIFICATION AND INSPECTION OF WOOD CONSTRUCTION**~~

~~VERIFICATION AND INSPECTION~~	~~CONTINUOUS~~	~~PERIODIC~~	~~REFERENCED STANDARD~~ a
~~1.Inspection of wood construction including framing, shear walls, diaphragms and shear panels for conformance with the *approved construction documents:*~~			
~~a.Verify grade stamp on framing lumber, plywood and OSB.~~		~~X~~	
~~b.Inspect wood framing including layout, member sizes, blocking, bridging and bearing lengths.~~		~~X~~	
~~c.Verify mechanical connectors including screws, powder actuated fasteners, bolts, anchor bolts, hold downs, anchors and other fastening components.~~		~~X~~	~~Applicable ASTM Standards~~
~~d.Inspect diaphragms, shear walls and wood structural panel sheathing size and thickness; sizes of framing members at adjoining panel edges and nail or staple size and spacing.~~		~~X~~	
~~e.Inspect wood connections including plates and components; nail quantity, size and spacing; bolt size and location; anchor bolt size, spacing and location; hold down size location and configuration; beam hangers and framing.~~		~~X~~	

~~a. Where applicable, see also Section 1705.10 Special inspections for wind resistance and Section 1705.11, Special inspections for seismic resistance.~~

1705.10.1 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 components within the main windforce-resisting system, including wood shear walls, wood diaphragms, drag struts, braces and hold-downs.

Exception: ~~For buildings and structures in Risk Category I or II that are 3 stories or less in height above grade plane,~~ S*pecial inspection* is not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other components of the main windforce-resisting system, where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center.

1705.10.2 Cold-formed steel light-frame construction. Periodic special inspection is required during welding operations of elements of the main windforce-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the main windforce-resisting system, including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

Exception: For buildings and structures in Risk Category I ~~or II,~~ or for buildings and structures in Risk Category II ~~and~~ that are 3 or less stories in height above grade plane, s*pecial inspection* is not required for cold-formed steel light-frame shear walls, braces, diaphragms, collectors (drag struts) and hold-downs where either of the following apply:

1. The sheathing is gypsum board or fiberboard.
2. The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c.).

1705.11.2 Structural wood. Continuous special inspection is required during field gluing operations of elements of the seismic force-resisting system. Periodic special inspection is required for nailing, bolting, anchoring and other fastening of components within the seismic force-resisting system, including wood shear walls, wood diaphragms, drag struts, braces, shear panels and hold-downs.

Exception~~: For buildings and structures in Risk Category I or II and 3 stories or less stories in height above grade plane~~ *Special inspection* is not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other components of the seismic force-resisting system, where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c.).

1705.11.3 Cold-formed steel light-frame construction. Periodic special inspection is required during welding operations of elements of the seismic force-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the seismic force-resisting system, including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

Exception: For buildings and structures in Risk Category I ~~or II~~ or for buildings and structures in Risk Category II ~~and~~ that are 3 or less stories in height above grade plane, s*pecial inspection* is not required for cold-formed steel light-frame shear walls, braces, diaphragms, collectors (drag struts) and hold-downs where either of the following apply:

1. The sheathing is gypsum board or fiberboard.
2. The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) o.c.

(Portions of proposal not shown remain unchanged)

Commenter's Reason: The proponent has submitted two Public Comments on this change in an attempt to address differing subject matter within a code section. NCSEA believes that light frame construction in wood and cold formed steel have become more commonly used for load bearing applications of significant height and in regions with moderate and high seismic and wind concerns. These types of construction should be subject to Special Inspections in a similar manner and to a comparable extent as other systems such as concrete, structural steel and masonry. There is a large group of buildings constructed with light frame construction that is not subject to the same requirements for Special Inspection as the same buildings constructed with structural steel, concrete or masonry. This proposal seeks to correct this deficiency in the Code.

The original proposal provided requirements to be consistent across both wood and cold-formed steel systems to avoid any competitive advantage of one system over the other. This proposal will improve the consistency of special inspections across all of the major structural materials. Opposition to the proposal was voice with regard to wood construction. This Public Comment has separated the two materials to be considered separately.

Exceptions are provided to limit the applicability of these provisions to exclude single and two family dwellings, small commercial, agricultural and buildings of lesser occupancies unless these minor structures are subject to the existing requirements of 1705.10 and 1705.11.

The proposed revisions improve the Special Inspection requirements for cold-formed steel light-frame construction in a manner consistent with Special Inspection requirements for structural steel, concrete and masonry.

The proposed revisions to 1705.10 and 1705.11 are to coordinate between the additional requirements for Special Inspections in high seismic and high wind conditions and the proposed provisions. The proposed changes to 1705.10 and 1705.11 do not reduce the requirements of these sections they only prevent the exceptions for these sections from conflicting with the new requirements. In addition, notes are added to the tables to refer to 1705.10 and 1705.11 for additional requirements.

There will be no increase in construction cost due to the increased Special Inspection that will take place. Currently structural engineers provide for these inspections in project specifications. However, individual requirements vary greatly and there is not a consistent level of requirements. Standardization of these requirements in the Code will reduce delays and added costs due to confusion created by varying specifications. The improved field quality assurance will improve safety and reduce field errors resulting in a savings in construction cost and schedule. The improved public safety and potential reduction in construction cost support adoption of this proposal.

The committee commented that insufficient justification was provided. The following table compares the Special Inspection requirements contained in the Code for a four story steel frame building, not in a high wind or high seismic condition, to the same building constructed using cold formed steel light frame construction. It demonstrates that there are forty five (45) different inspection tasks required for the structural steel building, none of which are required for the same building constructed with cold formed steel light frame.

Special Inspection Requirements Currently Contained in the Code

Special Inspection Tasks	**Structural Steel**	**Cold Formed Steel Light Frame**	**Task Number**
Review the material test reports	Required	Not Required	1
Submission of QA Reports	Required	Not Required	2
Inspection Tasks Prior to Welding			
Welding procedure specifications (WPSs) available	Required	Not Required	3
Manufacturer certifications for welding consumables available	Required	Not Required	4
Material identification (type/grade)	Required	Not Required	5
Welder identification system	Required	Not Required	6
Fit-up of welds (including joint geometry)			
Joint preparation	Required	Not Required	7
Dimensions and alignment	Required	Not Required	8
Cleanliness (condition of steel surfaces)	Required	Not Required	9
Tacking (tack weld quality and location)	Required	Not Required	10
Backing type and fit (if applicable)	Required	Not Required	11
Check welding equipment	Required	Not Required	12
Inspection Tasks During Welding			
Use of qualified welders	Required	Not Required	13
Control and handling of welding consumables			
Packaging	Required	Not Required	14
Exposure control	Required	Not Required	15
Environmental conditions			
Wind speed within limits	Required	Not Required	16
Precipitation and temperature	Required	Not Required	17
Welding Procedures Followed			
Settings on welding equipment	Required	Not Required	18
Technique	Required	Not Required	19
Selected welding materials	Required	Not Required	20
Inspection Tasks After Welding	Required	Not Required	21
Welds cleaned	Required	Not Required	22
Size, length and location of welds	Required	Not Required	23
Welds meet visual acceptance criteria	Required	Not Required	24
Crack prohibition	Required	Not Required	25
Repair activities	Required	Not Required	26
Document acceptance or rejection of welded joint or member	Required	Not Required	27
Inspection Tasks Prior to Fastening			
Manufacturer's certifications available for fastener materials	Required	Not Required	28
Fasteners marked in accordance with requirements	Required	Not Required	29
Proper fasteners selected for the joint detail	Required	Not Required	30
Proper fastening procedure selected for joint detail	Required	Not Required	31
Connecting elements, including the appropriate surface condition and hole preparation meet requirements.	Required	Not Required	32
Proper storage provided for fastener components	Required	Not Required	33
Inspection Tasks During Fastening			
Fastener assemblies, of suitable condition, placed in all locations and washers (if required) are positioned as required	Required	Not Required	34
Fastener installation technique	Required	Not Required	35
Inspection Tasks After Fastening			
Document acceptance or rejection of fastener connections	Required	Not Required	36
Inspection of Anchor Devices			

Compliance with Construction Documents	Required	Not Required	37
Diameter	Required	Not Required	38
Grade	Required	Not Required	39
Type Length of anchor	Required	Not Required	40
Depth of Embedment	Required	Not Required	41
Inspection of Steel Frame			
Braces	Required	Not Required	42
Stiffeners	Required	Not Required	43
Member Locations	Required	Not Required	44
Application of Joint Details at Each Connection	Required	Not Required	45

The above demonstrates the current level of Special Inspection required by the Code is seriously deficient for cold formed steel light frame construction when compared to structural steel. The same comparison can be made to concrete and masonry and the same conclusion will be reached.

It is unreasonable to expect the Building Official to undertake such exhaustive inspections. This level of inspection can only be achieved when incorporated into the Code requirements for Special Inspection. The safety of cold formed steel light frame buildings is in serious question when constructed without the requirements for inspections or Special Inspections contained in this proposal and comment.

We urge the Committee to approve Proposal S145 as modified by this Public Comment.

Public Comment 2:

D. Kirk Harman, The Harman Group representing, The National Council of Structural Engineers Associations (NCSEA) Code Advisory Committee, Quality Assurance and Special Inspection Subcommittee, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1705.2.2 Steel construction other than structural steel. Special inspection for steel construction other than structural steel shall be in accordance with Table 1705.2.2 and this section.

~~**Exceptions:**~~

1. ~~*Special inspection* of cold-formed steel light-frame construction for buildings and structures in Risk Category I shall not be required.~~
2. ~~*Special inspection* of cold-formed steel light-frame construction for buildings and structures in Risk Category II that are 3 stories or less in height above grade plane and that are not included in Sections 1705.10 or 1705.11, shall not be required.~~

TABLE 1705.2.2
REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]
1. Material verification of cold-formed steel deck ~~and cold-formed steel light-frame construction:~~			
a. Identification markings to conform to ASTM standards specified in the approved construction documents.	—	X	Applicable ASTM material standards
b. Manufacturer's certified test reports.	—	X	
2. Inspection of welding:			
a. Cold-formed steel deck ~~and cold-formed steel light-frame construction:~~			
1) Floor and roof deck welds.	—	X	AWS D1.3
~~2) Cold-formed steel light-frame construction welds.~~	~~---~~	~~X~~	~~AWS D1.3~~
b. Reinforcing steel:			

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]
1) Verification of weldability of reinforcing steel other than ASTM A 706.	—	X	AWS D1.4 ACI 318: Section 3.5.2
2) Reinforcing steel resisting flexural and axial forces in intermediate and special moment frames, and boundary elements of special structural walls of concrete and shear reinforcement.	X	—	
3) Shear reinforcement.	X	—	
4) Other reinforcing steel.	—	X	
~~3. Inspection of cold-formed steel light-frame construction including framing, shear walls, diaphragms and shear panels for conformance with the *approved construction documents:*~~			
~~f. Inspect member locations and sizes.~~		~~X~~	
~~g. Inspect bracing, strap bracing, drag strut and stiffener locations and sizes.~~		~~X~~	
~~h. Verify mechanical connectors including screws, powder actuated fasteners, bolts, anchor bolts, hold downs, anchors and other fastening components.~~		~~X~~	~~Applicable ASTM Standards~~
~~i. Inspect material thickness, grade and fastening of diaphragms, and sheathing for the lateral force resisting system.~~		~~X~~	
~~j. Inspect connections including plates and components; screw quantity, size and spacing; powder actuated fastener quantity size and location; bolt size and location; anchor bolt size, spacing and location; hold down size location and configuration; beam hangers and framing.~~		~~X~~	

For SI: 1 inch = 25.4 mm.

a. Where applicable, see also ~~Section 1705.10 Special inspections for wind resistance and~~ Section 1705.11, Special inspections for seismic resistance.

1705.2.2.1.1 Cold-formed steel. Welding inspection and welding inspector qualification for cold-formed steel floor and roof decks ~~and cold-formed steel light-frame construction~~ shall be in accordance with AWS D1.3.

1705.10.1 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 components within the main windforce-resisting system, including wood shear walls, wood diaphragms, drag struts, braces and hold-downs.

Exception: For buildings and structures in Risk Category I, or for buildings and structures in Risk Category II that are 3 or less stories ~~or less~~ in height above grade plane, *special inspection* is not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other components of the main wind-force-resisting system, where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center.

1705.10.2 Cold-formed steel light-frame construction. Periodic special inspection is required during welding operations of elements of the main windforce-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the main windforce-resisting system, including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

Exception: ~~For buildings and structures in Risk Category I or II and 3 stories or less in height above grade plane, S~~S*pecial inspection* is not required for cold- formed steel light-frame shear walls, braces, diaphragms, collectors (drag struts) and hold-downs where either of the following apply:

1. The sheathing is gypsum board or fiberboard.
2. The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c.).

1705.11.2 Structural wood. Continuous special inspection is required during field gluing operations of elements of the seismic force-resisting system. Periodic special inspection is required for nailing, bolting, anchoring and other fastening of components within the seismic force-resisting system, including wood shear walls, wood diaphragms, drag struts, braces, shear panels and hold-downs.

Exception: For buildings and structures in Risk Category I, or for buildings and structures in Risk Category II ~~and~~ that are 3 ~~stories~~ or less stories in height above grade plane *special inspection* is not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other components of the seismic force-resisting system, where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c.).

1705.11.3 Cold-formed steel light-frame construction. Periodic special inspection is required during welding operations of elements of the seismic force-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the seismic force-resisting system, including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

Exception: ~~For buildings and structures in Risk Category I or II and 3 stories or less in height above grade plane,~~ S*pecial inspection* is not required for cold-formed steel light-frame shear walls, braces, diaphragms, collectors (drag struts) and hold-downs where either of the following apply:

1. The sheathing is gypsum board or fiberboard.
2. The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) o.c.

Reason: The proponent has submitted two Public Comments on this change in an attempt to address differing subject matter within a code section. NCSEA believes that light frame construction in wood and cold formed steel have become more commonly used for load bearing applications of significant height and in regions with moderate and high seismic and wind concerns. These types of construction should be subject to Special Inspections in a similar manner and to a comparable extent as other systems such as concrete, structural steel and masonry. There is a large group of buildings constructed with light frame construction that is not subject to the same requirements for Special Inspection as the same buildings constructed with structural steel, concrete or masonry. This proposal seeks to correct this deficiency in the Code.

The original proposal provided requirements to be consistent across both wood and cold-formed steel systems to avoid any competitive advantage of one system over the other. This proposal would improve the consistency of special inspections across all of the major structural materials. Opposition to the proposal was voiced with regard to wood construction. This Public Comment has separated the two materials to be considered separately.

Exceptions are provided to limit the applicability of these provisions to exclude single and two family dwellings, small commercial, agricultural and buildings of lesser occupancies unless these minor structures are subject to the existing requirements of 1705.10 and 1705.11.

The proposed revisions improve the Special Inspection requirements for wood light-frame construction in a manner consistent with Special Inspection requirements for structural steel, concrete and masonry.

The proposed revisions to 1705.10 and 1705.11 are to coordinate between the additional requirements for Special Inspections in high seismic and high wind conditions and the proposed provisions. The proposed changes to 1705.10 and 1705.11 do not reduce the requirements of these sections they only prevent the exceptions for these sections from conflicting with the new requirements. In addition, notes are added to the tables to refer to 1705.10 and 1705.11 for additional requirements.

There will be no increase in construction cost due to the increased Special Inspection that will take place. Currently structural engineers provide for these inspections in project specifications. However, individual requirements vary greatly and there is not a consistent level of requirements. Standardization of these requirements in the Code will reduce delays and added costs due to confusion created by varying specifications. The improved field quality assurance will improve safety and reduce field errors resulting in a savings in construction cost and schedule. The improved public safety and potential reduction in construction cost support adoption of this proposal.

The committee commented that insufficient justification was provided. The following table compares the Special Inspection requirements contained in the Code for a four story commercial steel frame building, not in a high wind or high seismic condition, to the same building constructed using wood light frame construction. It demonstrates that there are twenty (20) different inspection tasks required for the structural steel building, none of which are required for the same building constructed with wood light frame.

Special Inspection Requirements Currently Contained in the Code

Special Inspection Tasks	**Structural Steel**	**Wood Light Frame**	**Task Number**
Review the material test reports	Required	Not Required	1
Submission of QA Reports	Required	Not Required	2
Inspection Tasks Prior to Fastening			
Manufacturer's certifications available for fastener materials	Required	Not Required	3
Fasteners marked in accordance with requirements	Required	Not Required	4
Proper fasteners selected for the joint detail	Required	Not Required	5
Proper fastening procedure selected for joint detail	Required	Not Required	6
Connecting elements, including the appropriate surface condition and hole preparation meet requirements.	Required	Not Required	7
Proper storage provided for fastener components	Required	Not Required	8
Inspection Tasks During Fastening			
Fastener assemblies, of suitable condition, placed in all locations and washers (if required) are positioned as required	Required	Not Required	9
Fastener installation technique	Required	Not Required	10
Inspection Tasks After Fastening			
Document acceptance or rejection of fastener connections	Required	Not Required	11
Inspection of Anchor Devices			
Compliance with Construction Documents	Required	Not Required	12
Diameter	Required	Not Required	13
Grade	Required	Not Required	14
Type Length of anchor	Required	Not Required	15
Depth of Embedment	Required	Not Required	16
Inspection of Frame			
Braces	Required	Not Required	17
Stiffeners	Required	Not Required	18
Member Locations	Required	Not Required	19
Application of Joint Details at Each Connection	Required	Not Required	20

The above demonstrates the current level of Special Inspection required by the Code is seriously deficient for wood light frame construction when compared to structural steel. The same comparison can be made to concrete and masonry and the same conclusion will be reached.

It is unreasonable to expect the Building Official to undertake such exhaustive inspections. This level of inspection can only be achieved when incorporated into the Code requirements for Special Inspection. The safety of wood light frame buildings over 3 stories in height is in serious question when constructed without the requirements for inspections or Special Inspections contained in this proposal and comment.

We urge the Committee to approve Proposal S145 as modified by this Public Comment.

S145-12

Final Action: AS AM AMPC____ D

S146-12
Table 1705.2.2

Proposed Change as Submitted

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing myself (pbrazil@reidmiddleton.com)

Revise as follows:

TABLE 1705.2.2
REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]
1. Material verification of cold-formed steel deck:			
a. Identification markings to conform to ASTM standards specified in the approved construction documents.	—	X	Applicable ASTM material standards
b. Manufacturers' certified test reports.	—	X	
2. Inspection of welding:	—	—	
a. Cold-formed steel deck			
1. Floor and roof deck welds		X	AWS D1.3
b. Reinforcing steel:			
1. Verification of weldability of reinforcing steel other than ASTM A 706.	—	X	AWS D1.4 ACI 318 Section 3.5.2
2. Reinforcing steel resisting flexural and axial forces in intermediate and special moment frames, and boundary elements of special structural walls of concrete and shear reinforcement.	X	—	
3. Shear reinforcement.	X	—	
4. Other reinforcing steel.	—	X	
3. Installation of open web steel joists and joist girders in accordance with the approved construction documents and steel joist placement plans		X	

For SI: 1 inch = 25.4 mm.
a. Where applicable, see also Section 1705.11, Special inspection for seismic resistance.

Reason: The purpose for this proposal is to require special inspections for the installation of open web steel joists and joist girders. Their structural design is sufficiently complex to warrant inspection from a person with the expertise of a special inspector who is approved by the building official as having the competence necessary to inspect the installation of the joists. Refer to the definitions of "special inspection" and "special inspector" for further information. Examples of the complexity of the structural design that warrant special inspection of the installation are the bearing seat attachments, field splices and bridging attachments.

The standard specifications for open web steel joists (SJI-K-2010 and SJI-LH/DLH-2010), joist girders (SJI-JG-2010) and composite steel joists (SJI-CJ-2010) by the Steel Joist Institute contain provisions for inspections but these are limited to inspections by the manufacturer before shipment to verify compliance and workmanship with the requirements of the specifications. Refer to Section 5.12 of SJI-K-2010, Section 104.13 of SJI-LH/DLH-2010, Section 1004.10 of SJI-JG-2010 and Section 104.13 of SJI-CJ-2010. The sections of the SJI standards noted above are also referenced in Section 4 of the codes of standard practice for steel joists and joist girders (no identifier) and composite steel joists (SJI-CJCOSP-2010). The identifiers cited above match those from the published documents but they are abbreviated in Chapter 35 of the 2012 IBC to K-10, LH/LDH-10, JG-10 and CJ-10, respectively; and are specified as SJI-K-1.1, SJI-LH/LDH-1.1, SJI-JG-1.1 and SJI-CJ-1.0, respectively, in Section 2207.1. Note that the codes of standard practice published by the Steel Joist Institute are not referenced standards of the 2012 IBC.

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

T1705.2.2 #1-S-BRAZIL.doc

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

TABLE 1705.2.2
REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]
1. Material verification of cold-formed steel deck:			
a. Identification markings to conform to ASTM standards specified in the approved construction documents.	–	X	Applicable ASTM material standards
b. Manufacturers' certified test reports.	–	X	
2. Inspection of welding:	–	–	
a. Cold-formed steel deck			
1. Floor and roof deck welds		X	AWS D1.3
b. Reinforcing steel:			
1. Verification of weldability of reinforcing steel other than ASTM A 706.	–	X	AWS D1.4 ACI 318 Section 3.5.2
2. Reinforcing steel resisting flexural and axial forces in intermediate and special moment frames, and boundary elements of special structural walls of concrete and shear reinforcement.	X	–	
3. Shear reinforcement.	X	–	
4. Other reinforcing steel.	–	X	
3. Installation of open web steel joists and joist girders ~~in accordance with the approved construction documents and steel joist placement plans~~		~~X~~	
a. End connections – welding or bolted		X	SJI – Standard Specification
b. Bridging – horizontal or diagonal		X	SJI – Standard Specification

For SI: 1 inch = 25.4 mm.
a. Where applicable, see also Section 1705.11, Special inspection for seismic resistance.

Committee Reason: The committee believes that the installation of joist and joist girders warrants special inspection. The modification provides specificity on these inspections and removed the reference to steel joist placement plans.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Bonnie E. Manley, American Iron and Steel Institute, representing Steel Joist Institute, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1705.2.2 Open web steel joists and joist girders. Special inspections of open web steel joists and joist girders shall be in accordance with Table 1705.2.2

TABLE 1705.2.2 REQUIRED SPECIAL INSPECTIONS OF COLD-FORMED STEEL DECK, REINFORCING STEEL AND OPEN WEB STEEL JOISTS AND JOIST GIRDERS ~~VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL~~

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]
1. Material verification of cold-formed steel deck:			
a. Identification markings to conform to ASTM standards specified in the approved construction documents.	—	X	Applicable ASTM material standards
b. Manufacturers' certified test reports.	—	X	
2. inspection of welding: a. Cold-formed steel deck:			
1) Floor and roof deck welds.	—	X	AWS D1.3
a. Reinforcing steel:			
1) Verification of weldability of reinforcing steel other than ASTM A 706.	—	X	AWS D1.4 or ACI 318: Section 3.5.2
2) Reinforcing steel-resisting flexural and axial forces in intermediate and special moment frames, and boundary elements of special reinforced concrete shear walls and shear reinforcement.	X	—	
3) Shear reinforcement.	X	—	
4) Other reinforcing steel.	—	X	
3. Installation of open web steel joists and joist girders			
a. End connections – welded or bolted		X	**~~SJI – Standard Specification~~SJI specifications listed in Section 2207.1.**
b. Bridging – horizontal or diagonal		~~X~~	**~~SJI – Standard Specification~~**
1. Standard Bridging		X	**SJI specifications listed in Section 2207.1.**
2. Bridging that differs from the SJI specifications listed in Section 2207.1		X	

Commenter's Reason: The purpose of public comment is twofold. First, it fully charges the new special inspection requirements for open web steel joists and joist girders by adding a new Section 1705.2.2 and correctly identifying this type of construction in the title of Table 1705.2.2. Please note that Proposal S142-12 deletes the cold-formed steel deck provisions in Table 1705.2.2 and Proposal S144-12 deletes the reinforcing steel provisions in Table 1705.2.2. Both proposals were approved as submitted. Consequently, the change to the title is not intended to reintroduce these references, but rather to make sure that, when the dust settles, the title of Table 1705.2.2 correctly reads: "Required Special Inspections of Open Web Steel Joists and Joist Girders."

The second purpose of this public comment is to modify the text in Table 1705.2.2 to reflect the editorial changes successfully made in Proposal S240-12. That proposal, which was approved as modified, eliminated the generic reference to "SJI – Standard Specifications" in favor of the more accurate "SJI specifications listed in Section 2207.1". Proposal S240-12 also better clarified the difference between "standard bridging" and "bridging that differs from the SJI specifications listed in Section 2207.1". This language needs to be accurately reflected in this Table as well.

S146-12

Final Action: AS AM AMPC____ D

S150-12
1705.3

Proposed Change as Submitted

Proponent: Stephen Kerr, S.E., Josephson Werdowatz and Associates, representing Structural Engineers Association of California (SEAOC) (skerr@jwa-se.com)

Revise as follows:

1705.3 Concrete construction. The *special inspections* and verifications for concrete construction shall be as required by this section and Table 1705.3. The following exceptions shall not apply where Section 1705.10 or 1705.11 invoke *special inspections* or where special inspection of column anchor bolts for structural steel lateral force resisting frames is required by Section 1705.11.1.

Exception: *Special inspections* shall not be required for:

~~1. Isolated spread concrete footings of buildings three stories or less above *grade plane* that are fully supported on earth or rock.~~

~~2~~1. *Isolated spread concrete footings and* continuous concrete footings supporting walls of buildings three stories or less above *grade plane* that are fully supported on earth or rock and where any of the following conditions apply:

~~2.1.~~ 1.1 The footings support walls of light-frame construction;

~~2.2.~~ 1.2 The footings are designed in accordance with Table 1809.7; or

~~2.3.~~ 1.3 The structural design of the footing is based on a specified compressive strength, f'_c, no greater than 2,500 pounds per square inch (psi) (17.2 MPa), regardless of the compressive strength specified in the *construction documents* or used in the footing construction.

~~3~~2. 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).

~~4~~3. Concrete foundation walls constructed in accordance with Table 1807.1.6.2.

~~5~~4. Concrete patios, driveways and sidewalks, on grade.

Reason: Special inspections for concrete include such items as proper mix, reinforcing steel, bolts installed in concrete, post-installed anchors, formwork, concrete placement, curing, etc. Under Exception 1, the building could be of any type (concrete, masonry, steel, light frame), utilize high-strength concrete, and have heavily-loaded "isolated" footings. This change proposal makes the exception for isolated spread footings subject to the same limitations as those for continuous footings.

Note also that there are no additional inspection requirements for concrete under 1705.10 (wind), 1705.11 (seismic) and 1705.12 (testing for seismic). Therefore, anchorage elements such as anchor bolts for holdowns or steel frames used in the lateral system would not require special inspection when used in conjunction with light-frame construction or at isolated footings. The proposed change ensures that, when special inspection for light-frame construction is required by Section 1705.10 or 1705.11, the placement of anchor bolts will require special inspection, and that the placement of anchor bolts for steel frames resisting seismic loads will also require special inspection.

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

1705.3-S-KERR.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee felt that the proposed revisions to special inspection of concrete construction have possibilities, but as written the proposal needs work. There is a preference for keeping the first exception for isolated footings. Also there's concern that the additional limitations would require concrete testing for some nonstructural slab on grade construction.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Stephen Ker S.E. - Structural Engineers Association of California and Philip Brazil, P.E., S.E., representing self, request Approval as Modified by this Public Comment.

Modify the proposal as follows:

1705.3 Concrete construction. The *special inspections* and verifications for concrete construction shall be as required by this section and Table 1705.3. ~~The following exceptions shall not apply where Section 1705.10 or 1705.11 invoke *special inspections* or where special inspection of column anchor bolts for structural steel lateral force resisting frames is required by Section 1705.11.1.~~

Exception: Special inspections shall not be required for:

1. ~~Isolated spread concrete footings and~~ Concrete footings supporting walls of buildings three stories or less above *grade plane* that are fully supported on earth or rock and ~~where any of the following conditions apply~~ <u>meet one of the following requirements</u>:
   1.1. The footings support walls of light-frame construction~~;~~
   1.2. The footings are designed in accordance with Table 1809.7~~; or~~
   1.3. The structural design of the footing is based on a specified compressive strength, f'_c, no greater than 2,500 pounds per square inch (psi) (17.2 Mpa), regardless of the compressive strength specified in the *construction documents* or used in the footing construction~~.~~
2. 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).
3. Concrete foundation walls constructed in accordance with Table 1807.1.6.2.
4. Concrete patios, driveways and sidewalks, on grade.

Commenter's Reason: Special inspections for concrete include such items as proper mix, reinforcing steel, bolts installed in concrete, post-installed anchors, formwork, concrete placement, curing, etc. Under the existing Exception 1, the building could be of any type (concrete, masonry, steel, light frame), utilize high-strength concrete, and have eccentrically or heavily-loaded "isolated" footings. This change proposal makes the exception for isolated spread footings subject to the same limitations as those for continuous footings.

S150-12
Final Action: AS AM AMPC____ D

S156-12

1705.10.1, 1705.10.1.1 (New), 1705.10.1.2 (New), 1705.10.1.3 (New), 1705.10.2, 1705.10.2.1 (New), 1705.10.2.2 (New), 1705.11.2, 1705.10.11.2.1 (New), 1705.10.11.2.2 (New), 1705.11.2.3 (New), 1705.11.3, 1705.11.3.1 (New), 1705.11.3.2 (New)

Proposed Change as Submitted

Proponent: Stephen Kerr, S.E., Structural Engineers Association of California (skerr@jwa-se.com)

Revise as follows:

1705.10.1 Structural wood. Special inspection for wood construction within the main windforce-resisting system shall be as required by this section. Special inspection for wood construction in accordance with this section shall also be provided where vertical elements of the main windforce-resisting system are comprised of other materials, such as steel frames and concrete or masonry shear walls. ~~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 components within the main windforce-resisting system, including wood shear walls, wood diaphragms, drag struts, braces, and hold-downs.~~

> ~~**Exception:** *Special inspection* is not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other components of the main windforce-resisting system where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center.~~

1705.10.1.1 Field gluing operations. Continuous special inspection is required during field gluing operations of wood elements of the main windforce-resisting system.

1705.10.1.2 Shear walls. Periodic special inspection shall be required for the sheathing fastening, and for other connections within the shear wall. Such connections shall include hold-down or tie-down connections, sill plate and sole plate anchorage and connections, and connections between the top of the wall and the horizontal diaphragm above.

> **Exception:** Special inspection for wood shear walls is not required where the sheathing is gypsum board or fiberboard or where the fastener spacing along shear wall sheathing edges is more than 4 inches (102 mm) on center,

1705.10.1.3 Horizontal diaphragms. Periodic special inspection shall be required for the sheathing fastening, diaphragm chord connections and splices, and collector connections and fastening.

> **Exception:** Special inspection for horizontal wood diaphragms is not required where the sheathing is gypsum board or fiberboard or where the least fastener spacing along sheathing edges or diaphragm boundaries is more than 4 inches (102 mm) on center.

1705.10.2 Cold-formed steel light-frame construction. Special inspection for cold-formed light-frame construction within the main windforce-resisting system shall be as required by this section. Special inspection for cold-formed light-frame construction in accordance with this section shall be provided where vertical elements of the main windforce-resisting system are comprised of other materials, such as steel frames and concrete or masonry shear walls. ~~Periodic special inspection is required during welding operations of elements of the main windforce-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the main windforce-resisting system, including shear walls, braces, diaphragms, collectors (drag struts), and hold-downs.~~

~~**Exception:** *Special inspection* is not required for cold-formed steel light-frame shear walls, braces, diaphragms, collectors (drag struts) and hold-downs where either of the following apply:~~

1. ~~The sheathing is gypsum board or fiberboard.~~
2. ~~The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c).~~

1705.10.2.1 Shear walls and strap-braced walls. Periodic special inspection shall be required for the sheathing fastening, the welding or screw attachment of the strap bracing, and for other connections within the shear wall or strap-braced wall. Such connections shall include hold-down or tie-down connections, bottom track anchorage and connections, and connections between the top of the wall and the horizontal diaphragm above.

Exception: Special inspection for cold-formed light-frame shear walls is not required where either of the following apply:

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, and the fastener spacing along sheathing edges is more than 4 inches (102 mm) on center.

1705.10.2.2 Horizontal diaphragms. Periodic special inspection shall be required for the sheathing fastening, welding or screw attachment of diagonal strap bracing, diaphragm chord connections and splices, and collector connections and fastening.

Exception: Special inspection for cold-formed light-frame horizontal diaphragms is not required where either of the following apply:

1. The sheathing is gypsum board or fiberboard.
2. The sheathing is wood structural panel or steel sheets on only one side of the framing, and the least fastener spacing along sheathing edges or diaphragm boundaries is more than 4 inches (102 mm) on center.

1705.11.2 Structural wood. Special inspection for wood construction within the seismic force-resisting system shall be as required by this section. Special inspection for wood construction in accordance with this section shall be provided where vertical elements of the seismic force-resisting system are comprised of other materials, such as steel frames and concrete or masonry shear walls.~~Continuous special inspection is required during field gluing operations of elements of the seismic force-resisting system. Periodic special inspection is required for nailing, bolting, anchoring and other fastening of components within the seismic force-resisting system, including wood shear walls, wood diaphragms, drag struts, braces, shear panels and hold-downs.~~

~~**Exception:** *Special inspection* is not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other components of the main seismic force-resisting system where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c).~~

1705.11.2.1 Field gluing operations. Continuous special inspection shall be required during field gluing operations of wood elements of the seismic force-resisting system.

1705.11.2.2 Shear walls. Periodic special inspection shall be required for the sheathing fastening, and for other connections within the shear wall. Such connections shall include hold-down or tie-down connections, sill plate and sole plate anchorage and connections, and connections between the top of the wall and the horizontal diaphragm above.

Exception: Special inspection for wood shear walls is not required where the sheathing is gypsum board or fiberboard or where fastener spacing along shear wall sheathing edges is more than 4 inches (102 mm) on center,

1705.11.2.3 Horizontal diaphragms. Periodic special inspection shall be required for the sheathing fastening, diaphragm chord connections and splices, and collector connections and fastening.

Exception: Special inspection for horizontal wood diaphragms is not required where the sheathing is gypsum board or fiberboard or where least fastener spacing along sheathing edges or diaphragm boundaries is more than 4 inches (102 mm) on center.

1705.11.3 Cold-formed steel light-frame construction. Special inspection for cold-formed light-frame construction within the seismic force-resisting system shall be as required by this section. Special inspection for cold-formed light-frame construction in accordance with this section shall be provided where vertical elements of the seismic force-resisting system are comprised of other materials, such as steel frames and concrete or masonry shear walls~~Periodic special inspection is required during welding operations of elements of the seismic force-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the seismic force-resisting system, including shear walls, braces, diaphragms, collectors (drag struts), and hold-downs.~~

~~**Exception:** *Special inspection* is not required for cold-formed steel light-frame shear walls, braces, diaphragms, collectors (drag struts) and hold-downs where either of the following apply:~~

1. ~~The sheathing is gypsum board or fiberboard.~~
2. ~~The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c).~~

1705.11.3.1 Shear walls and strap-braced walls. Periodic special inspection shall be required for the sheathing fastening, the welding or screw attachment of the strap bracing, and for other connections within the shear wall or strap-braced wall. Such connections shall include hold-down or tie-down connections, bottom track anchorage and connections, and connections between the top of the wall and the horizontal diaphragm above.

Exception: Special inspection for cold-formed light-frame shear walls is not required where either of the following apply:

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, and the fastener spacing along sheathing edges is more than 4 inches (102 mm) on center.

1705.11.3.2 Horizontal diaphragms. Periodic special inspection shall be required for the sheathing fastening, welding or screw attachment of diagonal strap bracing, diaphragm chord connections and splices, and collector connections and fastening.

Exception: Special inspection for cold-formed light-frame horizontal diaphragms is not required where either of the following apply:

1. The sheathing is gypsum board or fiberboard.
2. The sheathing is wood structural panel or steel sheets on only one side of the framing, and the least fastener spacing along sheathing edges or diaphragm boundaries is more than 4 inches (102 mm) on center.

Reason: As currently written, it is not clear how to apply the exceptions to special inspection for wind and seismic as applicable to wood framing and cold-formed steel light frame construction (together "light-frame construction). The exceptions use "fastener spacing of the sheathing" as the trigger for special inspection. However, the following aspects of light-frame construction are not covered adequately by the exception language:

1. Fastener spacing for shear walls could vary throughout the building. It is not clear that the exception would only be applicable to the particular shear wall or diaphragm with the larger fastening spacing, and to the other elements of the lateral force-resisting system associated with that shear wall or diaphragm.
2. The main elements of the lateral force-resisting system of light-frame buildings are the shear walls and the horizontal diaphragms. Elements associated with the shear walls include hold-downs, and the parts use to make connection to the foundation or the horizontal diaphragms, including sill plates, sole plates, bottom tracks, and blocking and framing clips. Elements associated with the horizontal diaphragms include chords, collectors, and elements used to anchor concrete and masonry walls for out-of-plane forces (such as blocking, straps, and hold-down hardware used horizontally. As written, it is not clear when special inspection would be required for the elements associated with the shear walls and diaphragms.
3. Shear wall sheathing is fastened at the sheathing edges, and in the middle of the panel. It is not clear that the reference to sheathing fastening is intended to apply to fastening along sheathing edges.
4. Diaphragm sheathing fastening is often specified with different spacing at sheathing edges, and at diaphragm boundaries. It is not clear what fastening (edge or boundary) is being referred to, or what portions of a horizontal diaphragm and associated elements would be affected by the exception.
5. Buildings of pre-dominantly light-frame construction often use vertical lateral force-resisting elements made up of other materials, such as steel frames, or concrete shear walls or masonry shear walls. It is not clear under what conditions special inspection would be required for the elements used to connect such vertical lateral force-resisting elements to the light-frame building system.
6. Light-frame diaphragms are often used in buildings where all of the vertical lateral force-resisting elements are made up of other materials, such concrete tilt-up shear or masonry shear walls. It is not clear under what conditions special inspection would be required for the wood, light-frame, and/or steel elements used to anchor the concrete or masonry walls for out-of plane forces.

The proposed change includes similar revisions to the provisions for structural wood, and for cold-formed light-frame construction.

Shear walls and horizontal diaphragms are handled separately and the elements associated with each are identified. This makes it clear, once the special inspection is triggered (by fastener spacing, double sided sheathing, or the use of strap bracing) which elements other than the sheathing fastening, require inspection.

The requirements for inspection of anchorage elements in horizontal diaphragms for out-of-plane support of concrete and masonry walls are made explicit.

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

1705.10.1-S-KERR.doc

Public Hearing Results

Committee Action: Disapproved

Committee Reason: This code change seems to add confusion to wood and cold-formed steel inspection, rather than clarifying them. As written this would actually change the current requirements. There's some concern of unintended consequences. There was specific concern that "within the MWFRS" should be changed to "of the MWFRS".

Assembly Action: None

Individual Consideration Agenda

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

Public Comment 1:

Stephen Kerr representing Structural Engineers Association of California, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1705.10.1 Structural wood. Special inspection for wood construction within the main windforce-resisting system shall be as required by this section. ~~Special inspection for wood construction in accordance with this section shall also be provided where vertical elements of the main windforce-resisting system are comprised of other materials, such as steel frames and concrete or masonry shear walls.~~

1705.10.1.1 Field gluing operations. Continuous special inspection is required during field gluing operations of wood elements of the main windforce-resisting system.

1705.10.1.2 Shear walls. Periodic special inspection shall be required for the sheathing fastening, and for other connections within the shear wall. Such connections shall include hold-down or tie-down connections, sill plate and sole plate anchorage and connections, and connections between the top of the wall and the horizontal diaphragm above.

> **Exception:** Special inspection for wood shear walls is not required where the sheathing is gypsum board or fiberboard or where ~~the~~ specified fastener spacing along shear wall sheathing edges is more than 4 inches (102 mm) on center,

1705.10.1.3 Horizontal diaphragms. Periodic special inspection shall be required for the sheathing fastening, diaphragm chord connections and splices, and collector connections and fastening.

> **Exception:** Special inspection for horizontal wood diaphragms is not required where the sheathing is gypsum board ~~or fiberboard~~ or where the least specified fastener spacing along sheathing edges or diaphragm boundaries is more than 4 inches (102 mm) on center.

1705.10.2 Cold-formed steel light-frame construction. Special inspection for cold-formed light-frame construction within the main windforce-resisting system shall be as required by this section. ~~Special inspection for cold-formed light-frame construction in accordance with this section shall be provided where vertical elements of the main windforce-resisting system are comprised of other materials, such as steel frames and concrete or masonry shear walls~~.

1705.10.2.1 Shear walls and strap-braced walls. Periodic special inspection shall be required for the sheathing fastening, the welding or screw attachment of the strap bracing, and for other connections within the shear wall or strap-braced wall. Such connections shall include hold-down or tie-down connections, bottom track anchorage and connections, and connections between the top of the wall and the horizontal diaphragm above.

> **Exception:** Special inspection for cold-formed light-frame shear walls is not required where either of the following apply:
>
> 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, and the specified fastener spacing along sheathing edges is more than 4 inches (102 mm) on center.

1705.10.2.2 Horizontal diaphragms. Periodic special inspection shall be required for the sheathing fastening, welding or screw attachment of diagonal strap bracing, diaphragm chord connections and splices, and collector connections and fastening.

> **Exception:** Special inspection for cold-formed light-frame horizontal diaphragms is not required where either of the following apply:
>
> 1. The sheathing is gypsum board ~~or fiberboard~~.
> 2. The sheathing is wood structural panel or steel sheets on only one side of the framing, and the least specified fastener spacing along sheathing edges or diaphragm boundaries is more than 4 inches (102 mm) on center.

1705.11.2 Structural wood. Special inspection for wood construction within the seismic force-resisting system shall be as required by this section. ~~Special inspection for wood construction in accordance with this section shall be provided where vertical elements of the seismic force-resisting system are comprised of other materials, such as steel frames and concrete or masonry shear walls.~~

1705.11.2.1 Field gluing operations. Continuous special inspection shall be required during field gluing operations of wood elements of the seismic force-resisting system.

1705.11.2.2 Shear walls. Periodic special inspection shall be required for the sheathing fastening, and for other connections within the shear wall. Such connections shall include hold-down or tie-down connections, sill plate and sole plate anchorage and connections, and connections between the top of the wall and the horizontal diaphragm above.

> **Exception:** Special inspection for wood shear walls is not required where the sheathing is gypsum board or fiberboard or where specified fastener spacing along shear wall sheathing edges is more than 4 inches (102 mm) on center,

1705.11.2.3 Horizontal diaphragms. Periodic special inspection shall be required for the sheathing fastening, diaphragm chord connections and splices, and collector connections and fastening.

> **Exception:** Special inspection for horizontal wood diaphragms is not required where the sheathing is gypsum board ~~or fiberboard~~ or where least specified fastener spacing along sheathing edges or diaphragm boundaries is more than 4 inches (102 mm) on center.

1705.11.3 Cold-formed steel light-frame construction. Special inspection for cold-formed light-frame construction within the seismic force-resisting system shall be as required by this section. ~~Special inspection for cold-formed light-frame construction in accordance with this section shall be provided where vertical elements of the seismic force-resisting system are comprised of other materials, such as steel frames and concrete or masonry shear walls.~~

1705.11.3.1 Shear walls and strap-braced walls. Periodic special inspection shall be required for the sheathing fastening, the welding or screw attachment of the strap bracing, and for other connections within the shear wall or strap-braced wall. Such connections shall include hold-down or tie-down connections, bottom track anchorage and connections, and connections between the top of the wall and the horizontal diaphragm above.

Exception: Special inspection for cold-formed light-frame shear walls is not required where either of the following apply:

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, and the specified fastener spacing along sheathing edges is more than 4 inches (102 mm) on center.

1705.11.3.2 Horizontal diaphragms. Periodic special inspection shall be required for the sheathing fastening, welding or screw attachment of diagonal strap bracing, diaphragm chord connections and splices, and collector connections and fastening.

Exception: Special inspection for cold-formed light-frame horizontal diaphragms is not required where either of the following apply:

1. The sheathing is gypsum board ~~or fiberboard~~.
2. The sheathing is wood structural panel or steel sheets on only one side of the framing, and the least specified fastener spacing along sheathing edges or diaphragm boundaries is more than 4 inches (102 mm) on center.

Reason: As currently written, it is not clear how to apply the exceptions to special inspection for wind and seismic as applicable to wood framing and cold-formed steel light frame construction (together "light-frame construction). The exceptions use "fastener spacing of the sheathing" as the trigger for special inspection. However, the following aspects of light-frame construction are not covered adequately by the exception language:

1. Fastener spacing for shear walls could vary throughout the building. It is not clear that the exception would only be applicable to the particular shear wall or diaphragm with the larger fastening spacing, and to the other elements of the lateral force-resisting system associated with that shear wall or diaphragm.
2. The main elements of the lateral force-resisting system of light-frame buildings are the shear walls and the horizontal diaphragms. Elements associated with the shear walls include hold-downs, and the parts use to make connection to the foundation or the horizontal diaphragms, including sill plates, sole plates, bottom tracks, and blocking and framing clips. Elements associated with the horizontal diaphragms include chords, collectors, and elements used to anchor concrete and masonry walls for out-of-plane forces (such as blocking, straps, and hold-down hardware used horizontally. As written, it is not clear when special inspection would be required for the elements associated with the shear walls and diaphragms.
3. Shear wall sheathing is fastened at the sheathing edges, and in the middle of the panel. It is not clear that the reference to sheathing fastening is intended to apply to fastening along sheathing edges.
4. Diaphragm sheathing fastening is often specified with different spacing at sheathing edges, and at diaphragm boundaries. It is not clear what fastening (edge or boundary) is being referred to, or what portions of a horizontal diaphragm and associated elements would be affected by the exception.
5. Buildings of pre-dominantly light-frame construction often use vertical lateral force-resisting elements made up of other materials, such as steel frames, or concrete shear walls or masonry shear walls. It is not clear under what conditions special inspection would be required for the elements used to connect such vertical lateral force-resisting elements to the light-frame building system.
6. Light-frame diaphragms are often used in buildings where all of the vertical lateral force-resisting elements are made up of other materials, such concrete tilt-up shear or masonry shear walls. It is not clear under what conditions special inspection would be required for the wood, light-frame, and/or steel elements used to anchor the concrete or masonry walls for out-of plane forces.

The proposed change includes similar revisions to the provisions for structural wood, and for cold-formed light-frame construction. Shear walls and horizontal diaphragms are handled separately and the elements associated with each are identified. This makes it clear, once the special inspection is triggered (by specified fastener spacing, double sided sheathing, or the use of strap bracing) which elements other than the sheathing fastening, require inspection.

The requirements for inspection of anchorage elements in horizontal diaphragms for out-of-plane support of concrete and masonry walls are made explicit.

Public Comment 2:

Mark Nowak, MNowak Consulting, LLC, representing Steel Framing Alliance, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1705.10.1 Structural wood. Special inspection for wood construction ~~within~~ comprising the main windforce-resisting system shall be as required by this section. ~~Special inspection for wood construction in accordance with this section shall also be provided where vertical elements of the main windforce-resisting system are comprised of other materials, such as steel frames and concrete or masonry shear walls.~~

1705.10.1.1 Field gluing operations. Continuous special inspection is required during field gluing operations of wood elements of the main windforce-resisting system~~.~~

~~**1705.10.1.2 Shear walls.** Periodic special inspection shall be required for the sheathing fastening, and for other connections within the shear wall. Such connections shall include hold-down or tie-down connections, sill plate and sole plate anchorage and connections, and connections between the top of the wall and the horizontal diaphragm above.~~

> ~~**Exception:** Special inspection for wood shear walls is not required where the sheathing is gypsum board or fiberboard or where the fastener spacing along shear wall sheathing edges is more than 4 inches (102 mm) on center,~~

~~**1705.10.1.3 Horizontal diaphragms.** Periodic special inspection shall be required for the sheathing fastening, diaphragm chord connections and splices, and collector connections and fastening.~~

> ~~**Exception:** Special inspection for horizontal wood diaphragms is not required where the sheathing is gypsum board or fiberboard or where the least fastener spacing along sheathing edges or diaphragm boundaries is more than 4 inches (102 mm) on center.~~

1705.10.1.2 Unusual main windforce-resisting systems, components, and connections . Periodic special inspection is required for installation and fastening of unusual materials and methods used in the construction of the main windforce-resisting system. Unusual materials and methods include those conditions which require special tools and techniques and which require a higher than normal level of installation precision to achieve intended performance.

1705.10.2 Cold-formed steel light-frame construction. Special inspection for cold-formed light-frame construction ~~within~~ comprising the main windforce-resisting system shall be as required by this section. ~~Special inspection for cold-formed light-frame construction in accordance with this section shall be provided where vertical elements of the main windforce-resisting system are comprised of other materials, such as steel frames and concrete or masonry shear walls.~~

~~**1705.10.2.1 Shear walls and strap-braced walls.** Periodic special inspection shall be required for the sheathing fastening, the welding or screw attachment of the strap bracing, and for other connections within the shear wall or strap-braced wall. Such connections shall include hold-down or tie-down connections, bottom track anchorage and connections, and connections between the top of the wall and the horizontal diaphragm above.~~

> ~~**Exception:** Special inspection for cold-formed light-frame shear walls is not required where either of the following apply:~~
>
> 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, and the fastener spacing along sheathing edges is more than 4 inches (102 mm) on center.~~

~~**1705.10.2.2 Horizontal diaphragms.** Periodic special inspection shall be required for the sheathing fastening, welding or screw attachment of diagonal strap bracing, diaphragm chord connections and splices, and collector connections and fastening.~~

> ~~**Exception:** Special inspection for cold-formed light-frame horizontal diaphragms is not required where either of the following apply:~~
>
> 1. ~~The sheathing is gypsum board or fiberboard.~~
> 2. ~~The sheathing is wood structural panel or steel sheets on only one side of the framing, and the least fastener spacing along sheathing edges or diaphragm boundaries is more than 4 inches (102 mm) on center.~~

1705.10.2.1 Field welding operations. Periodic special inspection is required for field welding operations of steel elements of the main wind force-resisting system.

1705.10.2.2 Unusual main windforce-resisting systems, components, and connections. Periodic special inspection is required for installation and fastening of unusual materials and methods used in the construction of the main windforce-resisting system. Unusual materials and methods include those conditions which require special tools and techniques and which require a higher than normal level of installation precision to achieve intended performance.

1705.11.2 Structural wood. Special inspection for wood construction ~~within~~ comprising the seismic force-resisting system shall be as required by this section. ~~Special inspection for wood construction in accordance with this section shall be provided where vertical elements of the seismic force-resisting system are comprised of other materials, such as steel frames and concrete or masonry shear walls.~~

1705.11.2.1 Field gluing operations. Continuous special inspection shall be required during field gluing operations of wood elements of the seismic force-resisting system.

~~**1705.11.2.2 Shear walls.** Periodic special inspection shall be required for the sheathing fastening, and for other connections within the shear wall. Such connections shall include hold-down or tie-down connections, sill plate and sole plate anchorage and connections, and connections between the top of the wall and the horizontal diaphragm above.~~

> ~~**Exception:** Special inspection for wood shear walls is not required where the sheathing is gypsum board or fiberboard or where fastener spacing along shear wall sheathing edges is more than 4 inches (102 mm) on center,~~

1705.11.2.2 Unusual seismic force-resisting systems, components, and connections . Periodic special inspection is required for installation and fastening of unusual materials and methods used in the construction of the seismic force-resisting system.

Unusual materials and methods include those conditions which require special tools and techniques and which require a higher than normal level of installation precision to achieve intended performance.

~~**1705.11.2.3 Horizontal diaphragms.** Periodic special inspection shall be required for the sheathing fastening, diaphragm chord connections and splices, and collector connections and fastening.~~

> ~~**Exception:** Special inspection for horizontal wood diaphragms is not required where the sheathing is gypsum board or fiberboard or where least fastener spacing along sheathing edges or diaphragm boundaries is more than 4 inches (102 mm) on center.~~

1705.11.3 Cold-formed steel light-frame construction. Special inspection for cold-formed light-frame construction ~~within~~ comprising the seismic force-resisting system shall be as required by this section. ~~Special inspection for cold-formed light-frame construction in accordance with this section shall be provided where vertical elements of the seismic force-resisting system are comprised of other materials, such as steel frames and concrete or masonry shear walls.~~

~~**1705.11.3.1 Shear walls and strap-braced walls.** Periodic special inspection shall be required for the sheathing fastening, the welding or screw attachment of the strap bracing, and for other connections within the shear wall or strap-braced wall. Such connections shall include hold-down or tie-down connections, bottom track anchorage and connections, and connections between the top of the wall and the horizontal diaphragm above.~~

> ~~**Exception:** Special inspection for cold-formed light-frame shear walls is not required where either of the following apply:~~
>
> 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, and the fastener spacing along sheathing edges is more than 4 inches (102 mm) on center.~~

1705.11.3.1 Field welding operations. Periodic special inspection is required for field welding operations of steel elements of the seismic force-resisting system.

~~**1705.11.3.2 Horizontal diaphragms.** Periodic special inspection shall be required for the sheathing fastening, welding or screw attachment of diagonal strap bracing, diaphragm chord connections and splices, and collector connections and fastening.~~

> ~~**Exception:** Special inspection for cold-formed light-frame horizontal diaphragms is not required where either of the following apply:~~
>
> 1. ~~The sheathing is gypsum board or fiberboard.~~
> 2. ~~The sheathing is wood structural panel or steel sheets on only one side of the framing, and the least fastener spacing along sheathing edges or diaphragm boundaries is more than 4 inches (102 mm) on center.~~

1705.11.3.2 Unusual seismic force-resisting systems, components, and connections. Periodic special inspection is required for installation and fastening of unusual materials and methods used in the construction of the seismic force-resisting system. Unusual materials and methods include those conditions which require special tools and techniques and which require a higher than normal level of installation precision to achieve intended performance.

Commenter's Reason: This public comment addresses the CDC's concern with the original S156-12 proposal adding confusion to the special inspection requirements. Instead, this PC clarifies and streamlines requirements to conform more closely with the role and purpose of special inspections as defined in Section 1705.1.1 (e.g., must meet the criteria of "unusual design").

S156-12

Final Action: AS AM AMPC____ D

S159-12
1705.10.3

Proposed Change as Submitted

Proponent: Stephen Kerr, S.E. Josephson Werdowatz and Associates, representing Structural Engineers Association of California (SEAOC) (skerr@jwa-se.com)

Revise as follows:

1705.10.3 Wind-resisting components. Periodic special inspection is required for fastening of the following systems and components:

1. Roof ~~cladding~~ covering, roof deck, and roof framing connections.
2. ~~Wall cladding~~ Exterior covering, and wall connections to roof and floor diaphragms and framing.

Reason: The purpose of this change is to provide clarity and detail for the special inspection requirements for wind-resisting components in high-wind regions. The 2009 IBC identified “roof cladding and roof framing connections” and “wall connections to roof and floor diaphragms and framing” as wind-resisting components that needed to be included in the statement of special inspections, but only referenced “roof cladding” and “wall cladding” in the section describing the actual inspection. However, as part of the reorganization of Chapter 17 approved in the previous code change cycle, the more detailed language was deleted when the inspection requirements were combined with the requirements for inclusion in the statement of special inspections. In addition, “cladding” is not defined.

This proposal restores the more detailed description of the elements requiring special inspection, and uses terms defined in the code to identify the elements.

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

1705.10.3-S-KERR.doc

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

1705.10.3 Wind-resisting components. Periodic special inspection is required for fastening of the following systems and components:

1. Roof covering, roof deck, and roof framing connections.
2. Exterior wall covering and wall connections to roof and floor diaphragms and framing.

Committee Reason: This proposal makes the requirements for special inspections of wind-resisting components more specific, clarifying that the scope of this section should be focused on fastening and connections rather than the framing. The modification clarifies the applicability to exterior wall coverings.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Mark Nowak, MNowak Consulting, LLC, representing Steel Framing Alliance, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1705.10.3 Wind-resisting components. Periodic special inspection is required for ~~fastening of~~ the following systems and components:

1. Roof covering~~,~~ <u>and its attachment to the</u> roof deck~~, and roof framing connections~~.
2. Exterior wall covering and <u>its attachment to</u> wall ~~connections to roof and floor diaphragms and~~ framing.

Commenter's Reason: This public comment addresses the following two problems with the proposal:

1. Special inspection of roof coverings and exterior wall coverings should not just be limited to fastening. The inspection should include the covering materials <u>and</u> the attachment. Attachment of roof and exterior wall covering is often governed by the specific or unique nature of the covering materials or assembly and manufacturer instructions. The roof and exterior wall covering needs to be inspected as an assembly of components, not just the roof covering attachment.
2. Special inspection of roof framing connections and wall framing connections are part of the routine building department inspection process for the buildings structural framing system. These are not "unusual design" conditions as required to justify the need for special inspections in accordance with Section 1705.1.1.

With the above issues addressed this PC will ensure roof coverings and exterior wall coverings and their attachments are properly subject to special inspection in high wind zones.

S159-12

Final Action: AS AM AMPC____ D

S163-12
1705.11.5

Proposed Change as Submitted

Proponent: Stephen Kerr, S.E., Josephson Werdowatz and Associates, representing Structural Engineers Association of California (SEAOC) (skerr@jwa-se.com)

Revise as follows:

1705.11.5 Architectural components. Periodic *special inspection* is required during the erection and fastening of exterior cladding, interior and exterior nonbearing walls, suspended ceiling systems including their anchorage and interior and exterior veneer in structures assigned to *Seismic Design Category* D, E or F.

Exceptions:

1. *Special inspection* is not required for exterior cladding, interior and exterior nonbearing walls and interior and exterior veneer 30 feet (9144 mm) or less in height above grade or walking surface.
2. *Special inspection* is not required for exterior cladding and interior and exterior veneer weighing 5 psf (24.5 N/m^2) or less.
3. *Special inspection* is not required for interior nonbearing walls weighing 15 psf (73.5 N/m^2) or less.

Reason: This proposal restores the needed special inspection for suspended ceiling systems. The 2009 IBC identified "suspended ceiling systems and their anchorage" as components that needed to be included in the statement of special inspections for Seismic Design Category D, E or F, but did not list them in the section that invoked the actual inspection. Then, as part of the reorganization of Chapter 17 approved in the previous code change cycle, the requirement was deleted completely when the inspection requirements were combined with the requirements for inclusion in the statement of special inspections.

Suspended ceiling systems, when not properly anchored and braced, are well known to fail under strong shaking, resulting in debris that can block exits or otherwise impede egress from buildings.

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

1705.11.5-S-KERR.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee believes that there is no special expertise needed for the inspection of suspended ceilings and that its removal in a prior code cycle was not inadvertent. The committee did not hear any justification for requiring this special inspection.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

John Gillengerten, Code Resource Support Committee, representing FEMA/Building Seismic Safety Council, requests Approval as Submitted.

Commenter's Reason: Suspended ceilings systems pose a significant risk to public safety, and there are many instances of failures in earthquakes. The Commentary of the 2009 NEHRP Provisions, Section C13.1 states that:

"Suspended or attached nonstructural components that could detach either in full or in part from the structure during an earthquake are referred to as falling hazards and may represent a serious threat to property and life safety. Critical attributes that influence the hazards posed by these components include their weight, their attachment to the structure, their failure or breakage characteristics (e.g., certain types of glass), and their location relative to occupied areas (e.g., over an entry or exit, a public walkway, an atrium, or a lower adjacent structure)."

"Components whose collapse during an earthquake could result in blockage of the means of egress deserve special consideration. The term "means of egress" is used commonly in building codes with respect to fire hazard. Consideration of egress may include intervening aisles, doors, doorways, gates, corridors, exterior exit balconies, ramps, stairways, pressurized enclosures, horizontal exits, exit passageways, exit courts, and yards. Items whose failure could jeopardize the means of egress include walls around stairs and corridors, veneers, cornices, canopies, heavy partition systems, ceilings, architectural soffits, light fixtures, and other ornaments above building exits or near fire escapes."

Recent suspended ceiling damage in significant earthquakes is documented in FEMA E-74. Additional information on the performance of different suspended ceiling types may be viewed at the following web pages:

http://www.fema.gov/earthquake/fema-e-74-reducing-risks-nonstructural-earthquake-damage-76
http://www.fema.gov/earthquake/fema-e-74-reducing-risks-nonstructural-earthquake-damage-79

Earthquake experience has conclusively demonstrated that suspended ceiling performance depends on proper installation. Installation of safety wires for suspended light fixtures and diffusers, proper installation of supports at the ceiling perimeter, and use of the proper fasteners for connection of supports and bracing to the structure are all critical, and installation errors have resulted in failures. Good performance requires special inspection to ensure proper installation.

Special inspection of the installation of suspended ceilings was required in both the 2006 & 2009 IBC. It was removed in the 2012 IBC as part of an overall re-organization of Chapter 17. No justification was provided by the proposer of the 2012 IBC changes other than in his opinion it was not needed. We disagree for the reasons indicated above and request special inspection of suspended ceilings be re-instated which this proposal would do.

S163-12

Final Action: AS AM AMPC____ D

S171-12
1709.3.2

<u>Proposed Change as Submitted</u>

Proponent: Gary R. Searer, Wiss, Janey, Elstner Associates Inc., representing self

Revise as follows:

1709.3.2 Load test procedure not specified. In the absence of applicable load test procedures contained within a standard referenced by this code or acceptance criteria for a specific material or method of construction, such *existing structure* shall be subjected to a test procedure developed by a *registered design professional* that simulates applicable loading and deformation conditions. For components that are not a part of the seismic load-resisting system, the test load shall be equal ~~to two times the unfactored design loads~~ <u>to the minimum of the specified factored design loads. For statically loaded components,</u> the test load shall be left in place for a period of 24 hours. <u>For components such as machine supports or fall arrest anchors that carry dynamic loads, the load shall be left in place for a period consistent with the component's actual function.</u> The structure shall be considered to have successfully met the test requirements where the following criteria are satisfied:

1. Under the design load, the deflection shall not exceed the limitations specified in Section 1604.3.
2. Within 24 hours after removal of the test load, the structure shall have recovered not less than 75 percent of the maximum deflection.
3. During and immediately after the test, the structure shall not show evidence of failure.

Reason: This code change proposal does two things: 1) changes the required static test load from *precisely* "two times the unfactored design load" to a "minimum of the specified factored design loads", and 2) specifies how to test components that carry dynamic loads.

It is essentially not possible for the test load to be precisely two times any particular load, and the requirement to test to two times the unfactored load is arbitrary (i.e., why should you test to 2.0D+2.0L if the commonly accepted and statistically based load combination is 1.2D+1.6L?). By adding the phrase "a minimum of" to the requirement and by referencing factored loads, the intent of the provision is made clear -- that the test load should be *at least* the specified factored design load. Nationally recognized design standards such as the AISC Steel Specifications and ACI 318 have been developed with the intent to ensure that very few elements are unable to carry factored loads. To put it another way, if every element in a structure could carry factored loads, the structure's reliability would be consistent with the intent of such standards. In fact, the load testing provisions in each of the AISC and ACI standards make this clear by requiring proof test loads to essentially the full factored loads. This proposal is in-line with both AISC and ACI standards.

When an element is designed to carry short duration or dynamic loads, there is no need to sustain a proof test load for 24 hours.

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

1714.3.2-S-SEARER.doc

<u>Public Hearing Results</u>

Committee Action: **Approved as Modified**

Modify proposal as follows:

1709.3.2 Load test procedure not specified. In the absence of applicable load test procedures contained within a standard referenced by this code or acceptance criteria for a specific material or method of construction, such *existing structure* shall be subjected to a test procedure developed by a *registered design professional* that simulates applicable loading and deformation conditions. For components that are not a part of the seismic load-resisting system, <u>at a minimum</u> the test load shall be equal to ~~the minimum of~~ the specified factored design loads. For statically loaded components, the test load shall be left in place for a period of 24 hours. For components such as machine supports or fall arrest anchors that carry dynamic loads, the load shall be left in place for a period consistent with the component's actual function. The structure shall be considered to have successfully met the test requirements where the following criteria are satisfied:

1. Under the design load, the deflection shall not exceed the limitations specified in Section 1604.3.

2. Within 24 hours after removal of the test load, the structure shall have recovered not less than 75 percent of the maximum deflection.
3. During and immediately after the test, the structure shall not show evidence of failure.

Committee Reason: This proposal clears up the issue of duration of load for the test procedure and removes the arbitrary factor of two. The modification improves the wording to indicate you don't have to test to all load combinations.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Gary Searer, Wiss, Janney, Elstner Associates, Inc. (WJE) representing self, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1709.3.2 Load test procedure not specified. In the absence of applicable load test procedures contained within a standard referenced by this code or acceptance criteria for a specific material or method of construction, such existing structure shall be subjected to a test procedure developed by a registered design professional that simulates applicable loading and deformation conditions. For components that are not a part of the seismic load-resisting system, at a minimum the test load shall be equal to the specified factored design loads. <u>For materials such as wood that have strengths that are dependent on load duration, the test load shall be adjusted to account for the difference in load duration of the test compared to the expected duration of the design loads being considered.</u> For statically loaded components, the test load shall be left in place for a period of 24 hours. For components such as machine supports or fall arrest anchors that carry dynamic loads, the load shall be left in place for a period consistent with the component's actual function. The structure shall be considered to have successfully met the test requirements where the following criteria are satisfied:

1. Under the design load, the deflection shall not exceed the limitations specified in Section 1604.3.
2. Within 24 hours after removal of the test load, the structure shall have recovered not less than 75 percent of the maximum deflection.
3. During and immediately after the test, the structure shall not show evidence of failure.

Commenter's Reason: I am the author of the original code change proposal, which was approved by the IBC-Structural Committee. The code change proposal was written to bring the testing requirements into line with the standards of most major materials, including concrete (ACI) and steel (AISC). However, wood responds differently under short duration loads and long duration loads, and the test load needs to be adjusted to account for differences caused by load duration. This change accomplishes this goal.

Consequentially, I respectfully ask that the proposed code change be approved as modified by this public comment.

S171-12

Final Action: AS AM AMPC____ D

S172-12
1710.5

Proposed Change as Submitted

Proponent: Julie Ruth, P.E. JRuth Code Consulting, representing American Architectural Manufacturers Association (AAMA) (julruth@aol.com)

Revise as follows:

1710.5 Exterior window and door assemblies. The design pressure rating of exterior windows and doors in buildings shall be determined in accordance with Section 1710.5.1 or 1710.5.2.

> **Exception:** Structural wind load design pressures for window units ~~smaller~~ other than the size tested in accordance with Section 1710.5.1 or 1710.5.2 shall be permitted to be ~~higher~~ different than the design value of the tested unit provided such ~~higher~~ pressures are determined by accepted engineering analysis or validated by an additional test of the window unit to the alternate allowable design pressure in accordance with Section 1710.5.2. All components of the ~~small~~ alternate size unit shall be the same as the tested or labeled unit. Where ~~such calculated design pressures are~~ engineering analysis is used, ~~they shall be validated by an additional test of the window unit having the highest allowable design pressure~~ the glass shall comply with Section 2403.

Reason: The current exception limits the use of comparative analysis to window units smaller than the size originally tested for labeling purposes. If comparative analysis is used to provide a higher design pressure rating of the smaller unit, its resistance to air infiltration and water penetration at the correspondingly higher design pressure required by AAMA/WDMA/CSA 101/I.S.2/A440 must be verified by testing of the unit. These characteristics cannot be determined by calculation.

Comparative analysis is also appropriate to rate window units larger than the size originally tested for labeling purposes to lower design pressures. In this scenario, the corresponding design pressure used to verify resistance to air infiltration and water penetration would also be lower. Testing would not be required to verify this level of performance since a higher level has already been determined by testing of the same components in a smaller window unit.

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 for labeling. The last sentence of the section is also revised to specify that when engineering analysis is used, the glass in the fenestration product must also comply with Section 2403. Section 2403 establishes specific criteria for the deflection of the framing supporting the glass.

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

1710.5 #1-S-RUTH

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee has reservations about allowing test results to be scaled up in order to allow large window units. Preference would be to have this issue resolved within the referenced standards.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Julie Ruth, JRuth Code Consulting, representing American Architectural Manufacturers Association requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1710.5 Exterior window and door assemblies. The design pressure rating of exterior windows and doors in buildings shall be determined in accordance with Section 1710.5.1 or 1710.5.2.

Exceptions:

1. Structural wind load design pressures for window and door units other than the size tested in accordance with Section 1710.5.1 ~~or 1710.5.2~~ shall be permitted to be different than the design value of the tested unit provided such different pressures are determined by accepted engineering analysis or validated by an additional test of the window or door unit to the ~~alternate allowable~~ different design pressure in accordance with Section ~~1710.5.2~~ 1710.5.1. All components of the alternate size unit shall be the same as the tested or labeled unit. ~~Where engineering analysis is used, the glass shall comply with Section 2403.~~
   - 1.1 Operable windows and doors rated in this manner shall comply with the following:
     - 1.1.1. The frame area of the alternate size unit shall not exceed the frame area of the tested unit.
     - 1.1.2. The alternate size unit shall vary from the tested unit only in width, height or load requirements.
     - 1.1.3. The proportional deflection for fiber stress of intermediate members of the alternate size unit shall not exceed 100 percent of the proportional deflection for fiber stress of the intermediate members of the tested unit.
     - 1.1.4. The concentrated load at the juncture of the intermediate members and the frame of the alternate size unit shall not exceed 100 percent of the concentrated load at the juncture of the intermediate members and the frame of the tested unit.
     - 11.5. The rated air and water infiltration resistance of the alternate size unit shall not exceed the air and water infiltration resistance of the tested unit.
     - 1.1.6. The maximum cyclic pressure of the alternate size unit shall not exceed the maximum cyclic pressure of the tested unit when tested in accordance with ASTM E 1886 and ASTM E 1996, where applicable.
   - 1.2 Non-operable windows and doors rated in this manner shall comply with the following:
     - 1.2.1. The frame area of the alternate size unit shall not exceed the frame area of the tested unit.
     - 1.2.2. The alternate size unit shall vary from the tested unit only in width, height or load requirements.
     - 1.2.3. The maximum uniform load distribution (ULD) of any side of either unit shall be equal to the uniform load carried by the side divided by the length of the side.
     - 1.2.4. The ULD of any member of the alternate size unit shall not exceed the ULD of the corresponding member of the tested unit.
     - 1.2.5. The ULD of each member of both units shall be calculated in accordance with standard engineering analysis.
     - 1.2.6. The rated air and water infiltration resistance of the alternate size unit shall not exceed the air and water infiltration resistance of the tested unit.
     - 1.2.7. The maximum cyclic pressure of the alternate size unit shall not exceed the maximum cyclic pressure of the tested unit when tested in accordance with ASTM E 1886 and ASTM E 1996, where applicable.
2. For window and door units tested in accordance with Section 1710.5.2, structural wind load design pressures for window and door units other than the size tested in accordance with Section 1710.5.2 shall be permitted to be different than the design value of the tested unit provided such different pressures are determined by accepted engineering analysis or validated by an additional test of the window or door unit to the different design pressure in accordance with Section 1710.5.2. All components of the alternate size unit shall be the same as the tested unit. Where engineering analysis is used, the glass shall comply with Section 2403.

Commenter's Reason: This Public Comment seeks approval of the original proposal, with modifications that seek to address the concerns raised with the proposal by the IBC Structural Committee during the ICC Group A code development hearings.

At the present time Section 1710.5 requires the design pressure rating of exterior windows and doors to be determined in accordance with either Section 1710.5.1 or 1710.5.2.

Section 1710.5.1 requires exterior windows and sliding doors to be tested and labeled in accordance with AAMA/WDMA/CSA 101/I.S.2/A440. This standard establishes criteria for performance grade rating of the fenestration product as R, LC, CW or AW. It establishes criteria for resistance to air leakage and water penetration, as well as structural testing, based upon the performance grade and design pressure rating. Framing deflection criteria are also established in the standard for fenestration products rated for performance grade CW and AW.

Section 1710.5.2 addresses structural testing only. Through reference to Section 2403 it establishes more rigorous deflection criteria than Section 1710.5.1 for performance grade R and LC fenestration products.

An exception to the criteria of both Sections 1710.5.1 and 1710.5.2 is given in Section 1710.5. The exception permits the rating of smaller fenestration products to a higher design pressure based upon engineering analysis when specific criteria given in the exception are met. The criteria include the use of framing members in the smaller unit that are identical to those of the tested unit, and the testing of the unit that is to have the highest design pressure rating.

Resistance to air leakage and water penetration characteristics cannot be determined by engineering analysis alone. The pressure at which a unit is tested for resistance to air leakage is determined by the performance grade and water penetration is determined by the targeted design pressure rating of a product. Therefore, to verify the higher design pressure rating of the smaller unit in accordance with AAMA/WDMA/CSA 101/I.S.2/A440, it must be tested.

The procedure described in the exception is commonly referred to within the fenestration industry as “comparative analysis”. Basically, a window unit that meets or exceeds the size specified in AAMA/WDMA/CSA 101/I.S.2/A440 is tested to a lower design pressure rating, then a smaller unit is tested to the highest design pressure rating sought for that particular product line. Based upon the results of those 2 testing sequences the design pressure rating of intermediate size units can be interpolated using engineering analysis.

It is not uncommon for fenestration products to be tested to design pressures that are considerably higher than those required by the applicable code, or ASCE 7, for installation in a specific building or use on a certain project. These exceptions provide a method of determining the appropriate design pressure rating of these specialty units without requiring repetition of a complete sequence of testing, through comparison to existing, tested and approved units.

Although having this exception in Section 1710.5 of the IBC is helpful, there are specific scenarios that it does not address. The intent of the original proposal, and this proposed modification, is to seek to address these additional scenarios as accurately as possible.

In the first scenario, specific instances can occur under which it is not clear if the size of an alternate unit is smaller or larger than that of the tested unit. This can occur if the aspect ratio of height to width is different. For example, the height of an alternate unit may be greater than that of the tested unit, but its width narrower. Is such a unit larger or smaller than the reference unit? Proposed exceptions I.i and I.ii of this Public Comment brings in criteria that have been used successfully in Florida for the evaluation of the rating of an alternate size unit. This criteria is based upon consistency with the tested unit with regards to frame area, components, resistance to air and water infiltration and maximum cyclical pressure, when applicable. Additional criteria are given with regards to proportional deflection of intermediate framing members and concentrated loads at the intersection of intermediate members and the frame for operable units. Additional criteria are given with regards to uniform load distribution (ULD) for fixed units.

In the second scenario, engineering analysis alone could be used to evaluate window units larger than those evaluated in accordance with Section 1710.5.2, since that section does not include requirements for air leakage and water penetration resistance. In this scenario all units are subject to the deflection criteria of Section 2403.0. Proposed exception 2 of this Public Comment provides the criteria for evaluating these alternative size units based upon engineering analysis or additional testing, and the deflection criteria of Section 2403.0.

Since the two scenarios described above are not specifically addressed in the current exception, different interpretations of the IBC can result in the exception being applied inconsistently. These proposed modifications will increase the clarity of the exceptions, resulting in greater clarity and more consistent application of the comparative analysis provisions. We urge approval of S172 as modified by this Public Comment.

S172-12

Final Action: AS AM AMPC____ D

S175-12
1710.5.1

<u>Proposed Change as Submitted</u>

Proponent: Thomas S. Zaremba, Roetzel & Associates, representing Glazing Industry Code Committee (tzaremba@ralaw.com)

Revise as follows:

1710.5.1 Exterior windows and doors. Exterior windows and sliding doors shall be tested and labeled as conforming to AAMA/WDMA/CSA101/I.S.2/A440. The *label* shall state the name of the manufacturer, the *approved* labeling agency and the product designation as specified in AAMA/ WDMA/CSA101/I.S.2/A440. Exterior side-hinged doors shall be tested and *labeled* as conforming to AAMA/WDMA/CSA101/I.S.2/A440 or comply with Section 1710.5.2. Products <u>in Risk Category I and II buildings</u> tested and labeled as conforming to AAMA/WDMA/CSA 101/I.S.2/A440 shall not be subject to the requirements of Sections 2403.2 and 2403.3 <u>provided one of the following is met:</u>

1. <u>The required design pressure for the fenestration product does not exceed 60 psf or</u>
2. <u>All glass in the fenestration product is tempered or laminated.</u>

Reason: Chapter 24 and ASTM E1300 require that glazing be firmly supported to prevent breakage under the design load by establishing maximum framing deflection limits. The glass strength calculations in ASTM E1300 use this as a basis to establish a probability of glass breakage less than 8 in 1000. However, Section 1710.5.1 currently exempts certain residential and light commercial products from this requirement if they are labeled to the AAMA/WDMA/CSA 101/I.S.2/A440 standard. While this may be appropriate when these products are used in applications with lower design loads and/or lower risk building types, allowing this exception for *all* product types in *all* occupancies is far too broad. This proposal would correct this overbreadth by ensuring that products used in higher risk situations be firmly supported and meet the frame deflection limit to restore an appropriate safety margin consistent with ASTM E1300.

Specifically, this proposal would limit the exception to only risk category I and II buildings, and products used in higher risk category buildings must meet the Chapter 24 requirement for firmly supported glazing. This includes hospitals, public assembly areas with over 300 people, schools (often used as storm shelters), mission-critical facilities, and infrastructure. To provide flexibility, the proposal also maintains the exception for lower design pressures less than 60 psf, and where tempered or laminated glass is used as an alternative method to reduce the probability of glass breakage and/or potential risk of falling glass.

This proposal is significantly different than other proposals discussed in previous cycles, which would have removed the exception for all buildings other than lowrise residential. This proposal takes a much more moderate approach to restore the appropriate safety margin consistent with Chapter 24 and ASTM E1300 in higher risk situations, but leave the exception and flexibility for residential and light commercial products in lower risk applications.

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

1710.5.1-S-ZAREMBA.doc

<u>Public Hearing Results</u>

Committee Action: **Disapproved**

Committee Reason: The committee feels there is no justification for this proposal.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Thomas S. Zaremba, Roetzel & Andress, representing Glazing Industry Code Committee, requests Approval as Submitted.

Commenter's Reason: Simply put, the Committee got this one wrong. Without offering any technical justification or explanation for its action, the Committee disapproved S175-12, saying, nothing more than: "The Committee feels there is no justification for this proposal."

This code change proposal is an effort to correct a clear inconsistency that exists between Chapters 24 and 17 of the Code. In that regard, Sections 2403.2 and 2403.3 specify how to determine whether glass is adequately supported in its framing. These sections do this by limiting the amount of glass deflection the frame is permitted to allow when the glazing assembly is subjected to the larger of the negative or positive loads described in Chapter 16.

These sections of Chapter 24 are critical to a safe building environment. In that regard, if glass is not adequately supported by its frame, there is a risk that it will break and fall out of its frame under loads specified in Chapter 16. If the glass used in the inadequately supported frame is a non-safety glazing, such as annealed glass, there is a risk that the broken glass will injure a building occupant.

The technical basis for the deflection limit found in Section 2403.3 is ASTM E1300. Based on the engineering analysis and computations underlying the ASTM E1300 standard, the L/175 deflection specified in Section 2403.3 limits the probability of glass breakage to less than 8 in 1,000 (0.8%) under loading conditions described in Chapter 16. Since the exemption afforded by Section 1710.5.1 is significantly less stringent than Section 2403.3, a significantly higher incidence of glass breakage under the loads specified in Chapter 16 can be expected when Section 1710.5.1 is used rather than Section 2403.3. In fact, the standard referenced in Section 1710.5.1 states that if there is any glass breakage during testing, two retests are permitted. On retesting, a failure to meet the standard's acceptance criteria occurs only if all ***three*** test specimens fail the test. If any ***one*** specimen passes in either of two retests, the glazing is deemed to be in full compliance with the standard.

In short, application of the test standard prescribed in Section 1710.5.1 introduces a significant increase in the probability of glass failures into the as-built environment. In the worst case, use of Section 1710.5.1 may increase the probability of breakage from the 0.8% prescribed by Section 2403.3, to as much as 33%.

Even though there is no real justification for allowing framing support standards to fall below those set in Chapter 24, nevertheless, Section 1710.5.1 exempts all "exterior windows and sliding doors" labeled to AAMA/WDMA/CSA 101/I.S.2/A440 from the frame support and deflection limits specified in Chapter 24. While it makes sense to reduce framing support requirements for low risk occupancies, or, when safe breaking glass is used in an installation, it does not make any sense to reduce these requirements for occupancies with high risk profiles, especially when a glass that does not break safely, such as annealed glass, is being used.

Specifically, if adopted, S175-12 would limit the blanket exemption otherwise afforded by Section 1710.5.1 from Chapter 24's framing support requirements to applications (i) where the design pressure for the glazing did not exceed 60 psf (ii) where safe breaking glazing, either tempered or laminated, is used throughout the installation, and (iii) to low-risk, category I and II type buildings. Occupancies with risk profiles higher than category I and II, including hospitals, public assemblies with over 300 people, schools (often used as storm shelters), and mission-critical facilities, would be required to meet the more stringent framing support requirements of Chapter 24 unless safe breaking types of glazing are used.

If the Committee's unsubstantiated recommendation is allowed to stand, the overly broad exemption found in Section 1710.5.1 will continue to provide a blanket exemption from the technically sound and appropriately stringent framing support requirements prescribed by Chapter 24. This is completely unwarranted especially when continuing this blanket exemption would carry with it a significant increase in the risk of glass breakage when subjected to loads addressed in Chapter 16 in building installations where the use of annealed and other forms of non-safety glass may be allowed.

In order to ensure that glazing properly contributes to a safe, as-built environment, rational limits on the blanket exemption provided in Section 1710.5.1 are needed. As a result, the Glazing Industry Code Committee urges you to support S175-12 by voting against the standing motion to disapprove S175-12 and voting in favor of a motion to approve S175-12 as submitted.

S175-12

Final Action: AS AM AMPC____ D

S181-12

1803.5.7, 1804.1, 1804.2 (New), 1804.2.1 (New)

Proposed Change as Submitted

Proponent: Edwin Huston, National Council of Structural Engineers Associations- Code Advisory Committee - General Requirements Subcommittee (huston@smithhustoninc.com)

Revise as follows:

1803.5.7 Excavation near foundations. Where excavation will remove ~~lateral~~ support from any foundation~~, an investigation shall be conducted to assess the potential consequences and address mitigation measures~~ a Registered Design Professional shall prepare a report summarizing the condition of the structure as determined from examination of the structure, the review of available design documents and if necessary, the excavation of test pits. The Registered Design Professional shall determine the requirements for underpinning and protection and prepare site-specific plans, details, and sequence of work for submission. Such support may be provided by underpinning, sheeting, and bracing, or by other means acceptable to the building official.

1804.1 Excavation near foundations. Excavation for any purpose shall not remove lateral support from any foundation or adjacent foundation without first underpinning or protecting the foundation against settlement or lateral translation.

1804.2 Underpinning. Where the protection and/or support of adjacent structures is required, the underpinning system shall be designed and installed in accordance with provisions of this chapter and Chapter 33.

1804.2.1 Underpinning and bracing installation. Where underpinning is used for the support of adjacent structures, the piers, wall piles or footings shall be installed in such manner so as to prevent the lateral or vertical displacement of the adjacent structure, to prevent deterioration of the foundations or other effects that would disrupt the adjacent structure. The sequence of installation shall be identified in the design.

Reason: At present, excavation of foundations is not specifically addressed in relation to adjacent structures. Section 3307, Protection of Adjacent Property, states: “Adjoining public and private property shall be protected from damage during construction, remodeling and demolition work. Protection shall be provided for footings, foundations, party walls, chimneys, skylights and roofs.”

The code currently has minimal and vague requirements of the due diligence required for investigation for excavation near a neighboring structure. Failures to perform proper pre-construction investigations and monitoring procedures have led to failures in construction during underpinning and excavation operations. Improper excavations result nationally in doors and windows that don’t open, increasing through cracking of bearing walls and support members, failures of structural members and to collapse and fatalities.

Specific guidelines are provided to identify responsibilities and basic requirements for providing safe and successful underpinning and excavations.

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

1803.5.7-S-HUSTON.doc

Public Hearing Results

Committee Action: Disapproved

Committee Reason: This code change is considered a good effort to clarify requirements for excavations near a neighboring structure, but the committee believes there are details that must be worked out. Requirements for underpinning should make it clear that its not the only means permitted. There should be a link to Chapter 33. The report requirement may not be needed in all cases.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Edwin Huston, National Council of Structural Engineers Associations (NCSEA), representing NCSEA Code Advisory Subcommittee – General Requirements Subcommittee, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1803.5.7 Excavation near foundations. Where excavation will ~~remove~~ reduce support from any foundation, a Registered Design Professional shall prepare ~~a report summarizing the condition~~ an assessment of the structure as determined from examination of the structure, the review of available design documents and if necessary, the excavation of test pits. The Registered Design Professional shall determine the requirements for underpinning and protection and prepare site-specific plans, details, and sequence of work for submission. Such support may be provided by underpinning, sheeting, and bracing, or by other means acceptable to the building official.

1804.1 Excavation near foundations. Excavation for any purpose shall not ~~remove~~ reduce lateral support from any foundation or adjacent foundation without first underpinning or protecting the foundation against detrimental lateral or vertical movement, or both ~~settlement or lateral translation~~.

1804.2 Underpinning. Where underpinning is chosen to provide the protection ~~and/~~or support of adjacent structures ~~is required~~, the underpinning system shall be designed and installed in accordance with provisions of this chapter and Chapter 33.

~~**1804.2.1 Underpinning and bracing installation**. Where underpinning is used for the support of adjacent structures, the piers, wall piles or footings shall be installed in such manner so as to prevent the lateral or vertical displacement of the adjacent structure, to prevent deterioration of the foundations or other effects that would disrupt the adjacent structure. The sequence of installation shall be identified in the design.~~

1804.2.1 Underpinning Sequencing. Underpinning shall be installed in a sequential manner that protects the neighboring structure and the working construction site. The sequence of installation shall be identified in the construction documents.

Commenter's Reason: At present, excavation of foundations is not specifically addressed in relation to adjacent structures. Section 3307, Protection of Adjacent Property, states: "Adjoining public and private property shall be protected from damage during construction, remodeling and demolition work. Protection shall be provided for footings, foundations, party walls, chimneys, skylights and roofs."

The code currently has minimal and vague requirements of the due diligence required for investigation for excavation near a neighboring structure. Failures to perform proper pre-construction investigations and monitoring procedures have led to failures in construction during underpinning and excavation operations. Improper excavations result nationally in doors and windows that don't open, increasing through cracking of bearing walls and support members, failures of structural members and to collapse and fatalities.

At the Code Development Hearings the Structural Committee struggled with the prohibition of preventing all settlement or lateral translation, which is not possible. Alternate wording was considered. But the committee chose to disapprove this proposal and asked that it be reconsidered under a Public Comment. During testimony for a companion Code Change Proposal, S184-15, it was pointed out that the term "detrimental" is currently used to discuss settlement in Section 1805.1, 1808.4 and 1807.7.2, as well as in other chapters of the IBC and the structural committee approved S184-12 "As Modified" using the term "detrimental". This Public comment seeks to use that same terminology.

One member of the committee noted that a report is not always necessary, so we changed that requirement to require an assessment of the need for underpinning, or other means of providing support.

We are also changing remove support to reduce support, because removal of support could lead to failure.

As 1803.5.7 points out, underpinning is one way of providing support. So in 1804.2, we are noting requirements when underpinning is chosen to provide support.

We urge your support for AMPC.

S181-12

Final Action:	AS	AM	AMPC____	D

S184-12
1808.3.2 (New)

Proposed Change as Submitted

Proponent: Edwin Huston, National Council of Structural Engineers Associations (NCSEA), representing NCSEA Code Advisory Subcommittee – General Requirements Subcommittee (huston@smithhustoninc.com)

Add new text as follows:

<u>**1808.3.2 Surcharge.** No fill or other surcharge loads shall be placed adjacent to any building or structure unless such building or structure is capable of withstanding the additional loads caused by the fill or the surcharge. Existing footings or foundations which will be affected by any excavation shall be underpinned or otherwise protected against settlement and shall be protected against lateral movement.</u>

Reason: The code does not comment on permanent loads surcharging a neighboring structure. It references surcharge loads only in reference to construction loading in Chapter 33.

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

1808.3.2 (NEW)-S-HUSTON.doc

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

1808.3.2 Surcharge. No fill or other surcharge loads shall be placed adjacent to any building or structure unless such building or structure is capable of withstanding the additional loads caused by the fill or the surcharge. Existing footings or foundations which will be affected by any excavation shall be underpinned or otherwise protected against settlement and shall be protected against <u>detrimental</u> lateral <u>or vertical</u> movement<u>, or both</u>.

Committee Reason: This code change adds a needed provision on surcharge loads that affect an adjacent structure. Although Chapter 33 covers this during construction, the committee believes the proposed addition to Chapter 18 is useful and will help the building official. The modification clarifies that the vertical movement is also a concern and further states the protection is against detrimental movements. A public comment is suggested to provide an objective determination of detrimental movements.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Gary J. Ehrlich, P.E., National Association of Home Builders (NAHB), requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1808.3.2 Surcharge. No fill or other surcharge loads shall be placed adjacent to any building or structure unless such building or structure is capable of withstanding the additional loads caused by the fill or the surcharge. Existing footings or foundations which will be affected by any excavation shall be underpinned or otherwise protected against settlement and shall be protected against detrimental lateral or vertical movement, or both.

> <u>**Exception:** Minor grading for landscaping purposes shall be permitted where done with walk-behind equipment, where the grade is not increased more than one foot from original design grade, or where approved by the building official.</u>

Commenter's Reason: : As written, the proposed language would not permit any grading or landscaping against an existing building, even minor amounts placed for landscaping purposes (e.g. maintaining a French drain or adding mulch to plant bends) done with light-duty walk-behind equipment unless the owner hires an engineer to evaluate the foundation and foundation walls. This is unreasonable when the primary issue is major grading done with heavy-duty equipment, particularly where grading and compaction of soil is done perpendicular to the building wall. An exception is proposed for minor grading done with walk-behind equipment (which does not induce high forces against the wall), limited grading heights, or as approved by the building official.

It is noted that many jurisdictions require a "minor grading" permit for work of the nature covered by the exception. These permits typically limit the total cubic yards or square footage of grading, limit the work to the lot covered by the permit (i.e. a permit would not be granted for grading against a building on an adjacent lot), and typically require plans and details signed and sealed by a civil engineer. This permitting process supplies protection against abuse of the exception.

S184-12

Final Action: AS AM AMPC____ D

S190-12
1810.3.3.1.6

Proposed Change as Submitted

Proponent: Lori A. Simpson, P.E., GE, Treadwell & Rollo, a Langan Company, representing Deep Foundations Institute

Revise as follows:

1810.3.3.1.6 Uplift capacity of grouped deep foundation elements. For grouped deep foundation elements subjected to uplift, the allowable working uplift load for the group shall be calculated by an *approved* method of analysis. Where the deep foundation elements in the group are placed at a center-to-center spacing ~~of at least 2.5~~ less than three times the least horizontal dimension of the largest single element, the allowable working uplift load for the group is permitted to be calculated as the lesser of:

1. The proposed individual allowable working uplift ~~working~~ load times the number of elements in the group.
2. Two-thirds of the effective weight of the group and the soil contained within a block defined by the perimeter of the group and the length of the element, plus two-thirds of the ultimate shear resistance long the soil block.

Reason: A period is added because there was a run on sentence which rendered the section unclear. Also, the spacing is clarified to be consistent with Section 1810.2.5. Section 1810.3.3.1.6 had defined the need to evaluate group effects where spacing is at least 2.5 times the least horizontal dimension, but did not define a maximum spacing at which group effects did not need to be evaluated. The minimum spacing for evaluation of group effects on uplift capacity is not appropriate. Section 1810.2.5 says that group effects only need to be evaluated where the spacing is less than 3 times the least horizontal dimension, so that is repeated herein for consistency.

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

1810.3.3.1.6-S-SIMPSON.doc

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

1810.3.3.1.6 Uplift capacity of grouped deep foundation elements. For grouped deep foundation elements subjected to uplift, the allowable working uplift load for the group shall be calculated by ~~an *approved*~~ a generally accepted method of analysis. Where the deep foundation elements in the group are placed at a center-to-center spacing less than three times the least horizontal dimension of ~~the largest single~~ an element, the allowable working uplift load for the group is permitted to be calculated as the lesser of:

1. The proposed individual allowable working uplift load times the number of elements in the group.
2. Two-thirds of the effective weight of the group and the soil contained within a block defined by the perimeter of the group and the length of the element, plus two-thirds of the ultimate shear resistance along the soil block.

Committee Reason: Approval of these group effect clarifications is consistent with the committee's action on S185-12. The modification substitutes preferred wording that is intended to allow standard practice in various regions.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Lori A. Simpson, P.E., G.E., Treadwell & Rollo, a Langan Company, representing Deep Foundations Institute, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1810.3.3.1.6 Uplift capacity of grouped deep foundation elements. For grouped deep foundation elements subjected to uplift, the allowable working uplift load for the group shall be calculated by a generally accepted method of analysis. Where the deep foundation elements in the group are placed at a center-to-center spacing less than three times the least horizontal dimension of ~~an~~ the largest single element, the allowable working uplift load for the group is permitted to be calculated as the lesser of:

1. The proposed individual allowable working uplift load times the number of elements in the group.
2. Two-thirds of the effective weight of the group and the soil contained within a block defined by the perimeter of the group and the length of the element, plus two-thirds of the ultimate shear resistance along the soil block.

Commenter's Reason: The text "of the largest single" was deleted during the Code Change Proposal Hearing, however, it was realized that some foundation elements may have different dimensions (a belled pier, for example); in evaluating if the deep foundation elements are less than three times the least horizontal dimension, the least horizontal dimension of the largest element should be used. Therefore, the text "of the largest single" should be put back in.

S190-12

Final Action: AS AM AMPC____ D

S199-12
1904.1, 1904.2, Figure 1904.2, Table 1904.2

Proposed Change as Submitted

Proponent: Matthew Senecal, P.E., American Concrete Institute (ACI)

Delete and substitute as follows:

~~**1904.1 Exposure categories and classes.** Concrete shall be assigned to exposure classes in accordance with the durability requirements of ACI 318 based on:~~

1. ~~Exposure to freezing and thawing in a moist condition or deicer chemicals;~~
2. ~~Exposure to sulfates in water or soil;~~
3. ~~Exposure to water where the concrete is intended to have low permeability; and~~
4. ~~Exposure to chlorides from deicing chemicals, salt, saltwater, brackish water, seawater or spray from these sources, where the concrete has steel reinforcement.~~

~~**1904.2 Concrete properties.** Concrete mixtures shall conform to the most restrictive maximum water-cementitious materials ratios, maximum cementitious admixtures, minimum air-entrainment and minimum specified concrete compressive strength requirements of ACI 318 based on the exposure classes assigned in Section 1904.1.~~

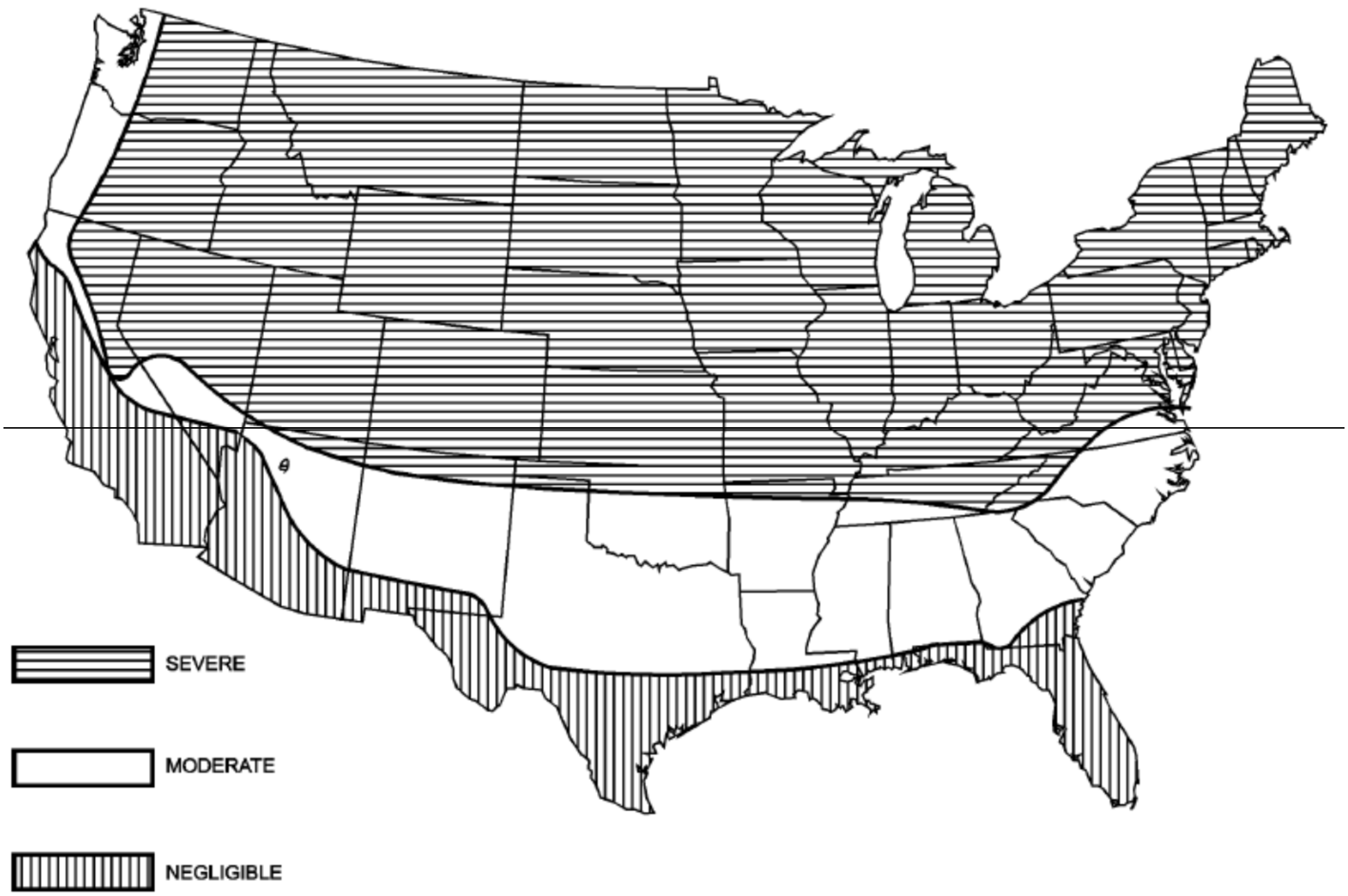
> ~~**Exception:** For occupancies and appurtenances thereto in Group R occupancies that are in buildings less than four stories above grade plane, normal-weight aggregate concrete is permitted to comply with the requirements of Table 1904.2 based on the weathering classification (freezing and thawing) determined from Figure 1904.2 in lieu of the durability requirements of ACI 318.~~

~~TABLE 1904.2~~
~~MINIMUM SPECIFIED COMPRESSIVE STRENGTH (f'_c)~~

~~**TYPE OR LOCATION OF CONCRETE CONSTRUCTION**~~	~~**MINIMUM SPECIFIED COMPRESSIVE STRENGTH (f'_c at 28 days, psi)**~~		
	~~**Negligible exposure**~~	~~**Moderate exposure**~~	~~**Severe exposure**~~
~~Basement walls[c] and foundations not exposed to the weather~~	~~2,500~~	~~2,500~~	~~2,500[a]~~
~~Basement slabs and interior slabs on grade, except garage floor slabs~~	~~2,500~~	~~2,500~~	~~2,500[a]~~
~~Basement walls[c], foundation walls, exterior walls and other vertical concrete surfaces exposed to the weather~~	~~2,500~~	~~3,000[b]~~	~~3,000[b]~~
~~Driveways, curbs, walks, patios, porches, carport slabs, steps and other flatwork exposed to the weather, and garage floor slabs~~	~~2,500~~	~~3,000[b,d]~~	~~3,500[b,d]~~

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

a. ~~Concrete in these locations that can be subjected to freezing and thawing during construction shall be of air-entrained concrete in accordance with Section 1904.2.~~
b. ~~Concrete shall be air entrained in accordance with ACI 318.~~
c. ~~Structural plain concrete basement walls are exempt from the requirements for exposure conditions of Section 1904.2.~~
d. ~~For garage floor slabs where a steel trowel finish is used, the total air content required by ACI 318 is permitted to be reduced to not less than 3 percent, provided the minimum specified compressive strength of the concrete is increased to 4,000 psi.~~



~~**FIGURE 1904.2**~~
~~**WEATHERING PROBABILITY MAP FOR CONCRETE**~~~~[a,b,c]~~

~~a. Lines defining areas are approximate only. Local areas can be more or less severe than indicated by the region classification.~~
~~b. A "severe" classification is where weather conditions encourage or require the use of deicing chemicals or where there is potential for a continuous presence of moisture during frequent cycles of freezing and thawing. A "moderate" classification is where weather conditions occasionally expose concrete in the presence of moisture to freezing and thawing, but where deicing chemicals are not generally used. A "negligible" classification is where weather conditions rarely expose concrete in the presence of moisture to freezing and thawing.~~
~~c. Alaska and Hawaii are classified as severe and negligible, respectively.~~

1904.1 Structural concrete. Structural concrete shall conform to the durability requirements of ACI 318.

1904.2 Nonstructural concrete. The registered design professional shall assign nonstructural concrete a freeze-thaw exposure class, as defined in ACI 318, based on the anticipated exposure of nonstructural concrete. Nonstructural concrete shall have a minimum specified compressive strength, f'_c, of 2500 psi for Class F0; 3000 psi for Class F1; and 3500 psi for Classes F2 and F3. Nonstructural concrete shall be air entrained in accordance with ACI 318.

Reason: This proposal replaces the weathering probability map with ACI 318's performance requirements; removes the exception for structural concrete; and clarifies the durability requirements for nonstructural concrete.

Probability map: The weathering probability map for concrete can be inaccurate since it is possible to have "severe," "moderate," or "negligible" environments in any of the predefined zones shown on the map. ACI 318 requires the designer to classify concrete into one of the freezing and thawing classes as follows:

F0 – Concrete not exposed to freezing-and-thawing cycles
F1 – Concrete exposed to freezing-and-thawing cycles and occasional exposure to moisture

F2 – Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture
F3 – Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture and exposed to deicing chemicals

The concrete classes must be applied by the designer, regardless of geographic location. The commentary to ACI 318 provides further discussion and examples to help the designer determine the appropriate class. It is therefore recommended to remove the map and adopt the ACI 318 approach.

Table: The first and second rows of the table provide limits for interior concrete. Interior concrete is equivalent to Class F0 in ACI 318, which requires a minimum concrete compressive strength of 2500 psi. Therefore, the minimum concrete compressive strength requirements listed in the first two rows are the same as the minimum requirements of ACI 318 and may be removed.

The third row of the table provides an exception for exterior structural concrete walls above or below ground. The exception allows for 3000 psi concrete for any environment other than "negligible" or Class F0. Research[1-2] shows that concrete with a minimum amount of hydrated cement resists the negative effects of freezing and thawing. ACI 318 has determined that 4500 psi concrete provides adequate cement hydration for the range of available concrete mixtures used in construction. It is therefore recommended to remove this exception for structural concrete.

The fourth row of the table states strength limits for exterior nonstructural concrete. ACI 318 does not have durability requirements for nonstructural concrete. Therefore, these limits are not an exception to 318 but a requirement. These limits are simply restated in terms of exposure classes as shown in the revision. The limitation on building category and concrete type have been removed, since this appears to be a misunderstanding of what is required in ACI 318.

References:

1. Klieger, P., 1956, "Curing Requirements for Scale Resistance of Concrete," Highway Research Board Bulletin 150, pp.18-31. (PCA Bulletin 82)
2. Mather, B., 1990, "How to Make Concrete that will be Immune to the Effects of Freezing and Thawing," Paul Klieger Symposium on Performance of Concrete, SP-122, D. Whiting, ed., American Concrete Institute, Farmington Hills, MI, pp. 1-18.

Cost Impact: The code change proposal may increase the cost of construction for structural concrete but decrease the cost for nonstructural concrete. By changing the requirement from geometric location to performance criteria, the cost will increase or decrease depending on location and exposure.

1904.1 (NEW)-S-SENECAL

Public Hearing Results

Committee Action: Approved as Submitted

Committee Reason: This proposal promotes coordination with ACI 318 durability requirements. A public comment is encouraged to bring back the current IBC exception for Group R occupancies with appropriate limitations.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment 1:

Matthew Senecal, American Concrete Institute, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

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 <u>1902 and </u>1905.1.1.

<u>**1902.2 NONSTRUCTURAL CONCRETE.** Any element made of plain or reinforced concrete that is not part of a structural system required to transfer either gravity or lateral loads to the ground.</u>

1904.1 Structural concrete. Structural concrete shall conform to the durability requirements of ACI 318.

> <u>**Exception:** For Group R3 occupancies not more than three stories above grade plane, the specified compressive strength, f'_c, for concrete in basement walls, foundation walls, exterior walls, and other vertical concrete work exposed to the weather shall be not less than 3000 psi.</u>

1904.2 Nonstructural concrete. The registered design professional shall assign nonstructural concrete a freeze-thaw exposure class, as defined in ACI 318, based on the anticipated exposure of nonstructural concrete. Nonstructural concrete shall have a minimum specified compressive strength, f'_c, of 2500 psi for Class F0; 3000 psi for Class F1; and 3500 psi for Classes F2 and F3. Nonstructural concrete shall be air entrained in accordance with ACI 318.

Commenter's Reason: The original proposal asked that exception for residential occupancies be removed along with the change to the durability classification system. During the CDH, the NAHB explained that this "type" of exception is common throughout the IBC. It allows for commercial structures that more closely resemble a residential structure in function and use to be built in accordance with IRC requirements. ACI accepts this explanation. An attempt was made to restore the exception at the CDH; however, the language for the type of occupancies could not be worked out. As stated in the ROH, the committee encouraged ACI and NAHB to submit a public comment to include the exception.

In addition, an attendee at the CDH mentioned that a definition for "nonstructural concrete" may be helpful. ACI accepts this suggestion.

Public Comment 2:

Gary J. Ehrlich, P.E. National Association of Home Builders (NAHB), requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1904.1 Structural concrete. Structural concrete shall conform to the durability requirements of ACI 318.

> <u>**Exception:** For Group R-2 and R-3 occupancies not more than three stories above grade plane, the specified compressive strength, f'c, for concrete in basement walls, foundation walls, exterior walls and other vertical surfaces exposed to the weather shall be not less than 3000 psi.</u>

1904.2 Nonstructural concrete. The registered design professional shall assign nonstructural concrete a freeze-thaw exposure class, as defined in ACI 318, based on the anticipated exposure of nonstructural concrete. Nonstructural concrete shall have a minimum specified compressive strength, f'_c, of 2500 psi for Class F0; 3000 psi for Class F1; and 3500 psi for Classes F2 and F3. Nonstructural concrete shall be air entrained in accordance with ACI 318.

SECTION 202
DEFINITIONS

<u>**NONSTRUCTURAL CONCRETE.** Any element made of plain or reinforced concrete that is not part of a structural system required to transfer either gravity or lateral loads to the ground.</u>

Commenter's Reason: The purpose of this public comment is to restore an exemption from the ACI 318 durability requirements for Group R-2 and R-3 occupancies. A definition for "nonstructural concrete" is also added for increased clarity, so that a code user does not have to go to ACI 318 first to decide whether Section 1904.1 or 1904.2 applies.

Group R-3 occupancies are typically one- and two-family dwellings and townhouses either constructed in jurisdictions that have not adopted the IRC, or that are outside the scope of the IRC limits for the purposes of the structural design. Reasons for the latter include being in areas prone to Category 3 or higher hurricanes, assigned to Seismic Design Category E, irregular dwellings in other moderate or high-Seismic Design Categories, or dwellings that exceed wall and story height limits. These dwellings often are designed without the involvement of an engineer and using the provisions of Section 2308 or prescriptive engineering-based standards such as ICC-600 or the Wood Frame Construction Manual. In these cases, the builder or designer has used prescriptive concrete requirements such as the Section 1904.2 exception and Table 1904.2 that were in the 2012 IBC and previous editions in lieu of purchasing and designing to ACI 318.

Group R-2 covers a range of residential buildings where the dwelling units or sleeping units are occupied for more than a month. Many of the structures covered under this group, such as fraternity and sorority houses, back-to-back rows of townhouses, and low-rise "garden style" condominium and apartment buildings typically containing 8-12 units. These structures are very similar in construction and loading to R-3 structures and often also designed using Section 2308 or other prescriptive standards.

This will maintain consistency between dwellings constructed to the IRC and those designed using the IBC, as well as between Group R-3 structures and similar Group R-2 structures. Otherwise, the IBC through reference to ACI 318 will require 4500 psi for concrete exposed to freeze/thaw action or deicing chemicals in Group R-3 structures, where such concrete has traditionally been designed using 3000 psi concrete. It is also noted that ACI 332 *Building Code Requirements for Residential Concrete*, which is referenced in the IRC, also specifies 3000 psi concrete for this condition. Since neither the IRC nor ACI 332 use the new exposure classes (F0, F1, F2, F3), the traditional "exposed to the weather" language is also retained.

S199-12
Final Action: AS AM AMPC____ D

S202-12
1905.1.1, 1905.1.3, 1905.1.4, 1905.1.9, 1905.1.10

Proposed Change as Submitted

Proponent: Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing self (pbrazil@reidmiddleton.com)

Revise as follows:

1905.1.1 ACI 318, Section 2.2. Modify existing definitions and add the following definitions to ACI 318, Section 2.2.

DESIGN DISPLACEMENT. Total lateral displacement expected for the design-basis earthquake, *as specified by Section 12.8.6 of ASCE 7.*

DETAILED PLAIN CONCRETE STRUCTURAL WALL. *A wall complying with the requirements of Chapter 22, including 22.6.7.*

ORDINARY PRECAST STRUCTURAL WALL. *A precast wall complying with the requirements of Chapters 1 through 18.*

ORDINARY REINFORCED CONCRETE STRUCTURAL WALL. A *cast-in-place* wall complying with the requirements of Chapters 1 through 18.

ORDINARY STRUCTURAL PLAIN CONCRETE WALL. A wall complying with the requirements of Chapter 22, *excluding 22.6.7*.

SPECIAL STRUCTURAL WALL. A cast-in-place or precast wall complying with the requirements of 21.1.3 through 21.1.7, 21.9 and 21.10, 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."*

~~***WALL PIER.*** *A wall segment with a horizontal length-to-thickness ratio of at least 2.5, but not exceeding 6, whose clear height is at least two times its horizontal length.*~~

1905.1.3 ACI 318, Section 21.4. Modify ACI 318, Section 21.4, by renumbering Section 21.4.3 to become 21.4.4 and adding new Sections 21.4.3, ~~21.4.5, 21.4.6 and 21.4.7~~ to read as follows:

> *21.4.3 - 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.*
> 21.4.4 - Elements of the connection that are not designed to yield shall develop at least 1.5 *Sy*.
> ~~*21.4.5 - Wall piers in Seismic Design Category D, E or F shall comply with Section 1905.1.4 of the International Building Code.*~~
> ~~*21.4.6 - Wall piers not designed as part of a moment frame in buildings assigned to Seismic Design Category C shall have transverse reinforcement designed to resist the shear forces determined from 21.3.3. Spacing of transverse reinforcement shall not exceed 8 inches (203 mm). Transverse reinforcement shall be extended beyond the pier clear height for at least 12 inches (305 mm).*~~
>
> ~~***Exceptions:***~~
>
> ~~*1. Wall piers that satisfy 21.13.*~~

2. ~~*Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers and such segments have a total stiffness of at least six times the sum of the stiffnesses of all the wall piers.*~~

~~*21.4.7 - Wall segments with a horizontal length-to-thickness ratio less than 2.5 shall be designed as columns.*~~

~~**1905.1.4 ACI 318, Section 21.9.** Modify ACI 318, Section 21.9, by deleting Section 21.9.8 and replacing with the following:~~

~~*21.9.8 - Wall piers and wall segments.*~~
~~*21.9.8.1 - Wall piers not designed as a part of a special moment frame shall have transverse reinforcement designed to satisfy the requirements in 21.9.8.2.*~~

~~***Exceptions:***~~

1. ~~*Wall piers that satisfy 21.13.*~~
2. ~~*Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers and such segment have a total stiffness of at least six times the sum of the stiffnesses of all the wall piers.*~~

~~*21.9.8.2 - Transverse reinforcement with seismic hooks at both ends shall be designed to resist the shear forces determined from 21.6.5.1. Spacing of transverse reinforcement shall not exceed 6 inches (152 mm). Transverse reinforcement shall be extended beyond the pier clear height for at least 12 inches (305 mm).*~~
~~*21.9.8.3 - Wall segments with a horizontal length-to-thickness ratio less than 2.5 shall be designed as columns.*~~

~~**1905.1.9 ACI 318, Section D.3.3.** Delete ACI 318 Sections D.3.3.4 through D.3.3.7 and replace with the following:~~

~~*D.3.3.4 - The anchor design strength associated with concrete failure modes shall be taken as*~~ $0.75\phi N_n$ ~~*and*~~ $0.75\phi V_n$~~*, where*~~ ϕ ~~*is given in D4.3 or D4.4 and*~~ N_n ~~*and*~~ V_n ~~*are deteremined in accordance with D5.2, D5.3, D5.4, D6.2 and D6.3, assuming the concrete is cracked unless it can be demonstrated that the concrete remains uncracked.*~~
~~*D.3.3.5 - Anchors shall be designed to be governed by the steel strength of a ductile steel element as determined in accordance with D.5.1 and D.6.1, unless either D.3.3.6 or D.3.3.7 is satisfied.*~~

~~***Exceptions:***~~

1. ~~*Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy* Section D.3.3.5.~~
2. ~~*D.3.3.5 need not apply and the design shear strength in accordance with D.6.2.1(c) need not be computed for anchor bolts attaching wood sill plates of bearing or non-bearing walls of light-frame wood structures to foundations or foundation stem walls provided all of the following are satisfied:*~~
   - ~~*2.1. The allowable in-plane shear strength of the anchor is determined in accordance with AF&PA NDS Table 11E for lateral design values parallel to grain.*~~
   - ~~*2.2 The maximum anchor nominal diameter is 5/8 inches (16 mm).*~~
   - ~~*2.3. Anchor bolts are embedded into concrete a minimum of 7 inches (178 mm).*~~
   - ~~*2.4. 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.*~~
   - ~~*2.5. 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.*~~
   - ~~*2.6. The sill plate is 2-inch or 3-inch nominal thickness.*~~

~~*3. Section D.3.3.5 need not apply and the design shear strength in accordance with Section D.6.2.1(c) need not be computed for anchor bolts attaching cold-formed steel track of bearing or non-bearing walls of light-frame construction to foundations or foundation stem walls provided all of the following are satisfied:*~~

~~*3.1. The maximum anchor nominal diameter is 5/8 inches (16 mm).*~~
~~*3.2. Anchors are embedded into concrete a minimum of 7 inches (178 mm).*~~
~~*3.3. Anchors are located a minimum of 13/4 inches (45 mm) from the edge of the concrete parallel to the length of the track.*~~
~~*3.4. Anchors are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the track.*~~
~~*3.5. The track is 33 to 68 mil 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.*~~

~~*4. In light-frame construction, design of anchors in concrete shall be permitted to satisfy D.3.3.8.*~~

~~*D.3.3.6 - Instead of D.3.3.5, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a force level corresponding to anchor forces no greater than the design strength of anchors specified in D.3.3.4.*~~

~~***Exceptions:***~~

~~*1. Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.6.*~~
~~*2. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.6.*~~

~~*D.3.3.7 - As an alternative to D.3.3.5 and D.3.3.6, it shall be permitted to take the design strength of the anchors as 0.4 times the design strength determined in accordance with D.3.3.4.*~~
~~*D.3.3.8 – In light-frame construction, bearing or nonbearing walls, shear strength of concrete anchors less than or equal to 1 inch [25 mm] in diameter of sill plate or track to foundation or foundation stem wall need not satisfy D.3.3.7 when the design strength of the anchors is determined in accordance with D.6.2.1(c).*~~

~~**1905.1.10 ACI 318, Section D.4.2.2.** Delete ACI 318, Section D.4.2.2, and replace with the following:~~

~~*D.4.2.2 - The concrete breakout strength requirements for anchors in tension shall be considered satisfied by the design procedure of D.5.2 provided Equation D-7 is not used for anchor embedments exceeding 25 inches. The concrete breakout strength requirements for anchors in shear with diameters not exceeding 2 inches shall be considered satisfied by the design procedure of D.6.2. For anchors in shear with diameters exceeding 2 inches, shear anchor reinforcement shall be provided in accordance with the procedures of D.6.2.9.*~~

Reason: The purpose for this proposal is to update the 2012 IBC for consistency with ACI 318-11 and as explained below.

1. In IBC Section 1905.1.1, the definition of “wall pier” is deleted because of the definition of “wall pier” in Section 2.2 of ACI 318-11.
2. In IBC Section 1905.1.3, Sections 21.4.5 through 21.4.7 are deleted because of Section 21.4.4 of ACI 318-11, which reads: “In structures assigned to SDC D, E or F, wall piers shall be designed in accordance with 21.9 or 21.13.”
3. IBC Section 1905.1.4 is deleted because of Section 21.9.8 of ACI 318-11, which specifies requirements for wall piers.
4. IBC Section 1905.1.9 is deleted because of Sections D.3.3.4 through D.3.5 of ACI 318-11, which specify seismic design requirements for anchors in structures that are substantially revised from the corresponding provisions in Sections D.3.3.3 through D.3.3.6 of ACI 318-08.

5. IBC Section 1905.1.10 is deleted because of Sections D.4.2.2 and D.4.3 of ACI 318-11, which specify requirements for concrete breakout strength and bond strength that are substantially revised from the corresponding provisions in Section D.4.2.2 of ACI 318-08.

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

1905.1.1-S-BRAZIL.doc

Public Hearing Results

Committee Action: Disapproved

Committee Reason: Disapproved at the proponent's request. This proposal would remove exceptions for light-frame construction that were approved in the last cycle.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Philip Brazil, P.E., S.E., representing self; and Matthew Senecal, P.E., American Concrete Institute, request Approval as Modified by this Public Comment.

Replace the proposal as follows:

1905.1 General. The text of ACI 318 shall be modified as indicated in Sections 1905.1.1 through 1905.1.10.

1905.1.1 ACI 318, Section 2.2. ~~Modify existing definitions and~~ Add the following definitions to ACI 318, Section 2.2.

> ~~**DESIGN DISPLACEMENT.** Total lateral displacement expected for the design-basis earthquake, *as specified by Section 12.8.6 of ASCE 7.*~~
>
> ***DETAILED PLAIN CONCRETE STRUCTURAL WALL.*** *A wall complying with the requirements of Chapter 22, including Section 22.6.7.*
>
> ***ORDINARY PRECAST STRUCTURAL WALL***. *A precast wall complying with the requirements of Chapters 1 through 18.*
>
> ~~**ORDINARY REINFORCED CONCRETE STRUCTURAL WALL**. A *cast-in-place* wall complying with the requirements of Chapters 1 through 18.~~
>
> ~~**ORDINARY STRUCTURAL PLAIN CONCRETE WALL.** A wall complying with the requirements of Chapter 22, *excluding Section 22.6.7.*~~
>
> ~~**SPECIAL STRUCTURAL WALL.** A cast-in-place or precast wall complying with the requirements of 21.1.3 through 21.1.7, 21.9 and 21.10, 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."*~~

1905.1.2 ACI 318, Section 21.1.1. Modify ACI 318 Sections 21.1.1.3 and 21.1.1.7 to read as follows:

> **21.1.1.3**. *Structures assigned to Seismic Design Category A* shall satisfy requirements of Chapters 1 to 19 and 22*; Chapter 21 does not apply*. Structures assigned to *Seismic Design Category* B, C, D, E, or F also shall satisfy 21.1.1.4 through 21.1.1.8, as applicable. *Except for structural elements of plain concrete complying with Section 1905.1.8 of the International Building Code, structural elements of plain concrete are prohibited in structures assigned to Seismic Design Category C, D, E or F*.
>
> **21.1.1.7**. 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 21 does not apply, the following provision shall be satisfied for each structural system designated as part of the seismic-force-resisting system, regardless of the *Seismic Design Category*:
>
> a. Ordinary moment frames shall satisfy 21.2.
> b. Ordinary reinforced concrete structural walls<u>, detailed plain concrete structural walls</u> *and ordinary precast structural walls* need not satisfy any provisions in Chapter 21.

c. Intermediate moment frames shall satisfy 21.3.
d. Intermediate precast *structural* walls shall satisfy 21.4.
e. Special moment frames shall satisfy 21.5 through 21.8.
f. Special structural wall shall satisfy 21.9.
g. Special structural walls constructed using precast concrete shall satisfy 21.10.

All special moment frames and special structural walls shall also satisfy 21.1.3 through 21.1.7.

Commenter's Reason: The public comment builds upon Proposal S203-12, which deleted modifications of ACI 318 from IBC Section 1905 that are no longer needed since they are incorporated into ACI 318-11.

The definitions of *design displacement*, *ordinary reinforced concrete structural wall*, *ordinary structural plain concrete wall* and *special structural wall* are deleted because definitions for them are in Section 2.2 of ACI 318-11.

The definitions for "detailed plain concrete structural wall" and "ordinary precast structural wall" are retained because definitions for them are not in ACI 318-11 but they are listed as seismic force-resisting systems in Table 12.2-1 of AISC 7-10 and their use is permitted in Seismic Design Category B without limitation. Refer to Items A.3, A.6, B.6 and B.9 of Table 12.2-1.

The addition of *detailed plain concrete structural walls* to Item (b) of ACI 318 Section 21.1.1.7 correlates this section with the addition of *detailed plain concrete structural walls* to Section 2.2 of ACI 318.

S202-12

Final Action: AS AM AMPC____ D

S203-12
1905.1, 1905.1.1, 1905.1.3, 1905.1.4

Proposed Change as Submitted

Proponent: Matthew Senecal, P.E., American Concrete Institute (ACI)

Revise as follows:

1905.1 General. The text of ACI 318 shall be modified as indicated in Sections 1905.1.1 through ~~1905.1.10~~ 1905.1.9.

~~***WALL PIER.*** *A wall segment with a horizontal length-tothickness ratio of at least 2.5, but not exceeding 6, whose clear height is at least two times its horizontal length.*~~

1905.1.3 ACI 318, Section 21.4. Modify ACI 318, Section 21.4, by adding new Section 21.4.3 and renumbering existing Section 21.4.3 to become 21.4.4. ~~and adding new Sections 21.4.3, 21.4.5, 21.4.6 and 21.4.7 to read as follows:~~

> *21.4.3 - 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.*
>
> 21.4.4 - Elements of the connection that are not designed to yield shall develop at least 1.5 S_y.
>
> ~~*21.4.5 - Wall piers in Seismic Design Category D, E or F shall comply with Section 1905.1.4 of the International Building Code.*~~
>
> ~~*21.4.6 - Wall piers not designed as part of a moment frame in buildings assigned to Seismic Design Category C shall have transverse reinforcement designed to resist the shear forces determined from 21.3.3. Spacing of transverse reinforcement shall not exceed 8 inches (203 mm). Transverse reinforcement shall be extended beyond the pier clear height for at least 12 inches (305 mm).*~~
>
> ~~***Exceptions:***~~
>
> 1. ~~*Wall piers that satisfy 21.13.*~~
> 2. ~~*Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers and such segments have a total stiffness of at least six times the sum of the stiffnesses of all the wall piers.*~~
>
> ~~*21.4.7 - Wall segments with a horizontal length-to thickness ratio less than 2.5 shall be designed as columns.*~~

~~**1905.1.4 ACI 318, Section 21.9.** Modify ACI 318, Section 21.9, by deleting Section 21.9.8 and replacing with the following:~~

> ~~*21.9.8 - Wall piers and wall segments.*~~
>
> ~~*21.9.8.1 - Wall piers not designed as a part of a special moment frame shall have transverse reinforcement designed to satisfy the requirements in 21.9.8.2.*~~
>
> ~~***Exceptions:***~~
>
> 1. ~~*Wall piers that satisfy 21.13.*~~

2. ~~*Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers and such segments have a total stiffness of at least six times the sum of the stiffnesses of all the wall piers.*~~

~~*21.9.8.2 - Transverse reinforcement with seismic hooks at both ends shall be designed to resist the shear forces determined from 21.6.5.1. Spacing of transverse reinforcement shall not exceed 6 inches (152 mm). Transverse reinforcement shall be extended beyond the pier clear height for at least 12 inches (305 mm).*~~

~~*21.9.8.3 - Wall segments with a horizontal length-to thickness ratio less than 2.5 shall be designed as columns.*~~

Reason: This proposal removes the requirements for wall piers. Wall pier requirements are in 1905 because ACI 318-08 did not address the design of this component. ACI 318 incorporated wall pier design in the 2011 edition. Therefore, these amendments should now be removed.

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

1905.1-S-SENECAL

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

1905.1.3 ACI 318, Section 21.4. Modify ACI 318, Section 21.4, by adding new Section 21.4.3 and renumbering existing Section<u>s</u> 21.4.3 <u>and 21.4.4</u> to become 21.4.4 <u>and 21.4.5, respectively</u>.

21.4.3 - 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.

21.4.4 - Elements of the connection that are not designed to yield shall develop at least 1.5 S_y.

<u>21.4.5 – In structures assigned to SDC D, E, or F, wall piers shall be designed in accordance with 21.9 or 21.13 *in ACI 318*.</u>

(Portions of proposal not shown are unchanged)

Committee Reason: The committee feels that adopting these provisions for wall piers from the consensus standard with fewer modifications allows that process to work. The modification reflects a renumbered section that keeps the ACI 318 provision intact.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Stephen Kerr, representing Structural Engineers Association of California, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1905.1.3 ACI 318, Section 21.4. Modify ACI 318, Section 21.4, by adding new Section<u>s</u> 21.4.3<u>, 21.4.6 and 21.4.7</u> and renumbering existing Sections 21.4.3 and 21.4.4 to become 21.4.4 and 21.4.5, respectively.

21.4.3 - 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.

21.4.4 - Elements of the connection that are not designed to yield shall develop at least 1.5 S_y.

21.4.5 – In structures assigned to SDC D, E, or F, wall piers shall be designed in accordance with 21.9 or 21.13 *in ACI 318*.

21.4.6 - Wall piers not designed as part of a moment frame in buildings assigned to Seismic Design Category C shall have transverse reinforcement designed to resist the shear forces determined from 21.3.3 in ACI 318. Spacing of transverse reinforcement shall not exceed 8 inches (203 mm). Transverse reinforcement shall be extended beyond the pier clear height for at least 12 inches (305 mm).

Exceptions:

1. Wall piers that satisfy 21.13.
2. Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers and such segments have a total stiffness of at least six times the sum of the stiffnesses of all the wall piers.

21.4.7 - Wall segments with a horizontal length-to thickness ratio less than 2.5 shall be designed as columns.

Commenter's Reason: The original code change proposal did not establish why wall pier in lower Seismic Design Categories need not include detailing provision to meet the risk-targeted earthquake throughout the United States. As stated by ACI representative at the CDH, ACI 318 has not developed requirement for wall piers in structures assigned to SDC C. Application of ACI 318 section 21.4.2 does not preclude flexural yielding at intersection of spandrel and wall pier as exhibited in concrete tilt-up frame panels testing. [Dew, Sexsmith, Weiler, 2001]. The flexural hinging can lead to premature shear failure in the wall pier.

While the original wall pier provisions for wall pier were introduced to legacy code 1988 UBC for high seismic regions, and has been part of IBC since the 2000 edition, the 2003 NEHRP provisions included parallel provisions for wall piers in seismic design category C which was adopted into ICC 2006. The provisions are essential to prevent possible buckling of longitudinal reinforcement in wall piers induced from flexural yielding propagated at the intersection of spandrel beam and wall pier.

Provision stated under 21.4.6 and 21.4.7 has been part of the model code since 2006. The provision reflects design and minimum detailing requirements for wall piers in the lower SDC. Since its inclusion in 2006, the provision has been widely used by the tilt-up industry. These two sections stated in this public comment are verbatim from 2012 IBC and should be re-instated.

Reference: Michael Dew, Robert Sexsmith, Gerry Weiler, (2001), "Effect of Hinge Zone Tie Spacing on ductility of Concrete Tilt-up Frame Panels," ACI Structural Journal, Nov.-Dec., 2001, American Concrete Institute, Farmington, IL

S203-12
Final Action: AS AM AMPC____ D

S213-12

Table 1705.3, 1908, 1908.1, 1908.2, Table 1908.2, 1908.3, 1908.4, 1908.5

Proposed Change as Submitted

Proponent: Matthew Senecal, P.E., American Concrete Institute (ACI)

Revise as follows:

TABLE 1705.3
REQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]	IBC REFERENCE
3. Inspection of anchors cast in concrete ~~where allowable loads have been increased or where strength design is used~~.	—	X	ACI 318: 8.1.3, 21.2.8	~~1908.5,~~ 1909.1

(Portions of Table not shown remain unchanged)

Delete without substitution:

~~**SECTION 1908 ANCHORAGE TO CONCRETE—ALLOWABLE STRESS DESIGN**~~

~~**1908.1 Scope.** The provisions of this section shall govern the *allowable stress design* of headed bolts and headed stud anchors cast in normal-weight concrete for purposes of transmitting structural loads from one connected element to the other. These provisions do not apply to anchors installed in hardened concrete or where load combinations include earthquake loads or effects. The bearing area of headed anchors shall be not less than one and one-half times the shank area. Where strength design is used, or where load combinations include earthquake loads or effects, the design strength of anchors shall be determined in accordance with Section 1909. Bolts shall conform to ASTM A 307 or an *approved* equivalent.~~

~~**1908.2 Allowable service load.** The allowable service load for headed anchors in shear or tension shall be as indicated in Table 1908.2. Where anchors are subject to combined shear and tension, the following relationship shall be satisfied:~~

~~*(Ps / Pt)5/3 + (Vs / Vt)* 5/3 ≤ 1 **(Equation 19-1)**~~

~~where:~~

~~*Ps* = Applied tension service load, pounds (N).~~
~~*Pt* = Allowable tension service load from Table 1908.2, pounds (N).~~
~~*Vs* = Applied shear service load, pounds (N).~~
~~*Vt* = Allowable shear service load from Table 1908.2, pounds (N).~~

~~**TABLE 1908.2**~~
~~**ALLOWABLE SERVICE LOAD ON EMBEDDED BOLTS (pounds)**~~

~~BOLT DIAMETER (inches)~~	~~MINIMUM EMBEDMENT (inches)~~	~~EDGE DISTANCE (inches)~~	~~SPACING (inches)~~	~~MINIMUM CONCRETE STRENGTH (psi)~~					
				~~f_c' = 2,500~~		~~f_c' = 3,000~~		~~f_c' = 4,000~~	
				~~Tension~~	~~Shear~~	~~Tension~~	~~Shear~~	~~Tension~~	~~Shear~~
~~1/4~~	~~2-1/2~~	~~1-1/2~~	~~3~~	~~200~~	~~500~~	~~200~~	~~500~~	~~200~~	~~500~~
~~3/8~~	~~3~~	~~2-1/4~~	~~4-1/2~~	~~500~~	~~1,100~~	~~500~~	~~1,100~~	~~500~~	~~1,100~~
~~1/2~~	~~4~~ ~~4~~	~~3~~ ~~5~~	~~6~~ ~~6~~	~~950~~ ~~1,450~~	~~1,250~~ ~~1,600~~	~~950~~ ~~1,500~~	~~1,250~~ ~~1,650~~	~~950~~ ~~1,550~~	~~1,250~~ ~~1,750~~
~~5/8~~	~~4-1/2~~ ~~4-1/2~~	~~3-3/4~~ ~~6-1/4~~	~~7-1/2~~ ~~7-1/2~~	~~1,500~~ ~~2,125~~	~~2,750~~ ~~2,950~~	~~1,500~~ ~~2,200~~	~~2,750~~ ~~3,000~~	~~1,500~~ ~~2,400~~	~~2,750~~ ~~3,050~~
~~3/4~~	~~5~~ ~~5~~	~~4-1/2~~ ~~7-1/2~~	~~9~~ ~~9~~	~~2,250~~ ~~2,825~~	~~3,250~~ ~~4,275~~	~~2,250~~ ~~2,950~~	~~3,560~~ ~~4,300~~	~~2,250~~ ~~3,200~~	~~3,560~~ ~~4,400~~
~~7/8~~	~~6~~	~~5-1/4~~	~~10-1/2~~	~~2,550~~	~~3,700~~	~~2,550~~	~~4,050~~	~~2,550~~	~~4,050~~
~~1~~	~~7~~	~~6~~	~~12~~	~~3,050~~	~~4,125~~	~~3,250~~	~~4,500~~	~~3,650~~	~~5,300~~
~~1-1/8~~	~~8~~	~~6-3/4~~	~~13-1/2~~	~~3,400~~	~~4,750~~	~~3,400~~	~~4,750~~	~~3,400~~	~~4,750~~
~~1-1/4~~	~~9~~	~~7-1/2~~	~~15~~	~~4,000~~	~~5,800~~	~~4,000~~	~~5,800~~	~~4,000~~	~~5,800~~

~~**1908.3 Required edge distance and spacing.** The allowable service loads in tension and shear specified in Table 1908.2 are for the edge distance and spacing specified. The edge distance and spacing are permitted to be reduced to 50 percent of the values specified with an equal reduction in allowable service load. Where edge distance and spacing are reduced less than 50 percent, the allowable service load shall be determined by linear interpolation.~~

~~**1908.4 Increase in allowable load.** Increase of the values in Table 1908.2 by one-third is permitted where the provisions of Section 1605.3.2 permit an increase in allowable stress for wind loading.~~

~~**1908.5 Increase for special inspection.** Where *special inspection* is provided for the installation of anchors, a 100- percent increase in the allowable tension values of Table 1908.2 is permitted. No increase in shear value is permitted.~~

Reason: This proposal removes allowable stress design for anchoring to concrete. This approach to anchor design is not consistent with the standards published by ACI, AISC, or ASCE.

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

1908-S-SENECAL

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This code change removes out of date provisions for concrete anchorage using allowable stress design.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Edwin Huston, National Council of Structural Engineers Association (NCSEA), representing NCSEA Code Advisory Subcommittee – General Requirements Subcommittee, requests Disapproval.

Commenter's Reason: Contrary to the proponent's reason statement on cost impact, removing Table 1908.2 will increase cost of construction, design and plan review time. While the allowable stress design method may be antique, Table 1908.2 provides an allowable bolt value in concrete applicable when wind force governs design and when seismic force governs in lower seismic design categories. The table bolt values offers uniformity in enforcement and may avoid needless design error in using ACI 318 Appendix D. Until Appendix D can be made more user friendly, this proposal should be disapproved.

We urge your disapproval of S213.

S213-12
Final Action: AS AM AMPC____ D

S235-12

2112.5, Table 2112.5 (New), Chapter 35 (New)

Proposed Change as Submitted

Proponent: Timothy N. Seaton, B.S.C.E., Empire Masonry Heaters LLC

Revise as follows:

2112.5 Masonry heater clearance. Combustible materials shall not be placed within 36 inches (765 mm) of the outside surface of a masonry heater in accordance with NFPA 211, Section 8-7 (clearances for solid fuel-burning appliances), and the required space between the heater and combustible material shall be fully vented to permit the free flow of air around all heater surfaces.

Exceptions:

1. Where the masonry heater wall thickness is at least 8 inches (203 mm) thick of solid masonry and the wall thickness of the heat exchange channels is at least 5 inches (127 mm) thick of solid masonry, ~~combustible materials shall not be placed within 4 inches (102 mm) of the outside surface of a masonry heater. A clearance of at least 8 inches (203 mm) shall be provided between the gas-tight capping slab of the heater and a combustible ceiling.~~ or when the wall thicknesses are similarly 4 inches (102 mm) at the firebox and 2 ½ inches (64 mm) at the heat exchange channel but are lined with at least the inner 2 inches (51 mm) and 1 inch (25 mm) respectively of firebrick(ASTM C27 or ASTM C1261) or refractory equivalent, clearances shall be according to Table 2112.5
2. Where masonry heaters *listed* and labeled in accordance with UL 1482 and installed in accordance with the manufacturer's instructions clearances will be as listed.

TABLE 2112.5
MASONRY HEATER CLEARANCES TO COMBUSTIBLE MATERIALS

CONTROLLING STANDARD PROVISIONS	MINIMUM MASONRY HEATER WALL CONSTRUCTION THICKNESS		CLEARANCES FROM COMBUSTIBLE					
			WALLS				CEILINGS	
	Firebox	Channels	Unprotected	Non-combustible wall surface material[b]	Protective shield[c] (from shield)	Both surface[b] and shield[b] (from shield)	Unprotected	Protective shield[c] (from shield)
2112.5 ASTM E 1602 (with NFPA 211)			36” (914 mm)	As per NFPA 211 Section 12.6			As per NFPA 211 Section 12.6	
2112.5.1 ASTM E 1602 (with Exception 1)	8” (203 mm)	5” (127 mm)	4” (102 mm)				8” (203 mm)	
	4” (100 mm) [including 2” (50 mm) firebrick lining[a]]	2.5” (64 mm) [including 1” (25 mm) firebrick lining[a]}	10” (250 mm)	6” (150 mm)	5” (127 mm)	3” (75 mm)	10” (260 mm)	5” (127 mm)
2112.5.2 UL 1482/EN 15250 (with Exception 2)	As per manufacturer		As per listing				As per listing	

a. “Firebrick lining” is a lining constructed of firebrick conforming to ASTM C27 or C1261 or refactor equivalent.

b. “Non-combustible wall surface material” is a wall covering facing the masonry heater made from non-combustible material (Fire Class A) and having at least a 30 minute Fire Resistance Rating

c. <u>“Protective shield” is a non-combustible protective shield placed between the masonry heater and the wall, which extends sideways beyond the heater, and is separated from the wall by at least 1.25 inches (30 mm) and from the floor and ceiling by at least 2 inches (50 mm). The clearance is measured from the shield.</u>

Add new standard to Chapter 35 as follows:

EN

<u>EN 15250-2007 Slow Heat Release Appliances Fired by Solid Fuel - Requirements and Test Methods</u>

Reason: North American masonry heater technology is virtually all sourced in Europe where the devices have been built for centuries. In conformance with typical European standards, ASTM E1602, *Standard Guide for Construction of Solid Fuel Burning Masonry Heaters,* does not stipulate masonry heater wall thickness nor relate it to clearances to combustibles. In contrast to masonry fireplace construction and operation, masonry heater wall thickness does not necessarily relate to surface temperature but instead to the time it takes for the heat to begin radiating from the surface and to the total time radiation will occur. For this reason thicker wall construction may in fact be more dangerous with overfiring situations than thinner wall construction.

Until recent IBC and IRC code revisions, all minimum masonry heater clearances were 4” (102 mm) to surface wall or protective shield as per ASTM E1602. I can locate no documented examples of wall ignition from masonry heaters of any wall thickness at this clearance or under ASTM E1602 as the sole ruling clearance standard.

In the recent IBC/IRC code revisions “NFPA 211, Section 8-7 (clearances for solid fuel-burning appliances)” (*sic)* was made the ruling standard for masonry heater clearances instead of ASTM E1602 even though this former standard was created for wood stoves and similar appliances and had no real application to masonry heaters. This standard stipulates 36” clearance to combustible materials with possible reduction to 12” with approved reduction methods. These clearances may be realistic for metal stoves and similar appliances but are unnecessarily restrictive for masonry heaters which in contrast by definition cannot exceed 230º F (110º C) surface temperatures in normal operation (ASTM E1602 Section 3.2.14).

The recent IBC/IRC revisions created two exceptions to the NFPA 211 rule; 1) for lab tested and listed devices, and 2) for masonry heaters with thick firebox and heat channel walls which by European practice are only used for masonry heaters with large heat storage intended to be fired at very long intervals. This latter class of masonry heaters is built increasingly rarely in Europe as the energy codes were written and tightened there and lower output and more responsive masonry heating was required. The same change in code structure is occurring here in North America, and the 36” clearance stipulation for other than thick walled masonry heaters is making masonry heater construction in new projects and particularly in renovation projects unnecessarily complex and expensive. The typical masonry heater sold is custom in design and cannot support laboratory safety testing.

I am not proposing removing existing code clearance provisions though they have not been lab safety tested and verified (as the code provisions for masonry fireplaces have not). The existing safety tests, UL127 and UL1482 were created for manufactured metal appliances and limited in their application to masonry devices. Instead I am proposing IBC adopt building code provisions from Europe for masonry heater clearances where such clearances have been verified through decades and centuries of use. There is no overall European Union document for code built (as opposed to listed) masonry heater clearances. I am attaching the prevailing Austrian standard TRVB 105:1986, *Technical Regulations for Preventive Fire Protection: Fireplaces for Solid Fuels* as a more conservative European example. I propose these clearances, which are more restrictive than ASTM E1602, be adopted for masonry heaters not covered by the existing IBC language under an expanded Exception 1. Please note that in this Austrian standard “fireplaces” refers collectively to iron stoves, open fireplaces, and masonry heaters.

Note also that the ASTM C27 and C1261 firebrick citation is borrowed from existing IBC/IRC fireplace provisions. C1261 is no longer listed in the ASTM standards volume and may not have been renewed.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] 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, 2012.

2112.5 #1-S-SEATON.doc

Public Hearing Results

Note: For staff analysis of the content of EN 15250 relative to CP#28, Section 3.6, please visit: http://www.iccsafe.org:8888/cs/codes/Documents/2012-13cycle/Proposed-A/00a_updates.pdf

Committee Action: Disapproved

Committee Reason: Disapproval is at the proponent’s request in order work on technical issues and needed improvements in a public comment.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Rod Zander, representing New England Hearth & Soapstone LLC, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

2112.5 Masonry heater clearance. Combustible materials shall not be placed within 36 inches (765 mm) of the outside surface of a masonry heater in accordance with NFPA 211, Section 8-7 (clearances for solid fuel-burning appliances), and the required space between the heater and combustible material shall be fully vented to permit the free flow of air around all heater surfaces.

Exceptions:

1. Where the masonry heater wall <u>firebox</u> thickness is at least 8 inches (203 mm) thick of solid masonry and the wall thickness of the heat exchange channels is at least 5 inches (127 mm) thick of solid masonry, or when the wall thicknesses are similarly 4 inches (102 mm) at the firebox and 2 ½ inches (64 mm) at the heat exchange channel but are lined with at least the inner 2 inches (51 mm) and 1 inch (25 mm) respectively of firebrick <u>complying with</u> ~~(~~ASTM C27 or ASTM C1261~~)~~ or refractory equivalent, clearances shall be ~~according to~~ <u>in accordance with</u> Table 2112.5.
2. Where masonry heaters <u>are</u> *listed* and labeled in accordance with UL 1482 and installed in accordance with the manufacturer's instructions clearances will be as listed.

(Portions of proposal not shown remain unchanged)

Commenter's Reason: As an experienced masonry heater builder and chair of the ASTM task group on masonry heaters I deal with building codes and officials all of the time.
The code as written is very confusing and difficult to interpret both for the building official and the masonry heater builder trying to work with the code and build a safe appliance. The code as written will not necessarily assure a safe installation. (See below*)
I have worked in Europe under the Austrian fire safety standard TRVB 105. (Adopted in 1986 see attached) this regulation was created in an open, consensus process directly analogous to NFPA 211 and their process.
I do recommend that ICC reference TRVB 105 as it is more stringent, comprehensive and would be easily interpreted in the field by the code officials. This proposal incorporates information from TRVB 105 but should explicitly reference it.
There is urgency regarding this matter as homes are getting tighter and smaller with the new energy codes. The results are that masonry heaters are getting smaller and more responsive. Smaller heaters with thinner wall thicknesses are no more or less safe than those with thick walls.

*There is a definite safety issue with the current code language. The current code only requires solid masonry construction of the walls. It does not require a refractory lining of any sort or any specification of what a masonry material is. This could lead to a failure of the firebox and heat exchange system, possibly causing a fire in the building. Masonry heater fire box and heat exchange channels are similar to a fireplace and chimney construction but will see higher service conditions. The ICC codes for fireplaces and chimneys would not allow this construction.
There is no data to backup the current code requirements of the stipulated 8"-5"wall thicknesses and the 4" clearance to combustibles. I am aware of no instances of unsafe installations when the clearances as outlined in ASTM 1602 are followed regardless of wall thicknesses.

Prior to 2006, ICC referenced ASTM E1602 for masonry heater safety clearances. In 2006, ICC effectively mandated, without supporting research or safety data, that masonry heaters with massive walls were safe at 4" clearance while other heater constructions could only be considered safe at 36" clearance. This is entirely contrary to European experience and practice in both regards, and unfairly discriminates between masonry heater manufacturers. ICC should revisit this matter.

S235-12
Final Action: AS AM AMPC____ D

S240-12
1604.3.3, 2203.2, 2207.1, 2207.1.1 (New), 2207.2, 2207.3, 2207.4, 2207.5,

Proposed Change as Submitted

Proponent: Bonnie Manley, P.E., American Iron and Steel Institute, representing Steel Joist Institute (bmanley@steel.org)

Revise as follows:

1604.3.3 Steel. The deflection of steel structural members shall not exceed that permitted by AISC 360, AISI S100, ASCE 8, SJI CJ~~-1.0~~, SJI JG~~-1.1~~, SJI K~~-1.1~~ or SJI LH/ DLH~~-1.1~~, as applicable.

2203.2 Protection. Painting of structural steel members shall comply with the requirements contained in AISC 360. Painting of open-web steel joists and joist girders shall comply with the requirements of SJI CJ~~-1.0~~, SJI JG~~-1.1~~, SJI K~~-1.1~~ and SJI LH/DLH~~-1.1~~. Individual structural members and assembled panels of cold-formed steel construction shall be protected against corrosion in accordance with the requirements contained in AISI S100. Protection of cold-formed steel light-frame construction shall also comply with the requirements contained in AISI S200.

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

1. SJI-CJ~~-1.0~~
2. SJI-K~~-1.1~~
3. SJI-LH/DLH~~-1.1~~
4. SJI-JG~~-1.1~~

<u>2207.1.1 Seismic design.</u> <u>Where required, the seismic design of buildings shall be in accordance with the additional provisions of Section 2205.2 or 2211.6.</u>

2207.2 Design. The *registered design professional* shall indicate on the *construction documents* the steel joist and/or steel joist girder designations from the specifications listed in Section 2207.1 and shall indicate the requirements for joist and joist girder design, layout, end supports, anchorage~~, non-SJI standard~~ bridging, bridging termination connections and bearing connection design to resist uplift and lateral loads. These documents shall indicate special requirements as follows:

1. Special loads including:
   - 1.1. Concentrated loads;
   - 1.2. Nonuniform loads;
   - 1.3. Net uplift loads;
   - 1.4. Axial loads;
   - 1.5. End moments; and
   - 1.6. Connection forces.
2. Special considerations including:
   - 2.1. Profiles for ~~nonstandard~~ joist and joist girder configurations ~~(standard joist and joist girder are as indicated in the SJI catalog)~~ <u>that differ from those defined by the SJI specifications listed in Section 2207.1</u>;
   - 2.2. Oversized or other nonstandard web openings; and
   - 2.3. Extended ends.
3. <u>Live and total load</u> deflection criteria ~~for live and total loads~~ for ~~non-SJI standard~~ joists <u>and joist girder configurations that differ from those defined by the SJI specifications listed in Section 2207.1</u>.

2207.3 Calculations. The steel joist and joist girder manufacturer shall design the steel joists and/or steel joist girders in accordance with the ~~current~~ SJI specifications ~~and load tables~~ listed in Section 2207.1 to support the load requirements of Section 2207.2. The *registered design professional* ~~may~~ shall be permitted to require submission of the steel joist and joist girder calculations as prepared by a *registered design professional* responsible for the product design. If requested by the *registered design professional*, the steel joist manufacturer shall submit design calculations with a cover letter bearing the seal and signature of the joist manufacturer's *registered design professional*. In addition to ~~standard~~ the design calculations submitted under ~~this~~ seal and signature, ~~submittal~~ of the following shall be included:

1. ~~Non-SJI standard~~ Bridging ~~details~~ design that differs from the SJI specifications listed in Section 2207.1 (e.g.for cantilevered conditions, net uplift, etc.).
2. Connection ~~details~~ design for:
   - 2.1. ~~Non-SJI standard~~ Connections that differ from the SJI specifications listed in Section 2207.1 (e.g.flushframed or framed connections);
   - 2.2. Field splices; and
   - 2.3. Joist headers.

2207.4 Steel joist drawings. Steel joist placement plans shall be provided to show the steel joist products as specified on the *construction documents* and are to be utilized for field installation in accordance with specific project requirements as stated in Section 2207.2. Steel joist placement plans shall include, at a minimum, the following:

1. Listing of all applicable loads as stated in Section 2207.2 and used in the design of the steel joists and joist girders as specified in the *construction documents*.
2. Profiles for ~~nonstandard~~ joist and joist girder configurations ~~(standard joist and joist girder configurations are as indicated in the SJI catalog)~~ that differ from those defined by the SJI specifications listed in Section 2207.1.
3. Connection requirements for:
   - 3.1. Joist supports;
   - 3.2. Joist girder supports;
   - 3.3. Field splices; and
   - 3.4. Bridging attachments.
4. Live and total load deflection criteria ~~for live and total loads~~ for ~~non-SJI standard~~ joists and joist girder configurations that differ from those defined by the SJI specifications listed in Section 2207.1.
5. Size, location and connections for all bridging.
6. Joist headers.

Steel joist placement plans do not require the seal and signature of the joist manufacturer's *registered design professional*.

2207.5 Certification. At completion of manufacture, the steel joist manufacturer shall submit a *certificate of compliance* in accordance with Section 1704.2.5.2 stating that work was performed in accordance with *approved construction documents* and with SJI ~~standard~~ specifications listed in Section 2207.1.

Reason: This code change is primarily editorial in nature with the intent to clarify and streamline the requirements for steel joists. Major changes include the following:

- Correction of short titles in Section 2207.1, 1604.3.3 and 2203.2 to reflect the appropriate short title listing in Chapter 35 and correction of SJI address in Chapter 35.
- Deletion of reference to the SJI catalog – it is not an adopted reference.
- Deletion of reference to the load tables; they are now incorporated into the relevant SJI specifications.
- Elimination of the vague terms "nonstandard", "non SJI standard", and "standard" used throughout the section. These terms are not defined. To clarify what is intended, a reference to the requirements found in the SJI specifications listed in Section 2207.1 is substituted.

Addition of "joist girders" to Section 2207.2, Item 3 and Section 2207.4, Item 4 for consistency.

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

2207.1-S-MANLEY.doc

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

2207.2 Design. The *registered design professional* shall indicate on the *construction documents* the steel joist and/or steel joist girder designations from the specifications listed in Section 2207.1 and shall indicate the requirements for joist and joist girder design, layout, end supports, anchorage, non-SJI standard bridging, bridging termination connections and bearing connection design to resist uplift and lateral loads. These documents shall indicate special requirements as follows:

1. Special loads including:
   1.1. Concentrated loads;
   1.2. Nonuniform loads;
   1.3. Net uplift loads;
   1.4. Axial loads;
   1.5. End moments; and
   1.6. Connection forces.
2. Special considerations including:
   2.1. Profiles for joist and joist girder configurations that differ from those defined by the SJI specifications listed in Section 2207.1;
   2.2. Oversized or other nonstandard web openings; and
   2.3. Extended ends.
3. Live and total load deflection criteria for joists and joist girder configurations that differ from those defined by the SJI specifications listed in Section 2207.1.

(Portions of proposal not shown are unchanged)

Committee Reason: This proposal clarifies the intent of steel joist requirements in Section 2207 by making series of editorial improvements.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Bonnie E. Manley, American Iron and Steel Institute, representing Steel Joist Institute requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

SECTION 2207
STEEL JOISTS

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

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

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

2207.2 Design. The *registered design professional* shall indicate on the *construction documents* the steel joist and/or steel joist girder designations from the specifications listed in Section 2207.1 and shall indicate the requirements for joist and joist girder design, layout, end supports, anchorage, ~~non-SJI standard~~ bridging design that differs from the SJI specifications listed in Section 2207.1, bridging termination connections and bearing connection design to resist uplift and lateral loads. These documents shall indicate special requirements as follows:

1. Special loads including:

    1.1. Concentrated loads;
    1.2. Nonuniform loads;
    1.3. Net uplift loads;
    1.4. Axial loads;
    1.5. End moments; and
    1.6. Connection forces.
2. Special considerations including:
    2.1. Profiles for joist and joist girder configurations that differ from those defined by the SJI specifications listed in Section 2207.1;
    2.2. Oversized or other nonstandard web openings; and
    2.3. Extended ends.
3. Live and total load deflection criteria for joist~~s~~ and joist girder configurations that differ from those defined by the SJI specifications listed in Section 2207.1.

2207.3 Calculations. The steel joist and joist girder manufacturer shall design the steel joists and/or steel joist girders in accordance with the SJI specifications listed in Section 2207.1 to support the load requirements of Section 2207.2. The *registered design professional* shall be permitted to require submission of the steel joist and joist girder calculations as prepared by a *registered design professional* responsible for the product design. If requested by the *registered design professional*, the steel joist manufacturer shall submit design calculations with a cover letter bearing the seal and signature of the joist manufacturer's *registered design professional*. In addition to the design calculations submitted under seal and signature, the following shall be included:

1. Bridging design that differs from the SJI specifications listed in Section 2207.1 (e.g. for cantilevered conditions, net uplift, etc.).
2. Connection design for:
    2.1. Connections that differ from the SJI specifications listed in Section 2207.1 (e.g.flushframed or framed connections);
    2.2. Field splices; and
    2.3. Joist headers.

2207.4 Steel joist drawings. Steel joist placement plans shall be provided to show the steel joist products as specified on the *construction documents* and are to be utilized for field installation in accordance with specific project requirements as stated in Section 2207.2. Steel joist placement plans shall include, at a minimum, the following:

1. Listing of all applicable loads as stated in Section 2207.2 and used in the design of the steel joists and joist girders as specified in the *construction documents*.
2. Profiles for joist and joist girder configurations that differ from those defined by the SJI specifications listed in Section 2207.1.
3. Connection requirements for:
    3.1. Joist supports;
    3.2. Joist girder supports;
    3.3. Field splices; and
    3.4. Bridging attachments.
4. Live and total load deflection criteria joist~~s~~ and joist girder configurations that differ from those defined by the SJI specifications listed in Section 2207.1.
5. Size, location and connections for all bridging.
6. Joist headers.

Steel joist placement plans do not require the seal and signature of the joist manufacturer's *registered design professional*.

2207.5 Certification. At completion of manufacture, the steel joist manufacturer shall submit a *certificate of compliance* in accordance with Section 1704.2.5.2 stating that work was performed in accordance with *approved construction documents* and with SJI specifications listed in Section 2207.1.

Commenter's Reason: The purpose of this public comment is to ensure that consistent language is used throughout Section 2207. The proposal was approved as modified, with the return of the language "non SJI standard" in Section 2207.2. While it is recognized that the deletion of this phrase in the original proposal expanded the applicability of the section beyond what was preferred, the language "non SJI standard" is awkward and unclear. Rather, we would like to see the section include the same exact phrasing that is used and was approved in Section 2207.3 Item 1 – "bridging design that differs from the SJI specifications listed in Section 2207.1."

S240-12
Final Action: AS AM AMPC____

S244-12
2210.1.1.3 (New), Chapter 35 (New)

Proposed Change as Submitted

Proponent: Thomas Sputo, Ph.D., P.E., S.E., Steel Deck Institute

Add new text as follows:

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 standard to Chapter 35 as follows:

SDI

SDI-C-2011 Standard for Composite Steel Floor Deck Slabs

Reason: This Standard contains provisions for the design and construction of composite steel deck-slabs of concrete on composite steel deck, and reflects current design and construction industry practices.

The 2012 IBC contains no provisions for the design of composite slabs on steel deck. The previous reference standard that was contained in the 2009 IBC was deleted from the 2012 IBC. Designers and code officials currently must rely on Section 104.11 of the IBC to use this very common structural system. Adding this Standard to the 2015 IBC would fill this gap.

This Standard is an update to the previous 2006 version of this Standard, and was developed and approved through a consensus process under ANSI guidelines, and complies with ICC CP 28. This Standard, along with all other Steel Deck Institute (SDI) Standards, will be available for free download from the SDI website for all parties.

For review purposes, the SDI C-2011 Standard that is being proposed is available for download and review from this website: http://www.sputoandlammert.com/standard.html

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] 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, 2012.

2210.1.1.3 (NEW)-S-SPUTO

Public Hearing Results

Note: For staff analysis of the content of SDI C relative to CP#28, Section 3.6, please visit: http://www.iccsafe.org:8888/cs/codes/Documents/2012-13cycle/Proposed-A/00a_updates.pdf

Committee Action: **Approved as Submitted**

Committee Reason: The committee feels it is good to include the proposed reference standard for composite slab construction now that it has completed the ANSI standard process.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Todd Hawkinson, Hawkinson Associates, LLC, representing self, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

2210.1.1.3.1 Sampling and inspection of mix proportions containing steel fiber added to concrete for uniform distribution: Sampling of steel fiber added to concrete shall be in accordance with ASTM C172/C172M except as modified and required below:

1. Separate samples of 1 ft^3 shall be taken after discharge of approximately 25 percent and 75 percent of 4 cubic yards of concrete. A minimum of two samples shall be taken every 4 cubic yards depending on placement method. For fibers added at the plant, every other truck shall be sampled. For fibers added at the project site, every truck shall be sampled as outlined here.
2. The samples shall remain separate to determine fiber quantity. Immediately rinse all concrete and remove aggregates and all noticeable water from the samples prior to weighing. The steel fibers shall be patted dry. Steel fibers shall be weighed.
3. Visually inspect the fibers for damage from mixing.
4. Weigh the fibers and compute the ratio of the fiber quantity by weight to volume of concrete sampled. The ratio of fiber to cubic yard of concrete shall be reported and this ratio shall be within 5 percent of the amount of added fiber specified in the design or mix requirements.

2210.1.1.3.2 Sampling and inspection of mix proportions containing micro fiber added to concrete for uniform distribution. Sampling of macro fiber added to concrete shall be in accordance with ASTM C172/C172M except as modified and required below:

1. Separate samples of 1 ft^3 shall be taken after discharge of approximately 25 percent and 75 percent of 4 cubic yards of concrete. A minimum of two samples shall be taken every 4 cubic yards depending on placement method. For fibers added at the plant, every other truck shall be sampled. For fibers added at the project site, every truck shall be sampled as outlined here.
2. The samples shall remain separate to determine fiber quantity. Immediately rinse all concrete and remove aggregates and all noticeable water from the samples prior to weighing. The macro fibers shall be patted dry. Macro fibers shall be weighed.
3. Visually inspect the fibers for damage from mixing.
4. Weigh the fibers and compute the ratio of the fiber quantity by weight to volume of concrete sampled. The ratio of fiber to cubic yard of concrete shall be reported and this ratio shall be within 5 percent of the amount of added fiber specified in the design or mix requirements.

2210.1.1.3.3 Inspection of the Vertical Distribution of Fibers Added to Concrete. Sampling of field placed concrete for verification of the vertical distribution of steel and macro fibers in concrete shall be as follows:

1. Separate samples of 0.5 ft^3 of the field placed concrete shall be taken randomly immediately after placing into forms and prior to finishing. The use of a plastic or metal container placed on the forms/metal deck shall be utilized to collect the sample.
2. One sample per 5 cubic yards of pour shall be taken and shall follow the same procedures as noted in Sections 2210.1.1.3.1 and 2210.1.1.3.2and modified as follows:
3. The sample shall be laid horizontal, poured on to a horizontal surface for visual inspection.
4. The sample then shall follow the procedures as listed in Section 2210.1.1.3.1, items 1 and 2.
5. Weigh the fibers and compute the ratio of the fiber quantity by weight to volume of concrete sampled. The ratio of fiber to cubic yard of concrete shall be reported and this ratio shall be within 5 percent of the amount of added fiber specified in the design or mix requirements.

(Portions of proposal not shown remain unchanged)

Commenter's Reason: It is imperative that the fiber distribution in the concrete be verified during and after placement. The distribution, both uniformly and vertically needs to be inspected.

Given that there is no standard in the United States that inspects or tests for the distribution of fibers in the mix, none that I have been able to find and in the absence of available standards that would require the user, engineer/designer, or owner to meet certain minimum performance requirements or quality control measures, this building code then must provide for those requirements.

S244-12

Final Action: AS AM AMPC____ D

S253-12
2303.2, Chapter 35 (New)

Proposed Change as Submitted

Proponent: Marcelo M. Hirschler, GBH International (gbhint@aol.com)

Revise as follows:

2303.2 Fire-retardant-treated wood. *Fire-retardant-treated wood* is any homogeneous wood product which, when impregnated with chemicals by a pressure process or other means during manufacture, ~~shall have, when tested in accordance with ASTM E 84 or UL 723, a *listed* flame spread index of 25 or less and show no evidence of significant progressive combustion when the test is continued for an additional 20-minute period. Additionally, the flame front shall not progress more than 101/2 feet (3200 mm) beyond the centerline of the burners at any time during the test~~ complies with the requirements of ASTM E 2768 and is listed.

Add new standard to Chapter 35 as follows:

ASTM

E2768-2011 Standard Test Method for Extended Duration Surface Burning Characteristics of Building Materials (30 min Tunnel Test)

Reason: ASTM has now issued a test method, ASTM E2768, which contains the three requirements discussed in section 2303.2, namely that a product be tested in accordance with ASTM E84 or UL 723, and exhibit a flame spread index of 25 or less, show no evidence of significant progressive combustion when the test is continued for 30 minutes (i.e. an additional 20-minute period over the standard ASTM E84 duration of 10 minutes) and that the flame front not progress more than 101/2 feet (3200 mm) beyond the centerline of the burners at any time during the test.

Note that products listed as fire-retardant treated wood to UL 723 or to ASTM E84 (with the additional requirements shown above) will be able to continue to be listed to ASTM E2768 without having to be retested as the ASTM E2768 test method contains all of those requirements. Therefore, this code proposal is basically simple clarification.

The addition of the requirement that fire-retardant treated wood must be a “homogeneous” product is necessary to ensure that products that are coated or only partially impregnated with chemicals are not considered “fire-retardant treated wood” as they are not.

Note that there also needs to be consistency between the definition of fire-retardant treated wood and the requirements in this Chapter 23. At the last cycle it was established that it is important that the code not place a requirement regarding the means of manufacture and the definition at present in Chapter 2 discusses purely “pressure treated wood”. A separate proposal has been made to change the definition. The two changes can be made independently.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] 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, 2012.

2303.2-S-HIRSCHLER.doc

Public Hearing Results

Note: For staff analysis of the content of ASTM E 2768 relative to CP#28, Section 3.6, please visit: http://www.iccsafe.org:8888/cs/codes/Documents/2012-13cycle/Proposed-A/00a_updates.pdf

Committee Action: **Disapproved**

Committee Reason: Disapproval is consistent with action taken on G25-12 and G26-12. Adding the proposed standard to the section on fire-retardant-treated wood is a little premature. The current language seems clear, but the proposed wording is not. Questions that were raised about the standard, like testing required on one or more surfaces, were not clarified.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment 1:

Marcelo M. Hirschler, GBH International, Craig McIntyre, McIntyre Associates and Kris Owen, Lonza Wood Protection, request Approval as Modified by this Public Comment.

Replace the proposal as follows:

2303.2 Fire-retardant-treated wood. *Fire-retardant-treated wood* is any wood product which, when impregnated with chemicals by a pressure process or other means during manufacture shall have, when tested in accordance with ASTM E84 or UL 723, a *listed* flame spread index of 25 or less and show no evidence of progressive combustion when the test is continued for an additional 20-minute period. Additionally, the flame front shall not progress more than 101/2 feet (3200 mm) beyond the centerline of the burners at any time during the test. <u>Alternately, *fire-retardant treated wood* is any wood product which, when impregnated with chemicals by a pressure process or other means during manufacture, is listed and complies with the requirements of ASTM E2768 on both the top and bottom surfaces</u>.

Add new standard to Chapter 35 as follows:

ASTM

<u>E2768-2011 Standard Test Method for Extended Duration Surface Burning Characteristics of Building Materials (30 min Tunnel Test)</u>

Commenter's Reason: ASTM E2768 was developed by the ASTM committee on fire standards and the code change provides the following:

1. A modification of ASTM E84 that is identical to what has been used and referenced in the IBC and IRC for many years.
2. Passing ASTM E2768 requires that the test specimen be tested to ASTM E84 for an extended period (30 minutes instead of 10 minutes) and that the test specimen comply with: a flame spread index not exceeding 25, a flame front that does not progress more than 101/2 feet (3200 mm) beyond the centerline of the burners at any time during the test and shows no evidence of significant progressive combustion on any surface tested.
3. ASTM E2768 states that, when the flame front does not progress more than 10 ½ feet beyond the centerline of the burners that is evidence of no significant progressive combustion. That is the way each fire test lab who has conducted the test for the last many years has produced the report.
4. The test now known as ASTM E2768 was commonly referred to as the "30-minute E84 tunnel test". However, ASTM E84 has no provisions for extending the test to a 30 minute duration.
5. The "Extended Test Method E84 test" is increasingly being used in requirements that are not limited to fire-retardant-treated wood products, such as ignition-resistant materials in the IWUIC and the California Building codes.
6. Beyond what is stated in ASTM E2768 (section 13.1.2) there has been no definition or clarification or interpretation of what constitutes "significant progressive combustion" anywhere, including in the codes or in any other known document.
7. There has never been a requirement for a material or product to meet either the fire tube test (ASTM E69, Standard Test Method for Combustible Properties of Treated Wood by the Fire-Tube Apparatus) or the "White House" test (NFPA 276, Standard Method of Fire Tests for Determining the Heat Release Rate of Roofing Assemblies with Combustible Above-Deck Roofing Components).
8. In order for a product to be able to be listed as fire-retardant treated wood (FRTW) it needs to meet the requirements of this section (and always has).
9. The public comment does not delete the existing requirements for FRTW nor does it require a product to be listed anew to ASTM E2768: it is simply an equivalent alternative.
10. The public comment will require a product to comply with the requirements on both the top and the bottom surfaces and for it to be impregnated with chemicals and, therefore, it will not be able to be complied with by coated products.

Public Comment 2:

Timothy T. Earl, GBH International, requests Approval as Modified by this Public Comment.

Replace the proposal as follows:

2303.2 Fire-retardant-treated wood. *Fire-retardant-treated wood* is any wood product which, when impregnated with chemicals by a pressure process or other means during manufacture, shall have, when tested in accordance with ASTM 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 101/2 feet (3200 mm) beyond the centerline of the burners at any time during the test..

Commenter's Reason: All the testing labs that conduct the "extended ASTM E84 test" use the concept that, when the flame front does not progress more than 10 ½ feet beyond the centerline of the burners that is evidence of no significant progressive combustion. They have used that concept for many years. Therefore the additional requirement is redundant and causes confusion. This public comment does nothing more than eliminate the redundant requirement stating "and show no evidence of significant progressive combustion", and retains the remainder of the section exactly as it is in the 2012 code.

S253-12

Final Action: AS AM AMPC____ D

S254-12
2303.4.3

Proposed Change as Submitted

Proponent: Larry Wainright, Qualtim, representing Structural Building Components Association (lwainright@qualtim.com)

Revise as follows:

2303.4.3 Truss submittal package. The truss submittal package provided by the truss manufacturer shall consist of each individual truss design drawing, the truss placement diagram, the permanent individual truss member restraint/bracing method and details and any other structural details germane to the trusses; and, as applicable, the cover/truss index sheet. The submittal package shall be submitted to the *registered design professional in responsible charge* for final approval prior to fabrication of trusses.

Reason: The purpose of this proposal is to help close the gap in communication that many times exists whereby the RDP does not get the truss submittal package for review to ensure the truss package meets the intent of the building design. The RDP should always have the opportunity to review these prior to fabrication. The language in this proposal is taken from the North Carolina Building Code where the issue of RDP approval has been thoroughly vetted.

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

2303.4.3-S-WAINRIGHT.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The proposal creates conflicts and the building official can't regulate what happens with outside parties.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Larry Wainright, Qualtim, representing Structural Building Components Association (SBCA), requests Approval as Modified by this Public Comment.

Replace the proposal as follows:

2303.4.3 Truss submittal package. The truss submittal package provided by the truss manufacturer shall consist of each individual truss design drawing, the truss placement diagram, the permanent individual truss member restraint/bracing method and details and any other structural details germane to the trusses; and, as applicable, the cover/truss index sheet. Where required by the owner, building design professional or building official, a designation shall be provided to show that the submittal package has been reviewed by the registered design professional in responsible charge for general conformance to the construction documents.

Commenter's Reason: The proponent asked for disapproval of this code change at the Code development hearings in order to work out problems with the originally proposed language. The purpose of this proposal is to help close the gap in communication that many times exists whereby the RDP does not get the truss submittal package for review to ensure the truss package meets the intent of the building design. The RDP should always have the opportunity to review these prior to fabrication. The language has been modified to address the concerns expressed by the committee and other stakeholders.

S254-12
Final Action: AS AM AMPC____ D

S256-12
Table 2304.6.1

Proposed Change as Submitted

Proponent: Edward L. Keith, P.E., APA – The Engineered Wood Association (ed.keith@apawood.org)

Revise as follows:

TABLE 2304.6.1
MAXIMUM NOMINAL DESIGN WIND SPEED, V_{asd} PERMITTED FOR WOOD STRUCTURAL PANEL WALL SHEATHING USED TO RESIST WIND PRESSURES [a,b,c]

b. The table is based on wind pressures acting toward and away from building surfaces in accordance with Section 30.7 of ASCE 7. Lateral requirements shall be in accordance with Section 2305 or Section 2308. The table was developed based on the requirement that the specified wood structural panels would alone resist 100% of the applied wind load. Evaluation includes stud strength, nail withdrawal, nail head pull-through, and the sheathing deflection criteria of *l*/120 in accordance with Table 1604.3, where *l* = distance between studs.

(Portions of table and footnotes not shown remain unchanged)

Reason: This code change is proposed to clarify the basis on which Table 2304.6.1 was developed and approved so as to provide guidance for any materials that are intended to establish equivalency to this table in accordance with Section 104.11 of the IBC.

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

T2304.6.1-S-KEITH.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: Providing guidance for determining equivalency is generally not bad, but the proposed wording is mainly commentary.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Edward L. Keith, APA The Engineered Wood Association, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

TABLE 2304.6.1
MAXIMUM NOMINAL DESIGN WIND SPEED, V_{asd} PERMITTED FOR WOOD STRUCTURAL PANEL WALL SHEATHING USED TO RESIST WIND PRESSURES [a,b,c]

b. The table is based on wind pressures acting toward and away from building surfaces in accordance with Section 30.7 of ASCE 7. Lateral requirements shall be in accordance with Section 2305 or Section 2308. ~~The table was developed based on the requirement that the specified~~ Specified wood structural panels and fastening to framing are sized to ~~would alone~~ resist 100% of the applied wind load, ~~. Evaluation includes~~ including stud strength, nail withdrawal, nail head pull-through, and the sheathing deflection limit between studs of ~~criteria of~~ *l*/120 ~~in accordance with Table 1604.3,~~ where *l* = distance between studs.

(Portions of table and footnotes not shown remain unchanged)

Commenter's Reason: Table 2304.6.1 is somewhat unique in the conventional construction provisions of the IBC, in that it provides a prescriptive structural solution for wood structural panel wall sheathing. Panels selected and based on this table will provide the structural capacity necessary to resist 100% of the wind load acting normal to the wall. The proposed changes do not change the table that was previously reviewed, approved, and published by ICC. Therefore, the justification of the table, which is not the subject of this public comment, is not repeated here.

With the advent and acceptance in the code of the concept of a structural "system approach" to wind resistance, a large number of engineers, designers and building officials are looking to the prescriptive provisions of the building code to provide them with solutions for traditional single element solutions and multi-part assembly solutions to meet the Section 1609.1 provisions of the building code. This is the provision that requires buildings to be designed to withstand the minimum wind loads.

Because of the relatively new "system approach" concept, confusion has arisen over the applicability of Table 2304.6.1. Is gypsum board required behind the sheathing? Is the use of this table applicable over all siding products or only those with specific wind ratings? Table 1604.3 provides 3 different deflection requirements for wind load, what is the basis of this table? Can stucco be applied over the sheathing thicknesses permitted by this table? These are a few of the questions that we have been asked by building officials and designers over the last two code cycles. While it is not possible to write code to answer all the questions, we have attempted to make the basis of this table clear with our code change proposal. This should, at least, resolve many of the questions relating to this table by making it more clear and transparent.

We were opposed by two camps at the Code Development Hearings:

1. The first camp was adamant that making the basis of the table was not appropriate through the use of the proposed footnote. We challenge this argument by asking to claimant to find a single table in the IBC that has footnotes (most of them) where the footnotes are NOT used to provide the basis for the table and make it more useable. Need examples? I opened the code randomly and the first three tables I found were the below:

Footnote a to Table 1610.1 tells the user that the lateral soil loads given in the table are based on moist conditions.

Footnote a to Table 1609.1.2 tells the user that the table above is based on 140 mph wind speeds and a 45-foot mean roof height. Footnote b tells the user that the table is based on fastener placement 1 inch in from the edge. Footnote c provides the anchor embedment length upon which the table is based.

Footnote a to Table 2306.3(3) tells the user that the values given are based on short-term wind or seismic loading. Footnote c provides the framing spacing basis for the whole table except for those with footnote d. Footnote e tells us that blocked values are based on ALL edges blocked or over framing. Footnotes f and g provide similar guidance about staple geometry that must be met to make that table valid.

All of these footnotes have one thing in common with the footnotes we are proposing – They all provide necessary information for the user of the table to ensure that it is used in a proper and safe manner. All we ask is that we be permitted to do the same for this table.

2. The second camp was those opponents who were concerned that providing the basis for this table would provide a de facto standard for other products that were not able to meet this "standard". This argument is spurious for a couple of different reasons. First off, this table is unique. There are no other single-product tables of this kind in the IBC and any other organizations that want to propose similar tables may do so and should clearly establish the basis for this table as they see fit. The argument to keep this table purposely vague in order to permit some future table to be equally vague for purposes of gaining a market advantage should not be a precedent that this code body should endorse. Secondly, it is the code process and ultimately the building officials who will decide whether this future proposed table is appropriate based on its own merit, not the basis for a pre-existing table that has been in the code for 3 code cycles.

You will also note that an attempt was made to clean up the language in the footnote, hoping to placate at least some of the opponents' issues.

We are simply trying to clarify the basis and use of this existing table to ensure its safe and proper use. We are responding to inquiries by building officials, designers and builders. We ask you to overturn the committee's recommendation for disapproval

S256-12
Final Action: AS AM AMPC____ D

S257-12

2301.2, 2308.2.1, Table 2304.9.1, 2304.7.2.1(New), 2304.7.2.1.1 (New), Figure 2304.7.2.1.1 (New)

Proposed Change as Submitted

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

Revise as follows:

2301.2 General design requirements. The design of structural elements or systems, constructed partially or wholly of wood or wood-based products, shall be in accordance with one of the following methods:

1. *Allowable stress design* in accordance with Sections 2304, 2305 and 2306.
2. *Load and resistance factor design* in accordance with Sections 2304, 2305 and 2307.
3. *Conventional light-frame construction* in accordance with Sections 2304 and 2308.

   Exception: Buildings designed in accordance with the provisions of the AF&PA WFCM <u>and Section 2304.7.2.1</u> shall be deemed to meet the requirements of the provisions of Section 2308.

4. The design and construction of log structures shall be in accordance with the provisions of ICC 400.

2308.2.1 Nominal design wind speed greater than 100 mph (3-second gust). Where V_{asd} as determined in accordance with Section 1609.3.1 exceeds 100 mph (3-second gust), the provisions of either AF&PA WFCM, or the ICC 600 are permitted to be used. Wind speeds in Figures 1609A, 1609B, and 1609C shall be converted in accordance with Section 1609.3.1 for use with AF&PA WFCM or ICC 600. <u>Section 2304.7.2.1 shall apply to roof sheathing attachment when using the AF&PA WFCM or ICC 600.</u>

TABLE 2304.9.1
FASTENING SCHEDULE

CONNECTION	FASTENING[a,m]	LOCATION
31. Wood structural panels and particleboard[b] Subfloor, roof and wall sheathing (to framing) <u>Where V_{ult} equals or exceeds 130 mph, wood structural panel roof sheathing shall be fastened in accordance with Section 2304.7.2.1</u> Single floor (combination subfloor-underlayment to framing)		

(Portions of Table not shown remain unchanged)

<u>**2304.7.2.1 Wood structural panel roof sheathing attachment.** Where V_{ult} equals or exceeds 130 mph, wood structural panels used as roof sheathing shall be installed with joints staggered and fastened in accordance with Section 2304.7.2.1.1.</u>

<u>**2304.7.2.1.1 Sheathing fastenings.** Wood structural panel sheathing shall be fastened to roof framing with 8d annular ring-shank nails at 6 inches on center at edges and 6 inches on center at intermediate framing. Ring-shank nails shall have the following minimum dimensions:</u>

1. 0.113 inch nominal shank diameter
2. Ring diameter of 0.012 over shank diameter
3. 16 to 20 rings per inch
4. 0.280 inch full round head diameter
5. 2 inch nail length

Where roof framing with a specific gravity, 0.42 ≤ G < 0.49 is used, spacing of ring-shank fasteners shall be 4 inches on center in nailing zone 3 in accordance with Figure 2304.7.2.1.1 where V_{ult} is 130 mph or greater.

Exceptions:

1. Where roof framing with a specific gravity, 0.42 ≤ G < 0.49 is used, spacing of ring-shank fasteners shall be permitted at 12 inches on center at intermediate framing in nailing zone 1 for any V_{ult} and in nailing zone 2 for V_{ult} less than or equal to 140 mph in accordance with Figure 2304.7.2.1.1.
2. Where roof framing with a specific gravity, G ≥ 0.49 is used, spacing of ring-shank fasteners shall be permitted at 12 inches on center at intermediate framing in nailing zone 1 for any V_{ult} and in nailing zone 2 for V_{ult} less than or equal to 150 mph in accordance with Figure 2304.7.2.1.1.
3. Where roof framing with a specific gravity, G ≥ 0.49 is used, 8d common or 8d hot dipped galvanized box nails at 6 inches on center at edges and 6 inches on center at intermediate framing shall be permitted for V_{ult} less than or equal to 120 mph in accordance with Figure 2304.7.2.1.1.
4. Where roof diaphragm requirements necessitate a closer fastener spacing.

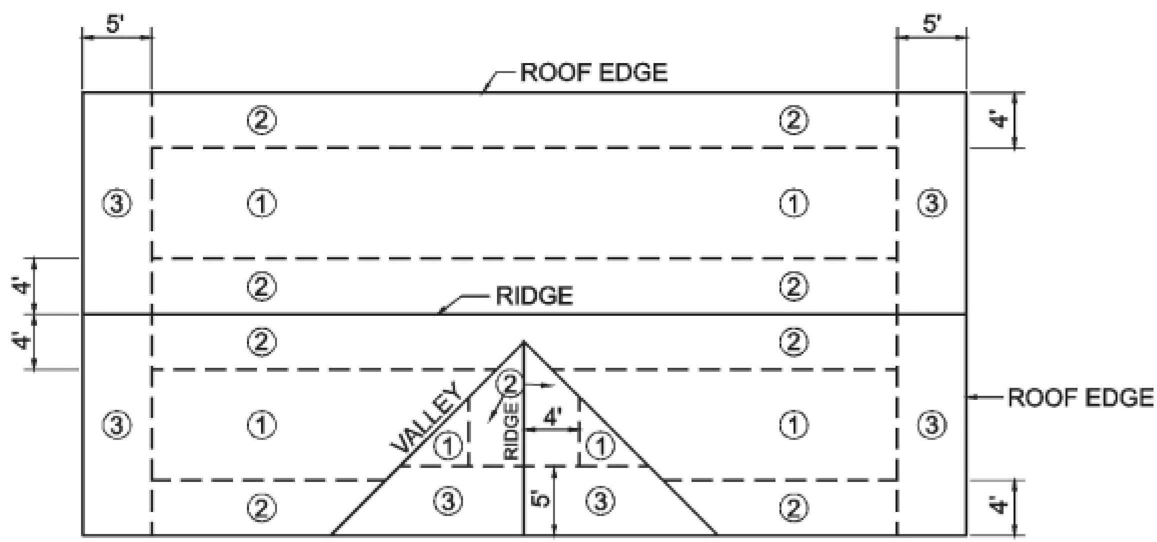


FIGURE 2304.7.2.1.1 ROOF SHEATHING NAILING ZONES

Reason: This proposed modification, if approved, will significantly improve the performance of wood structural panel roofs when subjected to high wind loads. It does so at a minimal to negligible cost which provides an extremely generous benefit/cost ratio. The requirements are based on hundreds of true wood structural panel tests. Extensive roof sheathing fastening tests at Clemson University (Reinhold 2000 – 2002, McKinley 2001) and at the International Hurricane Center – Florida International University (Reinhold, Alvarez 2003) compared the Mean Failure Pressure in psf for roof sheathing panels using both the 8d common and the 8d ring shank nails spaced at 6 inches as prescribed by the code. Sheathing consisted of 5/8 inch thick plywood attached to nominal 2x4 Southern Yellow Pine rafters.

The results of these tests were as follows:

(1) Mean ultimate uplift capacity for panels attached with 8d common nails at 6 inch spacing: 126 pounds per square foot
(2) Mean ultimate uplift capacity for panels attached with 8d ring shank nails at 6 inch spacing: 292 pounds per square foot

This shows a 131% improvement in performance when 8d ring shank nails are used instead of the currently prescribed 8d common nails.

Requiring the use of 8d ring shank nails would result in an almost negligible increase in cost. While variations will occur regionally, it's estimated that the cost increase will be less than $10 for 2000 square foot roof.

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

2301.2-S-STAFFORD.doc

Public Hearing Results

Committee Action: Disapproved

Committee Reason: This code change would require the use of specialty nails where other fasteners could be used.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

T. Eric Stafford, T. Eric Stafford & Associates, LLC, representing Insurance Institute for Business and Home Safety (IBHS), requests Approval as Submitted.

Commenter's Reason: We are seeking Approval as Submitted for S257-12. The primary purpose of this proposed code change was to significantly improve the performance of wood structural panel roofs when subjected to high wind loads at a minimal to negligible cost increase. Extensive roof sheathing tests at Clemson University (Reinhold 2000 – 2002, McKinley 2001) and at the International Hurricane Center – Florida International University (Reinhold, Alvarez 2003) showed a 131% improvement in panel uplift capacity when ring shank nails are used to fasten the roof deck instead of comparable smooth shank nails. This 131% improvement results in a cost increase of about $10 for a 2000 square foot roof, which provides an extremely generous benefit/cost ratio.

Much of the argument in opposition claimed that the nail was not covered by ASTM F 1667 and would require a specialty nail, both of which are not correct. Deformed shank nails are specifically covered by ASTM F 1667. Section 10.3 in ASTM F 1667, *Altered Shapes and Dimensions*, specifically addresses mechanically formed or deformed nail shanks. It is also worth noting that Note c to Table 2304.9.1 calls for common or deformed shank nails for wood structural panel roof sheathing. Proposed Section 2304.7.2.1.1 specifies the dimensional properties of the nail (shank diameter, ring diameter, etc.), which essentially standardizes this particular nail to be consistent with nails used for the panel tests. During the panel tests, these nails were readily available at local home improvement centers.

S257-12
Final Action: AS AM AMPC____ D

S258-12
2302.1, Table 2304.7(4)

Proposed Change as Submitted

Proponent: John Mulder, Intertek Testing Services, NA, Inc., representing International Standards Organization Technical Committee 77, *Products in Fibre-reinforced Cement,* and self

Revise as follows:

2302.1 Definitions. For the purposes of this chapter, and as used elsewhere in this code the following terms are defined in Chapter 2:

FIBER-CEMENT PRODUCTS

TABLE 2304.7(4)
ALLOWABLE SPAN FOR WOOD STRUCTURAL PANEL COMBINATION SUBFLOOR-UNDERLAYMENT (SINGLE FLOOR)[a, b]
(Panels Continuous Over Two or More Spans and Strength Axis Perpendicular to Supports)

For SI: 1 inch = 25.4 mm, 1 pound per square foot = 0.0479 kN/m^2.

a. Spans limited to value shown because of possible effects of concentrated loads. Allowable uniform loads based on deflection of $1/360$ of span is 100 pounds per square foot except allowable total uniform load for $1^1/_8$-inch wood structural panels over joists spaced 48 inches on center is 65 pounds per square foot. Panel edges shall have approved tongue-and-groove joints or shall be supported with blocking, unless $^1/_4$-inch minimum thickness wood panel-type or fiber-cement underlayment or $1^1/_2$ inches of approved cellular or lightweight concrete is placed over the subfloor, or finish floor is $^3/_4$-inch wood strip.

(Portions of table not shown remain unchanged)

Reason: A revision to Table 2304.7(4) is proposed to include “fiber-cement underlayment”. The term “fiber-cement products” is proposed to be included in the definitions here consistent with the definition published in the Terminology Standard ASTM C1154-06, *Standard Terminology for Non-Asbestos Fiber-Reinforced Cement Products* (see attached Standard) and also proposed for revision in Chapter 2 of the IBC code. The current footnote does not clearly describe the allowable type of permitted underlayment. The inclusion of references to ”wood panel-type” and “fiber-cement” clarifies the types of recognized products permitted in this type of Code-compliant subfloor/underlayment application (see attached ICC-ES ESR-1381[reference Section 4.3], ESR-2280[reference Sections 4.2.2.1 and 4.2.3.1], and ESR-2292[reference Section 4.2]). “See the ICC-ES website (http://www.icc-es.org/) to gain access to the referenced ESR reports. “

Cost Impact: The code change proposal will not increase the cost of construction because the proposed addition of fiber-cement underlayment to the table footnote only provides for the choice and use of a type of underlayment currently used in this type of application and permitted in Evaluation Service Reports.

2302.1-T2304.7(4)-S-MULDER.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The current wording is generic and does not exclude fiber-cement products. The proposed wording may exclude other products.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

John Mulder representing Intertek Testing Services NA, Inc. and self, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

TABLE 2304.7(4)
ALLOWABLE SPAN FOR WOOD STRUCTURAL PANEL COMBINATION SUBFLOOR-UNDERLAYMENT (SINGLE FLOOR)a, b
(Panels Continuous Over Two or More Spans and Strength Axis Perpendicular to Supports)
(No change to table)

a. Spans limited to value shown because of possible effects of concentrated loads. Allowable uniform loads based on deflection of 1/360 of span is 100 pounds per square foot except allowable total uniform load for $1^1/_8$-inch wood structural panels over joists spaced 48 inches on center is 65 pounds per square foot. Panel edges shall have approved tongue-and-groove joints or shall be supported with blocking, unless ¼-inch minimum thickness wood panel-type ~~or fiber-cement~~ underlayment or fiber-cement underlayment complying with ASTM C1288 or 1½ inches of approved cellular or lightweight concrete is placed over the subfloor, or finish floor is ¾-inch wood strip.

(footnotes not shown remain unchanged)

Commenter's Reason: The current footnote does not clearly describe the allowable type of permitted underlayment. The additional inclusion of a reference to "fiber-cement" clarifies the types of recognized products permitted in this type of Code-compliant subfloor/underlayment application (see attached ICC-ES ESR-1381[reference Section 4.3], ESR-2280[reference Sections 4.2.2.1 and 4.2.3.1], and ESR-2292[reference Section 4.2]).

S258-12
Final Action: AS AM AMPC____ D

S260-12
2304.9.6

Proposed Change as Submitted

Proponent: Jay Crandell, ARES Consulting, representing Foam Sheathing Committee (jcrandell@aresconsulting.biz)

Revise as follows:

2304.9.6 Load path. Where wall framing members are not continuous from foundation sill to roof, the members shall be secured to ensure a continuous load path. Where required, sheet metal clamps, ties or clips shall be formed of galvanized steel <u>not less than 0.0179 inch (0.45 mm) minimum thickness</u> or other *approved* corrosion-resistant material ~~not less than 0.040 inch (1.01 mm) nominal thickness~~ <u>capable of resisting the applied loads</u>.

Reason: The code needs to allow thinner steel based on performance to, when possible, avoid interference of uplift straps with fastening/installation of interior and exterior finishes and sheathings. AISI Standard S105 Product Data permits minimum steel thickness of 0.0179 inches thick for structural and non-structural applications. In addition, 24CFR Section 3280.305 also permits uplift straps of minimum 26 gage (0.0179 inch thick) for manufactured homes even in the highest of wind zones. The current minimum 0.040 inch thickness requirement is not consistent with existing industry consensus standards and needs to be changed such that minimum required steel thickness is governed by performance needed for a specific application.

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

2304.9.6-S-CRANDELL.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee believes that additional background on the current minimum steel tie thickness could help in evaluating this proposal.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment 1:

Jay H. Crandell, ARES Consulting, American Chemistry Council – Foam Sheathing Committee, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

2304.9.6 Load path. Where wall framing members are not continuous from foundation sill to roof, the members shall be secured to ensure a continuous load path. Where required, sheet metal clamps, ties or clips shall be formed of galvanized steel ~~not less than 0.0179 inch (0.45 mm) minimum thickness~~ or other *approved* corrosion-resistant material ~~capable of resisting the applied loads~~.

Commenter's Reason: This PC focuses the subject sentence on its intended purpose: to simply require corrosion resistant connectors where required. The level of corrosion resistance is currently not specified in the code because the amount of resistance depends on use conditions. Similarly, a minimum steel thickness should not be required because the thickness required depends on the use conditions (i.e., performance required to provide the necessary continuous load path). Some applications may require greater thickness and others thinner than the currently stated 0.040" thickness (see original code text).

This proposal was disapproved at the first hearing with the following reason statement by the structural CDC*: "The committee believes that additional background on the current minimum steel tie thickness could help in evaluating this proposal."* The background on the current code text is unclear. Apparently, this section was added during the drafting of the 2000 IBC and the reason for the current minimum 0.040" thickness was not found. However, the current minimum thickness limit of 0.040" is not consistent with the cold-formed steel industry standard minimum base steel thicknesses or available minimum thicknesses of approved connectors (refer to AISI S201 standard for example). Thus, the current language unnecessarily restricts or conflicts with accepted design practice and existing approved materials. Finally, the proposed language "capable of resisting the applied loads" (not in the current code) is deleted from the original proposal because this requirement is addressed by design requirements found elsewhere in the code and is unnecessary and redundant.

Public Comment 2:

Randall Shackelford, Simpson Strong-Tie Co., requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

2304.9.6 Load path. Where wall framing members are not continuous from foundation sill to roof, the members shall be secured to ensure a continuous load path. Where required, sheet metal clamps, ties or clips shall be formed of galvanized steel ~~not less than 0.0179 inch (0.45 mm) minimum thickness~~, or other *approved* corrosion-resistant material, not less than 0.0329 inch (0.0836 mm) base metal thickness ~~capable of resisting the applied loads~~.

Commenter's Reason: The intent of this change is to only change the required thickness of steel in this section. The current reference to 0.040" thick steel is not a standard thickness according to the newest AISI Product Data Standard, S201.
The assumption is that the thickness was added to represent galvanized 20 gage steel. The term "gage" is no longer a steel thickness designation. What was traditionally 20 ga is now designated as 33 mils. The base metal thickness for 33 mils according to the newest AISI Product Standard is 0.0329 inches. See table below:

Table B2-1
Standard Thickness

Designation Thickness	Minimum Base Steel Thickness		Design Thickness	
	(inch)	(mm)	(inch)	(mm)
18	0.0179	0.455	0.0188	0.478
27	0.0269	0.683	0.0283	0.719
30	0.0296	0.752	0.0312	0.792
33	0.0329	0.836	0.0346	0.879
43	0.0428	1.087	0.0451	1.146
54	0.0538	1.367	0.0566	1.438
68	0.0677	1.720	0.0713	1.811
97	0.0966	2.454	0.1017	2.583
118	0.1180	2.997	0.1242	3.155

S260-12
Final Action: AS AM AMPC____ D

S265-12
Table 2304.9.1

Proposed Change as Submitted

Proponent: Charles S. Bajnai, Chesterfield County, VA, representing ICC Building Code Action Committee (bajnaic@chesterfield.gov)

Delete and substitute as follows:

~~**TABLE 2304.9.1**~~
~~**FASTENING SCHEDULE**~~

Table 2304.9.1
FASTENING SCHEDULE

	DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION
		ROOF	
1	Blocking between ceiling joists or rafters to top plate	3-8d common (2.5" x 0.131"); or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, 7/16" crown	at each end, toenail
2	Ceiling joists to top plate	3-8d common (2.5" x 0.131"); or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, 7/16" crown	per joist, toenail
3	Ceiling joist not attached to parallel rafter, laps over partitions (no thrust) (see Section 2308.10.4.1, Table 2308.10.4.1)	3-16d common (3.5" x 0.162"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, 7/16" crown	Face nail
4	Ceiling joist attached to parallel rafter (heel joint) (see Section 2308.10.4.1, Table 2308.10.4.1)	Per table 2308.10.4.1	Face nail
5	Collar tie to rafter	3-10d common (3" x 0.148"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, 7/16" crown	Face nail
6	Rafter or roof truss to top plate (See Section 2308.10.1, Table 2308.10.1)	3-10 common (3" x 0.148"); or 3-16d box (3.5" x 0.135"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131 nails; or 4-3" 14 gage staples, 7/16" crown	Toenail[c]
7	Roof rafters to ridge valley or hip rafters; or, roof rafter to 2-inch ridge beam	2-16d common (3.5" x 0.162"); or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, 7/16" crown; or	End nail
		3-10d common (3.5" x 0.148"); or 3-16d box (3.5" x 0.135"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, 7/16" crown	Toenail

	DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION
		WALL	
8	Stud to stud (not at braced wall panels)	16d common (3.5” x 0.162”);	24” o.c. face nail
		10d box (3" x 0.128"); or 3” x 0.131” nails; or 3-3” 14 gage staples, 7/16” crown	16” o.c. face nail
9	Stud to stud and abutting studs at intersecting wall corners (at braced wall panels)	16d common (3.5” x 0.162”); or	16” o.c. face nail
		16d box (3.5” x 0.135”); or	12” o.c. face nail
		3” x 0.131” nails; or 3-3” 14 gage staples, 7/16” crown	12” o.c. face nail
10	Built-up header (2-inch to 2-inch header)	16d common (3.5" x 0.162"); or	16” o.c. each edge, face nail
		16d box (3.5” x 0.135”)	12” o.c. each edge, face nail
11	Continuous header to stud	4-8d common (2.5" x 0.131"); or 4-10d box (3" x 0.128")	Toenail
12	Top plate to top plate	16d common (3.5" x 0.162"); or	16” o.c. face nail
		10d box (3" x 0.128"); or 3” x 0.131” nails; or 3” 14 gage staples, 7/16” crown	12” o.c. face nail
13	Top plate to top plate, at end joints	8-16d common (3.5" x 0.162"); or 12-10d box (3" x 0.128"); or 12-3” x 0.131” nails; or 12-3” 14 gage staples, 7/16” crown	Face nail on each side of end joint (minimum 24” lap splice length each side of end joint)
14	Bottom plate to joist, rim joist, band joist or blocking (not at braced wall panels)	16d common (3.5" x 0.162"); or	16” o.c. face nail
		16d box (3.5" x 0.135"); or 3” x 0.131” nails; or 3” 14 gage staples, 7/16” crown	12” o.c. face nail
15	Bottom plate to joist, rim joist, band joist or blocking at braced wall panels	2-16d common (3.5" x 0.162"); or 3-16d box (3.5" x 0.135"); or 4-3” x 0.131” nails; or 4-3” 14 gage staples, 7/16” crown	16” o.c. face nail
16	Stud to bottom plate	4-8d common (2.5" 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	Toenail
		2-16d common (3.5" x 0.162"); or 3-10d box (3" x 0.128"); or 3-3” x 0.131” nails; or 3-3” 14 gage staples, 7/16” crown	End nail
17	Top or bottom plate to stud	2-16d common (3.5" x 0.162"); or 3-10d box (3" x 0.128"); or 3-3” x 0.131” nails; or 3-3” 14 gage staples, 7/16” crown	End nail
18	Top plates, laps at corners and intersections	2-16d common (3.5" x 0.162"); or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, 7/16” crown	Face nail
19	1” brace to each stud and plate	2-8d common (2.5" x 0.131"); or 2-10d box (3" x 0.128"); or 2-3" x 0.131" nails; or	Face nail

	DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION	
		2-3" 14 gage staples, 7/16” crown		
20	1” x 6” sheathing to each bearing	2-8d common (2.5" x 0.131"); or 2-10d box (3" x 0.128")	Face nail	
21	1” x 8” and wider sheathing to each bearing	3-8d common (2.5" x 0.131"); or 3-10d box (3" x 0.128")	Face nail	
	FLOOR			
22	Joist to sill, top plate, or girder	3-8d common (2.5" x 0.131"); or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, 7/16” crown	Toenail	
23	Rim joist, band joist, or blocking to sill or top plate	8d common (2.5" x 0.131"); or 10d box (3" x 0.128"); or 3" x 0.131" nails; or 3" 14 gage staples, 7/16” crown	6” o.c., toenail	
24	1” x 6” subfloor or less to each joist	2-8d common (2.5" x 0.131"); or 3-10d box (3" x 0.128")	Face nail	
25	2” subfloor to joist or girder	2-16d common (3.5" x 0.162")	Face nail	
26	2” planks (plank & beam – floor & roof)	2-16d common (3.5" x 0.162")	At each bearing, face nail	
27	Built-up girders and beams, 2-inch 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, 7/16” 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, 7/16” crown	Face nail at ends and at each splice	
28	Ledger strip supporting joists or rafters	3-16d common (3.5" x 0.162"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, 7/16” crown	At each joist or rafter, face nail	
29	Joist to band joist or rim joist	3-16d common (3.5" x 0.162"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, 7/16” crown	End nail	
30	Bridging to joist	2-8d common (2.5" 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	Each end, toenail	
	Wood structural panels (WSP), subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framing[a]			
			Edges (inches)	Intermediate supports (inches)
31	3/8” – 1/2"	6d common or deformed (2” x 0.113”) (subfloor and wall)	6	12
		8d box or deformed (2.5" x 0.113") (roof)	6	12

	DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION	
		2 3/8" x 0.113" nail (subfloor and wall)	6	12
		1 ¾" 16 gage staple, 7/16" crown (subfloor and wall)	4	8
		2 3/8 x 0.113" nail (roof)	4	8
		1 ¾" 16 gage staple, 7/16" crown (roof)	3	6
32	19/32" – 3/4"	8d common (2.5" x 0.131"); or 6d deformed (2" x 0.113)	6	12
		2 3/8" x 0.113" nail; or 2" 16 gage staple, 7/16" crown	4	8
33	7/8" – 1 1/4"	10d common (3" x 0.148"); or 8d deformed (2.5" x 0.131")	6	12
		Other exterior wall sheathing		
34	1/2" fiberboard sheathing[b]	1 ½" galvanized roofing nail (7/16" head diameter; or 6d common (2" x 0.113"); or 1 ¼" 16 gage staple with 7/16" or 1" crown	3	6
35	25/32" fiberboard sheathing[b]	1 ¾" galvanized roofing nail (7/16" diameter head); or 8d common (2.5" x 0.131"); or 1 ½" 16 gage staple with 7/16" or 1" crown	3	6
		Wood structural panels, combination subfloor underlayment to framing		
36	3/4" and less	8d common (2.5" x 0.131"); or6d deformed (2" x 0.113")	6	12
37	7/8" – 1"	8d common (2.5" x 0.131"); or 8d deformed (2 ½" x 0.131")	6	12
38	1 1/8" – 1 ¼"	10d common (3" x 0.148"); or 8d deformed (2 ½" x 0.131")	6	12
		Panel Siding to Framing		
39	½" or less	6d corrosion-resistant siding (1 7/8" × 0.106"); or 6d corrosion-resistant casing (2" × 0.099")	6	12
40	5/8"	8d corrosion-resistant siding (2 3/8" × 0.128"); or 8d corrosion-resistant casing (2 1/2" × 0.113")	6	12
		Interior Paneling		
41	¼"	4d casing (11/2" × 0.080"); or 4d finish (11/2" × 0.072")	6	12
42	3/8"	6d casing (2" × 0.099"); or 6d finish (Panel supports at 24 inches)	6	12

a. Nails spaced at 6 inches at intermediate supports where spans are 48 inches or more. For nailing of wood structural panel and particleboard diaphragms and shear walls, refer to Section 2305. Nails for wall sheathing are permitted to be common, box, or casing.
b. Spacing shall be 6 inches on center on the edges and 12 inches on center at intermediate supports for nonstructural applications. Panel supports at 16 inches (20 inches if strength axis in the long direction of the panel, unless otherwise marked).
c. Where a rafter is fastened to an adjacent parallel ceiling joist in accordance with this schedule and the ceiling joist is fastened to the top plate in accordance with this schedule, the number of toenails in the rafter shall be permitted to be reduced by one nail.

Reason: The ICC Building Code Action Committee sought to reformat and correlate the current fastening schedule for wood frame construction in Chapter 23 with the current fastening schedule in the IRC. The organization of the IRC table was thought to be easier to use, and it was generally acknowledged that it may help users of both codes if the tables more closely resembled each other in format and content.

Descriptions of specified fastening and their capacities in the IBC and IRC tables were compared. In developing the proposed new table, the committee tried to make as few technical changes as possible while reorganizing and reformatting the IBC table to look more like the IRC table. Care was taken to retain, for the most part, all fastening alternatives currently in the IBC, while at the same time adding appropriate alternatives that appear in the IRC for the same connection, if they were missing.

To attain complete coordination between the two tables was not possible because certain technical changes that would have been required were beyond the chosen scope of the committee's work. However, the proposed table is much closer to the IRC table and the committee will look at the IRC table in the Group B changes to attempt further correlations between the two.

When inconsistencies or apparent anomalies were discovered between tables or within the IBC table itself, in general the following principles were applied:

a. attempt to establish a reference common nail specification for each connection where it appeared to be lacking;
b. provide box nails alternatives, if lacking, where possible
c. retain all current alternatives for power-driven and staple alternatives (though in a few cases the number or size of fastener was adjusted to be consistent with the IRC or to achieve consistency within the IBC table itself based on other entries);
d. in creating box nail alternatives where they currently are missing, for simplicity assume 10d box nails (3" x 0.128") to be equivalent to 3" x 0.131" power-driven fasteners;
e. take into account calculated connection capacities. (These were also compared to the engineered connections specified in the AWC Wood Frame Construction Manual for like connections.)

Finally, this proposed IBC table is much cleaner and more complete than the current table. Besides adding many fastener alternatives, many detailed and difficult-to-use footnotes in the current table were eliminated since their content was incorporated directly into the proposed table.

The following three tables are provided: i) the proposed IBC Table 2304.9.1 with an additional column of notes explaining how it correlates to the existing IBC table, ii) the existing IBC Table 2304.9.1 with an additional column of notes explaining how it correlates to the proposed IBC table, and iii) the existing IRC table, shown for reference.

Proposed Table 2304.9.1 with additional column of explanation:

	DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION	*Notes:*
		ROOF		
1	Blocking between ceiling joists or rafters to top plate	3-8d common (2.5" x 0.131"); or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, 7/16" crown	at each end, toenail	*-Nailing from IBC Row 11.* *-10d box equivalent to 8d common added.*
2	Ceiling joists to top plate	3-8d common (2.5" x 0.131"); or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, 7/16" crown	per joist, toenail	*-Nailing from IBC Row 15.* *-10d box equivalent to 8d common added.* *-Correct power driven number from 5 to 3.*
3	Ceiling joist not attached to parallel rafter, laps over partitions (no thrust) (for parallel rafter case see Section 2308.10.4.1, Table 2308.10.4.1)	3-16d common (3.5" x 0.162"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, 7/16" crown	Face nail	*-Nailing from IBC Row 17.* *-10d box equivalent to power driven nail size added.*
4	Ceiling joist attached to parallel rafter (heel joint) (see Section 2308.10.4.1, Table 2308.10.4.1)	Per table 2308.10.4.1	Face nail	*-Nailing from IBC Row 18.*
5	Collar tie to rafter	3-10d common (3" x 0.148"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, 7/16" crown	Face nail	*-Nailing from IBC Row 26.* *-10d box equivalent to power driven nail size added.*

	DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION	*Notes:*
6	Rafter or roof truss to top plate (See Section 2308.10.1, Table 2308.10.1)	3-10 common (3" x 0.148"); or 3-16d box (3.5" x 0.135"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131 nails; or 4-3" 14 gage staples, 7/16" crown	Toenail[c]	*-Nailing from IRC Row 5.* *-10d box equivalent to power driven nail size added.*
7	Roof rafters to ridge valley or hip rafters; or, roof rafter to 2-inch ridge beam	2-16d common (3.5" x 0.162"); or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, 7/16" crown; or	End nail	*-Nailing from IBC Rows 27 and 28.* *-10d box equivalent to power driven nail size added.*
		3-10d common (3.5" x 0.148"); or 3-16d box (3.5" x 0.135"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, 7/16" crown	Toenail	*-Nailing from IBC Rows 27 and 28.* *-10d box equivalent to power driven nail size added.* *-16d box per IRC for toenailing of rafter in Row 6 added.*
		WALL		
8	Stud to stud (not at braced wall panels)	16d common (3.5" x 0.162");	24" o.c. face nail	*-Nailing from IBC Row 9.*
		10d box (3" x 0.128"); or 3" x 0.131" nails; or 3-3" 14 gage staples, 7/16" crown	16" o.c. face nail	*-10d box equivalent to power driven nail size added.* *-Corrected spacing for power driven nail to be equivalent to the specified common nail.*
9	Stud to stud and abutting studs at intersecting wall corners (at braced wall panels)	16d common (3.5" x 0.162"); or	16" o.c. face nail	*-Nailing from IBC Row 23.* *-16d box equivalent from IRC Row 8.*
		16d box (3.5" x 0.135"); or 3" x 0.131" nails; or 3-3" 14 gage staples, 7/16" crown	12" o.c. face nail	
10	Built-up header (2-inch to 2-inch header)	16d common (3.5" x 0.162"); or	16" o.c. each edge, face nail	*-Nailing from IBC Row 14.* *-16d box equivalent added but at 12" o.c. spacing.*
		16d box (3.5" x 0.135")	12" o.c. each edge, face nail	
11	Continuous header to stud	4-8d common (2.5" x 0.131"); or 4-10d box (3" x 0.128")	Toenail	*-Nailing from IBC Row 16.* *-10d box equivalent to 8d common added.*
12	Top plate to top plate	16d common (3.5" x 0.162"); or	16" o.c. face nail	*-Nailing from IBC Row 10 except that 16d common specified in lieu of 16d box to align with power driven sizes.* *-10d box equivalent to power driven sizes added.*
		10d box (3" x 0.128"); or 3" x 0.131" nails; or 3" 14 gage staples, 7/16" crown	12" o.c. face nail	
13	Top plate to top plate, at end joints	8-16d common (3.5" x 0.162"); or 12-10d box (3" x 0.128"); or 12-3" x 0.131" nails; or 12-3" 14 gage staples, 7/16" crown	Face nail on each side of end joint (minimum 24" lap splice length each side of end joint)	*-Nailing from IBC Row 10.* *-10d box equivalent to power driven sizes added.*

	DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION	*Notes:*
14	Bottom plate to joist, rim joist, band joist or blocking (not at braced wall panels)	16d common (3.5" x 0.162"); or	16" o.c. face nail	*-Nailing from IBC Row 6 except that 16d common used in lieu of 16d box. -16d box equivalent added at 12" o.c.*
		16d box (3.5" x 0.135"); or 3" x 0.131" nails; or 3" 14 gage staples, 7/16" crown	12" o.c. face nail	
15	Bottom plate to joist, rim joist, band joist or blocking at braced wall panels	2-16d common (3.5" x 0.162"); or 3-16d box (3.5" x 0.135"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, 7/16" crown	16" o.c. face nail	*-Nailing from IBC Row 6; 16d common equivalent added*
16	Stud to bottom plate	4-8d common (2.5" 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	Toenail	*-Nailing per IBC Row 8. -10d box equivalent to 8d common added.*
		2-16d common (3.5" x 0.162"); or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, 7/16" crown	End nail	*-Nailing per IBC Row 8. -10d box equivalent to power driven sizes added.*
17	Top or bottom plate to stud	2-16d common (3.5" x 0.162"); or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, 7/16" crown	End nail	*-Nailing per IBC Row 7. -10d box equivalent to power driven sizes added.*
18	Top plates, laps at corners and intersections	2-16d common (3.5" x 0.162"); or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, 7/16" crown	Face nail	*-Nailing per IBC Row 13. -10d box equivalent to power driven sizes added.*
19	1" brace to each stud and plate	2-8d common (2.5" 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	Face nail	*-Nailing per IBC Row 20. -10d box equivalent to 8d common added.*
20	1" x 6" sheathing to each bearing	2-8d common (2.5" x 0.131"); or 2-10d box (3" x 0.128")	Face nail	*-Nailing per IRC Row 21 . -10d box equivalent to 8d common added.*
21	1" x 8" and wider sheathing to each bearing	3-8d common (2.5" x 0.131"); or 3-10d box (3" x 0.128")	Face nail	*-Nailing per IRC Rows 22 and 23, and IBC Rows 4, 21 and 22. -10d box equivalent to 8d common added.*
		FLOOR		
22	Joist to sill, top plate, or girder	3-8d common (2.5" x 0.131"); or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, 7/16" crown	Toenail	*-Nailing from IBC Row 1. -10d box equivalent to 8d common added.*
23	Rim joist, band joist, or blocking to sill or top plate	8d common (2.5" x 0.131"); or 10d box (3" x 0.128"); or 3" x 0.131" nails; or 3" 14 gage staples, 7/16" crown	6" o.c., toenail	*-Nailing from IBC Row 12. -10d box equivalent to 8d common added.*

	DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION		*Notes:*
24	1" x 6" subfloor or less to each joist	2-8d common (2.5" x 0.131"); or 3-10d box (3" x 0.128")	Face nail		*-Nailing from IBC Row 3. -10d box equivalent to 8d common added*
25	2" subfloor to joist or girder	2-16d common (3.5" x 0.162")	Face nail		*-Nailing from IBC Row 5.*
26	2" planks (plank & beam – floor & roof)	2-16d common (3.5" x 0.162")	At each bearing, face nail		*-Nailing from IBC Row 25.*
27	Built-up girders and beams, 2-inch lumber layers	20d common (4" x 0.192")	32" o.c., face nail at top and bottom staggered on opposite sides		*-Nailing from IBC Row 24. -10d box equivalent to power driven nail size added.*
		10d box (3" x 0.128"); or 3" x 0.131" nails; or 3" 14 gage staples, 7/16" crown	24" o.c. face nail at top and bottom staggered on opposite sides		
		And: 2-20d common (4" x 0.192"); or 3-10d box (3" x 0.128"); or 3-3" x 0.131" nails; or 3-3" 14 gage staples, 7/16" crown	Face nail at ends and at each splice		*-Nailing from IBC Row 24. -10d box equivalent to power driven nail sizes added.*
28	Ledger strip supporting joists or rafters	3-16d common (3.5" x 0.162"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, 7/16" crown	At each joist or rafter, face nail		*-Nailing from IBC Row 30. -10d box equivalent to power driven nail size added.*
29	Joist to band joist or rim joist	3-16d common (3.5" x 0.162"); or 4-10d box (3" x 0.128"); or 4-3" x 0.131" nails; or 4-3" 14 gage staples, 7/16" crown	End nail		*-Nailing from IBC Row 29. -10d box equivalent to power driven nail size added.*
30	Bridging to joist	2-8d common (2.5" 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	Each end, toenail		*-Nailing from IBC Row 2. -10d box equivalent to 8d common nail added.*
	Wood structural panels (WSP), subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framing[a]				
			Edges (inches)	Intermediate supports (inches)	
31	3/8" – 1/2"	6d common or deformed (2" x 0.113") (subfloor and wall)	6	12	*-Nailing from IBC Row 31.*
		8d box or deformed (2.5" x 0.113") (roof)	6	12	*-Nailing from IBC Row 31 footnote "L".*
		2 3/8" x 0.113" nail (subfloor and wall)	6	12	*-Nailing from IBC Row 31.*
		1 ¾" 16 gage staple, 7/16" crown (subfloor and wall)	4	8	*-Nailing from IBC Row 31 and footnote "o".*
		2 3/8 x 0.113" nail (roof)	4	8	*-Nailing from IBC Row 31 and footnote "n".*
		1 ¾" 16 gage staple, 7/16" crown (roof)	3	6	*-Nailing from IBC Row 31 and footnote "o".*
32	19/32" – 3/4"	8d common (2.5" x 0.131"); or 6d deformed (2" x 0.113)	6	12	*-Nailing from IBC Row 31.*
		2 3/8" x 0.113" nail; or 2" 16 gage staple, 7/16" crown	4	8	*-Nailing from IBC Row 31 and footnote "p".*
33	7/8" – 1 1/4"	10d common (3" x 0.148"); or 8d deformed (2.5" x 0.131")	6	12	*-Nailing from IBC Row 31 and footnote "e".*
	Other exterior wall sheathing				
34	1/2" fiberboard sheathing[b]	1 ½" galvanized roofing nail (7/16" head diameter; or 6d common (2" x 0.113"); or	3	6	*-Nailing from IBC Row 33 and footnote "g" and "h" and "i".*

	DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION		*Notes:*
		1 ¼" 16 gage staple with 7/16" or 1" crown			
35	25/32" fiberboard sheathing[b]	1 ¾" galvanized roofing nail (7/16" diameter head); or 8d common (2.5" x 0.131"); or 1 ½" 16 gage staple with 7/16" or 1" crown	3	6	*-Nailing from IBC Row 33 and footnote "g" and "h" and "i".*
Wood structural panels, combination subfloor underlayment to framing					
36	3/4" and less	8d common (2.5" x 0.131"); or6d deformed (2" x 0.113")	6	12	*-Nailing from IBC Row 31 and footnote "e" and IRC Row 39 for common nail size.*
37	7/8" – 1"	8d common (2.5" x 0.131"); or 8d deformed (2 ½" x 0.131")	6	12	*-Nailing from IBC Row 31 and footnote "e" and IRC Row 40 for common nail size.*
38	1 1/8" – 1 ¼"	10d common (3" x 0.148"); or 8d deformed (2 ½" x 0.131")	6	12	*-Nailing from IBC Row 31 for common and deformed nail size.*
Panel Siding to Framing					
39	½" or less	6d corrosion-resistant siding (1 7/8" × 0.106"); or 6d corrosion-resistant casing (2" × 0.099")	6	12	*-Nailing from IBC Row 32 and footnote "f".*
40	5/8"	8d corrosion-resistant siding (2 3/8" × 0.128"); or 8d corrosion-resistant casing (2 1/2" × 0.113")	6	12	*-Nailing from IBC Row 32 and footnote "f".*
Interior Paneling					
41	¼"	4d casing (11/2" × 0.080"); or 4d finish (11/2" × 0.072")	6	12	*-Nailing from IBC Row 34 and footnote "j".*
42	3/8"	6d casing (2" × 0.099"); or 6d finish (Panel supports at 24 inches)	6	12	*-Nailing from IBC Row 34 and footnote "k".*

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.

<u>Current (Existing)</u> Table 2304.9.1 with additional column indicating new location:

CONNECTION	FASTENING[a, m]	LOCATION	*Notes:*
1. Joist to sill or girder	3-8d common (2 ½ " x 0.131") 3-3" x 0.131" nails 3-3" 14 gage staples	toenail	*to new row 22*
2. Bridging to joist	2-8d common (2 ½" x 0.131") 2-3" x 0.131" nails 2-3" 14 gage staples	toenail each end	*to new row 30*
3. 1" x 6" subfloor or less to each joist	2-8d common (2 ½" x 0.131")	face nail	*to new row 24*
4. Wider than 1" x 6" subfloor to each joist	3-8d common (2 ½" x 0.131")	face nail	*deleted from table, wider condition addressed by row 21*
5. 2" subfloor to joist or girder	2-16d common (3 ½" x 0.162")	Blind and face nail	*to new row 25*
6. sole plate to joist or blocking Sole plate to joist or blocking at braced wall panel	16d (3 ½" x 0.135") at 16" o.c. 3" x 0.131" nails at 8 o.c. 3" 14 gage staples at 12" o.c. 3-16d (3 ½" x 0.135") at 16" o.c. 4-3" x 0.131" nails at 16" o.c. 4-3" 14 gage staples at 16" o.c.	typical face nail braced wall panels	*to new row 14* *to new row 15*

CONNECTION	FASTENING[a, m]	LOCATION	*Notes:*
7. Top plate to stud	2-16d common (3 ½" x 0.162") 3-3" x 0.131" nails 3-3" 14 gage staples	end nail	*to new row 17*
8. Stud to sole plate	4-8d common (2 ½" x 0.131") 4-3" x 0.131" nails 3-3" 14 gage staples	toenail	*to new row 16 and 17*
	2-16d common (3 ½" x 0.162") 3-3" x 0.131" nails 3-3" 14 gage staples	end nail	*to new row 16 and 17*
9. Double studs	16d (3 ½" x 0.135") at 24" o.c. 3" x 0.131" nail at 8" o.c. 3" 14 gage staple at 8" o.c.	face nail	*to new rows 8 and 9*
10. Double top plates Double top plates	16d (3 ½" x 0.135") at 16" o.c. 3" x 0.131" nail at 12" o.c. 3" 14 gage staple at 8" o.c. 8-16d common (3 ½" x 0.162") 12-3" x 0.131" nails 12-3" 14 gage staples	typical face nail lap splice	*to new rows 12*
11. Blocking between joists or rafters to top plate	3-8d common (2 ½" x 0.131") 3-3" x 0.131" nails 3-3" 14 gage staples	toenail	*to new row 1*
12. Rim joist to top plate	8d (2 ½" x 0.131") at 6" o.c. 3" x 0.131" nail at 6" o.c. 3" 14 gage staple at 6" o.c.	toenail	*to new row 23*
13. Top plates, laps and intersections	2-16d common (3 ½" x 0.162") 3-3" x 0.131" nails 3-3" 14 gage staples	face nail	*to new row 18*
14. Continuous header, two pieces	16d common (3 ½" 0.162")	16" o.c. along edge	*to new row 10*
15. Ceiling joists to plate	3-8d common (2 ½" x 0.131") 5-3" x 0.131" nails 5-3" 14 gage staples	toenail	*to new row 2*
16. Continuous header to stud	4-8d common (2 ½" x 0.131")	toenail	*to new row 11*
17. Ceiling joists, laps over partitions (see Section 2308.10.4.1, Table 2308.10.4.1)	3-16d common (3 ½" x 0.162") minimum,Table 2308.10.4.1 4-3" x 0.131" nails 4-3" 14 gage staples	face nail	*to new rows 3 and 4*
18. Ceiling joists to parallel rafters (see Section 2308.10.4.1, Table 2308.10.4.1)	3-16d common (3 ½" x 0.162") minimum,Table 2308.10.4.1 4-3" x 0.131" nails 4-3" 14 gage staples	face nail	*to new row 4*
19. Rafter to plate (see Section 2308.10-.1, Table 2308.10.1)	3-8d common (2 ½" x 0.131") 3-3" x 0.131" nails 3-3" 14 gage staples	Face nail	*to new row 6*
20. 1" diagonal brace to each stud and plate	2-8d common (2 ½" x 0.131") 2-3" x 0.131" nails 3-3" 14 gage staples	Face nail	*to new row 19*
21. 1" x 8" sheathing to each bearing	3-8d common (2 ½" x 0.131")	face nail	*to new row 21*
22. Wider than 1" x 8" sheathing to each bearing	3-8d common (2 ½" x 0.131")	face nail	*to new row 21*
23. Built-up corner studs	16d common (2 ½" x 0.131") 3" x 0.131" nails 3" 14 gage staples	24" o.c. 16" o.c. 16" o.c.	*to new row 9*
24. Built-up girder and beams	20d common (4" x 0.192") 32" o.c. 3" x 0.131" nails @ 24" o.c. 3" 14 gage staples @ 24" o.c.	face nail at top and bottom staggered on opposite sides	*to new row 27*
	2-20d common (4" x 0.192") 3-3" x 0.131" nails @ 24" o.c. 3-3" 14 gage staples @ 24" o.c.	face nail at ends and at each splice	*to new row 27*
25. 2" planks	16d common (3 ½" x 0.162")	at each bearing	*to new row 26*
26. Collar tie to rafter	3-10d common (3" x 0.148") 4-3" x 0.131" nails	face nail	*to new row 5*

CONNECTION	FASTENING[a, m]		LOCATION	*Notes:*
	4-3" 14 gage staples			
27. Jack rafter to hip	3-10d common (3" x 0.148") 4-3" x 0.131" nails 4-3" 14 gage staples		toenail	*to new row 7*
	2-16d common (3 ½" x 0.162") 3-3" x 0.131" nails 3-3" 14 gage staples		face nail	*to new row 7*
28. Roof rafter to 2-by ridge beam	2-16d common (3 ½" x 0.162") 3-3" x 0.131" nails 3-3" 14 gage staples		toenail	*to new row 7 except 10d common is specified for toe-nail case to match jack to hip nailing.*
	2-16d common (3 ½" x 0.162") 3-3" x 0.131" nails 3-3" 14 gage staples		face nail	*to new row 7*
29. Joist to band joist	3-16d common (3 ½" x 0.162") 4-3" x 0.131" nails 4-3" 14 gage staples		face nail	*to new row 29*
30. Ledger strip	3-16d common (3 ½" x 0.162") 4-3" x 0.131" nails 4-3" 14 gage staples		face nail at each joist	*to new row 28*
31. Wood structural panels and particleboard[b] Subfloor, roof and wall sheathing (to framing)	½" and less	6d[c,l] 2 3/8" x 0.113" nail[n] 1 ¾" 16 gage[o]		*to new row 31*
	19/32" to ¾"	8d[d] or 6d[e] 2 3/8" x 0.113" nail[p] 2" 16 gage[p]		*to new rows 32-33*
	7/8" to 1"	8d[c]		
	1 1/8" to 1 ¼"	10d[d] or 8d[e]		*to new rows 36, 37, 38*
Single floor (combination subfloor-underlayment to framing)	¾" and less	6d[e]		
	7/8" to 1"	8d[e]		
	1 1/8" to 1 ¼"	10d[d] or 8d[e]		
32. Panel siding (to framing)	½" or less	6d[f]		*to new rows 39 and 40*
	5/8"	8d[f]		
33. Fiberboard sheathing[g]	½"	No. 11 gage roofing nail[h] 6d common nail (2" x 0.113") No. 16 gage staple[i]		*to new row 34*
	25/32"	No. 11 gage roofing nail[h] 8d common nail (2" x 0.113") No. 16 gage staple[i]		*to new row 35*
34. Interior paneling	¼"	4d[j]		*to new row 41*
	3/8"	6d[k]		*to new row 42*

For SI: 1 inch = 25.4 mm.

a. common or box nails are permitted to be used except where otherwise stated.

b. Nails spaced at 6 inches on center at edges, 12 inches at intermediate supports except 6 inches at 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.

c. Common or deformed shank (6d-2" x 0.113"; 8d-2 ½" x 0.131"; 10d-3" x 0.148").

d. Common (6d-2" x 0.113"; 8d-2 ½" x 0.131"; 10d-3" x 0.148").

e. Deformed shank (6d-2" x 0.113"; 8d-2 ½" x 0.131"; 10d-3" x 0.148").

f. Corrosion-resistant siding (6d-1 7/8 x 0.106"; 8d-2 3/8" x 0.128") or casing (6d-2" x 0.099"; 8d-2 ½" x 0.113") nail.

g. Fasteners spaced 3 inches on center at exterior edges and 6 inches on center at intermediate supports, when used as structural sheathing. Spacing shall be 6 inches on center on the edges and 12 inches on center at intermediate supports for nonstructural applications.

h. Corrosion-resistant roofing nails with 7/16-inch-diameter head and d1 1 1/2"-inch length for ½-inch sheathing and 1 ¾-inch length for 25/32-inch sheathing.

i. Corrosion-resistant staples with nominal 7/16-inch crown or 1-inch crown and 1 ¼-inch length for ½-inch sheathing and 1 ½-inch length for 25/32-inch sheathing. Panel supports at 16 inches (20 inches if strength axis in the long direction of the panel, unless otherwise marked).

j. Casing (1 ½" x 0.080") or finish (1 ½" x 0.072") nails spaced 6 inches on panel edges, 12 inches at intermediate supports
k. Panel supports at 24 inches. Casing or finish nails spaced 6 inches on panel edges, 12 inches at intermediate supports.
l. For roof sheathing applications, 8d nails (2 ½" x 0.113") are the minimum required for wood structural panels.
m. Staples shall have a minimum crown width of 7:16 inch.
n. For roof sheathing applications, fasteners spaced 4 inches on center at edges, 8 inches at intermediate supports.
o. Fasteners spaced 4 inches on center at edges, 8 inches at intermediate supports for subfloor and wall sheathing and 3 inches on center at edges, 6 inches at intermediate supports for roof sheathing.
p. Fasteners spaced 4 inches on center at edges, 8 inches at intermediate supports.

(The 2012 IRC fastener schedule is shown below for reference)

TABLE R602.3(1)
FASTENER SCHEDULE FOR STRUCTURAL MEMBERS

ITEM	DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER[a, b, c]	SPACING OF FASTENERS
		Roof	
1	Blocking between joists or rafters to top plate, toe nail	3-8d ($2^1/_2$″ × 0.113″)	—
2	Ceiling joists to plate, toe nail	3-8d ($2^1/_2$″ × 0.113″)	—
3	Ceiling joists not attached to parallel rafter, laps over partitions, face nail	3-10d	—
4	Collar tie to rafter, face nail or $1^1/_4$″ × 20 gage ridge strap	3-10d (3″ × 0.128″)	—
5	Rafter or roof truss to plate, toe nail	3-16d box nails ($3^1/_2$″ × 0.135″) or 3-10d common nails (3″ × 0.148″)	2 toe nails on one side and 1 toe nail on opposite side of each rafter or truss[j]
6	Roof rafters to ridge, valley or hip rafters: toe nail face nail	4-16d ($3^1/_2$″ × 0.135″) 3-16d ($3^1/_2$″ × 0.135″)	—
		Wall	
7	Built-up studs-face nail	10d (3″ × 0.128″)	24″ o.c.
8	Abutting studs at intersecting wall corners, face nail	16d (3 ½" x 0.135")	12″ o.c.
9	Built-up header, two pieces with $^1/_2$″ spacer	16d ($3^1/_2$″ × 0.135″)	16″ o.c. along each edge
10	Continued header, two pieces	16d ($3^1/_2$″ × 0.135″)	16″ o.c. along each edge
11	Continuous header to stud, toe nail	4-8d ($2^1/_2$″ × 0.113″)	—
12	Double studs, face nail	10d (3″ × 0.128″)	24″ o.c.
13	Double top plates, face nail	10d (3″ × 0.128″)	24″ o.c.
14	Double top plates, minimum 24-inch offset of end joints, face nail in lapped area	8-16d ($3^1/_2$″ × 0.135″)	—
15	Sole plate to joist or blocking, face nail	16d ($3^1/_2$″ × 0.135″)	16″ o.c.
16	Sole plate to joist or blocking at braced wall panels	3-16d ($3^1/_2$″ × 0.135″)	16″ o.c.
17	Stud to sole plate, toe nail	3-8d ($2^1/_2$″ × 0.113″) or 2-16d ($3^1/_2$″ × 0.135″)	— —
18	Top or sole plate to stud, end nail	2-16d ($3^1/_2$″ × 0.135″)	—
19	Top plates, laps at corners and intersections, face nail	2-10d (3″ × 0.128″)	—
20	1″ brace to each stud and plate, face nail	2-8d ($2^1/_2$″ × 0.113″) 2 staples $1^3/_4$″	— —
21	1″ × 6″ sheathing to each bearing, face nail	2-8d ($2^1/_2$″ × 0.113″) 2 staples $1^3/_4$″	— —
22	1″ × 8″ sheathing to each bearing, face nail	2-8d ($2^1/_2$″ × 0.113″) 3 staples $1^3/_4$″	— —
23	Wider than 1″ × 8″ sheathing to each bearing, face nail	3-8d ($2^1/_2$″ × 0.113″) 4 staples $1^3/_4$″	— —
		Floor	
24	Joist to sill or girder, toe nail	3-8d ($2^1/_2$″ × 0.113″)	—
25	Rim joist to top plate, toe nail (roof applications also)	8d ($2^1/_2$″ × 0.113″)	6″ o.c.
26	Rim joist or blocking to sill plate, toe nail	8d (2 ½″ × 0.113″)	6″ o.c.
27	1″ × 6″ subfloor or less to each joist, face nail	2-8d ($2^1/_2$″ × 0.113″) 2 staples $1^3/_4$″	— —
28	2″ subfloor to joist or girder, blind and face nail	2-16d ($3^1/_2$″ × 0.135″)	—
29	2″ planks (plank & beam - floor & roof)	2-16d ($3^1/_2$″ × 0.135″)	at each bearing
30	Built-up girders and beams, 2-inch lumber layers	10d (3″ × 0.128″)	Nail each layer as follows: 32″ o.c. at top and bottom and staggered. Two nails at ends and at each splice.
31	Ledger strip supporting joists or rafters	3-16d ($3^1/_2$″ × 0.135″)	At each joist or rafter

(continued)

TABLE R602.3(1)—continued
FASTENER SCHEDULE FOR STRUCTURAL MEMBERS

ITEM	DESCRIPTION OF BUILDING MATERIALS	DESCRIPTION OF FASTENER[b, c, e]	SPACING OF FASTENERS	
			Edges (inches)[i]	Intermediate supports[c, e] (inches)
Wood structural panels, subfloor, roof and interior wall sheathing to framing and particleboard wall sheathing to framing				
32	$^3/_8$″ - $^1/_2$″	6d common (2″ × 0.113″) nail (subfloor wall)[j] 8d common ($2^1/_2$″ × 0.131″) nail (roof)[f]	6	12[g]
33	$^{19}/_{32}$″ - 1″	8d common nail ($2^1/_2$″ × 0.131″)	6	12[g]
34	$1^1/_8$″ - $1^1/_4$″	10d common (3″ × 0.148″) nail or 8d ($2^1/_2$″ × 0.131″) deformed nail	6	12
Other wall sheathing[h]				
35	$^1/_2$″ structural cellulosic fiberboard sheathing	$^1/_2$″ galvanized roofing nail, $^7/_{16}$″ crown or 1″ crown staple 16 ga., $1^1/_4$″ long	3	6
36	$^{25}/_{32}$″ structural cellulosic fiberboard sheathing	$1^3/_4$″ galvanized roofing nail, $^7/_{16}$″ crown or 1″ crown staple 16 ga., $1^1/_2$″ long	3	6
37	$^1/_2$″ gypsum sheathing[d]	$1^1/_2$″ galvanized roofing nail; staple galvanized, $1^1/_2$″ long; $1^1/_4$ screws, Type W or S	7	7
38	$^5/_8$″ gypsum sheathing[d]	$1^3/_4$″ galvanized roofing nail; staple galvanized, $1^5/_8$″ long; $1^5/_8$″ screws, Type W or S	7	7
	Wood structural panels, combination subfloor underlayment to framing			
39	$^3/_4$″ and less	6d deformed (2″ × 0.120″) nail or 8d common ($2^1/_2$″ × 0.131″) nail	6	12
40	$^7/_8$″ - 1″	8d common ($2^1/_2$″ × 0.131″) nail or 8d deformed ($2^1/_2$″ × 0.120″) nail	6	12
41	$1^1/_8$″ - $1^1/_4$″	10d common (3″ × 0.148″) nail or 8d deformed ($2^1/_2$″ × 0.120″) nail	6	12

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 mile per hour = 0.447 m/s; 1 Ksi = 6.895 MPa.

a. All nails are smooth-common, box or deformed shanks except where otherwise stated. Nails used for framing and sheathing connections shall have minimum average bending yield strengths as shown: 80 ksi for shank diameter of 0.192 inch (20d common nail), 90 ksi for shank diameters larger than 0.142 inch but not larger than 0.177 inch, and 100 ksi for shank diameters of 0.142 inch or less.

b. Staples are 16 gage wire and have a minimum $^7/_{16}$-inch on diameter crown width.

c. Nails shall be spaced at not more than 6 inches on center at all supports where spans are 48 inches or greater.

d. Four-foot by 8-foot or 4-foot by 9-foot panels shall be applied vertically.

e. Spacing of fasteners not included in this table shall be based on Table R602.3(2).

f. For regions having basic wind speed of 110 mph or greater, 8d deformed ($2^1/_2$″ × 0.120) nails shall be used for attaching plywood and wood structural panel roof sheathing to framing within minimum 48-inch distance from gable end walls, if mean roof height is more than 25 feet, up to 35 feet maximum.

g. For regions having basic wind speed of 100 mph or less, nails for attaching wood structural panel roof sheathing to gable end wall framing shall be spaced 6 inches on center. When basic wind speed is greater than 100 mph, nails for attaching panel roof sheathing to intermediate supports shall be spaced 6 inches on center for minimum 48-inch distance from ridges, eaves and gable end walls; and 4 inches on center to gable end wall framing.

h. Gypsum sheathing shall conform to ASTM C 1396 and shall be installed in accordance with GA 253. Fiberboard sheathing shall conform to ASTM C 208.

i. Spacing of fasteners on floor sheathing panel edges applies to panel edges supported by framing members and required blocking and at all floor perimeters only. Spacing of fasteners on roof sheathing panel edges applies to panel edges supported by framing members and required blocking. Blocking of roof or floor sheathing panel edges perpendicular to the framing members need not be provided except as required by other provisions of this code. Floor perimeter shall be supported by framing members or solid blocking.

j. Where a rafter is fastened to an adjacent parallel ceiling joist in accordance with this schedule, provide two toe nails on one side of the rafter and toe nails from the ceiling joist to top plate in accordance with this schedule. The toe nail on the opposite side of the rafter shall not be required.

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

T2304.9.1-S-BAJNAI-BCAC.doc

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This reformatting and reorganizing of the fastener schedule makes it easier to use and is an excellent idea. Note that the changes approved in S261-12 and S263-12 will be incorporated in items 1, 6, 14 and 23.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Paul Coats, P.E., CBO, American Wood Council, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

Table 2304.9.1
FASTENING SCHEDULE

ITEM	DESCRIPTION OF BUILDING ELEMENTS	NUMBER AND TYPE OF FASTENER	SPACING AND LOCATION
24	1" x 6" subfloor or less to each joist	2-8d common (2.5" x 0.131"); or 2 ~~3~~-10d box (3" x 0.128")	Face nail

(Portions of table not shown remain unchanged)

Commenter's Reason: As part of the overall table revision, the 10d box nail (3" x 0.128") was added as an equivalent to the 8d common nail (2.5" x 0.131"). The correct number of 10d box nails is 2 which matches the required number of 8d common nails.

S265-12
Final Action: AS AM AMPC____ D

S268-12

2304.11, 2304.11.1, 2304.11.2, 2304.11.2.1, 2304.11.2.2, 2304.11.2.3, 2304.2.4, 2304.11.2.5, 2304.11.2.6, 2304.11.2.7, 2304.11.3, 2304.11.4, 2304.11.4.1, 2304.11.4.2, 2304.11.5, 2304.11.6, 2304.11.7

Proposed Change as Submitted

Proponent: Dennis Pitts, American Wood Council, (dpitts@awc.org)

Revise as follows:

2304.11 Protection against decay and termites. Wood shall be protected from decay and termites in accordance with the applicable provisions of Sections 2304.11.1 through ~~2304.11.9~~ 2304.11.7.

~~**2304.11.1 General.** Where required by this section, protection from decay and termites shall be provided by the use of naturally durable or *preservative-treated wood*.~~

~~**2304.11.2 Wood used above ground**~~ **2304.11.1 Location requiring water-borne preservatives.** Wood used above ground in the locations specified in Sections ~~2304.11.2.1~~ 2304.11.1.1 through ~~2304.11.2.7~~ 2304.11.1.5, 2304.11.3 and 2304.11.5 shall be naturally durable wood or *preservative-treated wood* using water-borne preservatives, in accordance with AWPA U1 ~~(Commodity Specifications A or F)~~ for above-ground use.

~~**2304.11.2.1**~~ **2304.11.1.1 Joists, girders and subfloor.** ~~Where~~ Wood joists or ~~the bottom of~~ a wood structural floor ~~without joists are~~ closer than 18 inches (457 mm), or wood girders ~~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~~, the floor construction (including posts, girders, joists and subfloor)~~ shall be of naturally durable or *preservative-treated wood*.

~~**2304.11.2.2**~~ **2304.11.1.2 Wood supported by exterior foundation walls.** Wood framing members, including wood sheathing, that ~~rest on~~ 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*.

~~**2304.11.2.3**~~ **2304.11.1.3 Exterior walls below grade.** Wood framing members and furring strips ~~attached directly to~~ in direct contact with the interior of exterior masonry or concrete walls below grade shall be of naturally durable or *preservative-treated wood*.

~~**2304.11.2.4**~~ **2304.11.1.4 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*.

~~**2304.11.2.6**~~ **2304.11.1.5 Wood siding.** Clearance between wood siding and earth on the exterior of a building shall not be less than 6 inches (152 mm) or less than 2 inches (51 mm) vertical from concrete steps, porch slabs, patio slabs and similar horizontal surfaces exposed to the weather except where siding, sheathing and wall framing are of naturally durable or *preservative-treated wood*.

2304.11.2 Other locations. Wood used in the locations specified in Sections 2304.11.2.1 through 2304.11.2.5 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.

~~**2304.11.2.5**~~ **2304.11.2.1 Girder ends.** The ends of wood girders entering exterior masonry or concrete walls shall be provided with a 1/2-inch (12.7 mm) air space on top, sides and end, unless naturally durable or *preservative-treated wood* is used.

~~2304.11.2.7~~ 2304.11.2.2 **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*.

Exceptions:

~~1.~~ Posts or columns that are ~~either~~ not exposed to the weather ~~or located in basements or cellars,~~ are supported by concrete piers or metal pedestals projected at least 1 inch (25 mm) above the slab or deck and ~~6~~ 8 inches (152 mm) above exposed earth, and are separated ~~therefrom~~ by an impervious moisture barrier.

~~2. Posts or columns in enclosed crawl spaces or unexcavated areas located within the periphery of the building, supported by a concrete pier or metal pedestal at a height greater than 8 inches (203 mm) from exposed ground, and are separated therefrom by an impervious moisture barrier.~~

~~2304.11.5~~ 2304.11.2.3 **Supporting member for permanent appurtenances.** Naturally durable or *preservative-treated wood* shall be utilized for those portions of wood members that form the structural supports of buildings, balconies, porches or similar permanent building appurtenances where such members are exposed to the weather without adequate protection from a roof, eave, overhang or other covering to prevent moisture or water accumulation on the surface or at joints between members.

Exception: When a building is located in a geographical region where experience has demonstrated that climatic conditions preclude the need to use durable materials where the structure is exposed to the weather.

~~2304.11.3~~ 2304.11.2.4 **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*.

2304.11.2.5. 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.

~~2304.11.4~~ 2304.11.3 **Wood in contact with the ground or fresh water.** Wood used in contact with the ground (exposed earth) ~~in the locations specified in Sections 2304.11.4.1 and 2304.11.4.2~~ shall be naturally durable (species for both decay and termite resistance) or preservative treated ~~using water-borne preservatives~~ in accordance with AWPA U1 ~~(Commodity Specifications A or F)~~ for soil or fresh water use.

Exception: Untreated wood is permitted where such wood is continuously and entirely below the groundwater level or submerged in fresh water.

~~2304.11.4.1~~ 2304.11.3.1 **Posts or columns.** Posts and columns supporting permanent structures that are embedded in concrete that is ~~in direct contact with the earth, embedded in concrete that is~~ exposed to the weather or in direct contact with the earth shall be of *preservative-treated wood*.

~~**2304.11.4.2 Wood structural members.** 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.~~

~~2304.11.6~~ 2304.11.4 **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 2304.11.1.1 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.

~~2304.11.7~~ 2304.11.5 Wood used in retaining walls and cribs. Wood installed in retaining or crib walls shall be preservative treated in accordance with AWPA U1 ~~(Commodity Specifications A or F)~~ for soil and fresh water use.

Reason: This code change contains few technical changes but addresses many editorial clean-ups and some re-organization. The technical change is a delineation of exactly where waterborne preservatives should be required and where they should not. In a reorganization of this section in the 2005 code change cycle, glued laminated and certain exterior applications were lumped under a general section for the purposes of citing the new AWPA U1 standard, but a requirement for waterborne preservatives was inadvertently imposed for all applications in that reorganization. This proposed code change restores the ability for glued laminated beams and wood in exterior applications to be treated with other-than waterborne preservatives in accordance with the U1 standard. As a precaution, a requirement for the drying of treated wood and its sealing was added where used on the interior of a building (proposed section 2304.11.2).

Other changes are explained as follows:

Existing section 2304.11.1 deletion: This section became superfluous.

Proposed 2304.11.1: Section references are changed, and the specific mention of commodity specifications in the U1 standard was deleted because it is unnecessary.

Proposed 2304.11.1.1: Removing "the floor construction (including posts, girders, joists and subfloor)" makes it clear that only those floor elements within proximity to exposed ground need to be protected.

Proposed 2304.11.1.2: Better wording to meet current intent.

Proposed 2304.11.1.3: Better wording to meet current intent.

Proposed 2304.11.2: This new section is needed to introduce the subsections for locations where other-than waterborne preservatives are permitted under certain circumstances, as long as treatment is in accordance with the AWPA U1 standard.

Proposed 2304.11.2.2 Exceptions: The first exception was worded incorrectly and would seem to exempt exposed wood from protection; the proposed wording is a fix. With Exception 1 fixed, exception 2 was so similar in requirement that it was combined with Exception 1 and the clearance dimension was changed from 6 to 8 inches to preserve the intent of the deleted exception and be consistent with the clearance required for wood supported by exterior foundation walls in proposed Section 2304.11.1.2.

Proposed 2304.11.2.5: This is not a new section, but is re-titled and moved up in the text from Section 2304.11.4.2 (shown struck-out further down). There is no obvious reason why it must be a subsection of current 2304.11.4.

Proposed 2304.11.3: The requirement that water-borne preservatives be used exclusively has been struck in accordance with the purpose of this change, which indicates those locations where water-borne preservatives must be used up in proposed Section 2304.11.1 and subsections.

Existing section 2304.11.4.1 and 2304.11.4.2 (shown struck out): These were not lost. The current 2304.11.4.2 was moved up to become proposed 2304.11.2.5, and the current 2304.11.4.1 became 2304.11.3.1 with some editorial rewording for clarity.

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

2304.11-S-PITTS.doc

Public Hearing Results

Committee Action: Approved as Submitted

Committee Reason: This code makes improvements to the current language regarding preservative treated and naturally durable wood.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Randall Shackelford, Simpson Strong-Tie Co., requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

2304.11.2.2 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*.

> **Exceptions:** Posts or columns that are not exposed to the weather or that are protected from moisture by a roof, eave, or similar covering, are supported by concrete piers or metal pedestals ~~projected~~ projecting at least 1 inch (25 mm) above the slab or deck and 8 inches (~~152~~ 203 mm) above exposed earth, and are separated from concrete by an impervious moisture barrier.

(Portions of proposal not shown remain unchanged)

Commenter's Reason: The original proponent made this requirement much more restrictive by changing this wording. The most common application for the one-inch tall metal pedestal is for posts supporting a porch. The approved wording would require all porch posts to be treated. The existing IBC language, in both 2304.11.3 (Laminated timbers) and 2304.11.5 (Supporting member for permanent appurtenances) provides an exception for wood members that are protected from moisture by a roof, eave, or similar covering. This public comment takes that wording (word for word from 2304.11.3) and inserts it here, also to clarify when it is applicable. Without the clarification, it is open to interpretation what "exposed to the weather" means. It could easily be interpreted to any wood column or post that is outside. Changed "projected" to "projecting" so that it better describes the pier or pedestal, and then added "from concrete" to clarify that the impervious moisture barrier is only needed for the concrete pier, since the metal pedestal already provides the separation.

S268-12

Final Action: AS AM AMPC____ D

S273-12
2308 (New)

Proposed Change as Submitted

Proponent: Robert Rice, Josephine County, OR (structdesigner@yahoo.com)

Delete and substitute as follows:

~~**SECTION 2308**~~
~~**CONVENTIONAL LIGHT-FRAME CONSTRUCTION**~~

SECTION 2308
CONVENTIONAL LIGHT-FRAME CONSTRUCTION

2308.1 General. The requirements of this section are intended for *conventional light-frame construction*. Other construction methods are permitted to be used, provided a satisfactory design is submitted showing compliance with other provisions of this code. Interior non-load-bearing partitions, ceilings and curtain walls of *conventional light-frame construction* are not subject to the limitations of section 2308.2. Alternatively, compliance with AF&PA WFCM shall be permitted subject to the limitations therein and the limitations of this code. Detached one- and two-family dwellings and multiple single-family dwellings (townhouses) not more than three *stories above grade plane* in height with a separate *means of egress* and their accessory structures shall comply with the *International Residential Code*.

2308.2 Limitations. Buildings are permitted to be constructed in accordance with the provisions of *conventional light-frame construction*, subject to the following limitations;

2308.2.1 Stories. Structures of *conventional light-frame construction* shall be limited in story height according to Table 2308.2.1

TABLE 2308.2.1
ALLOWABLE STORY HEIGHT

Seismic Design Category	Allowable Story above grade plane
A and B	Three stories
C	Two Stories
D and E [a]	One story

a. For the purposes of this section, for buildings assigned to *Seismic Design Category* D or E, cripple walls shall be considered to be a *story* unless cripple walls are solid blocked and do not exceed 14 inches in height,

2308.2.2 Allowable floor-to-floor height. Maximum floor-to-floor height shall not exceed 11 feet, 7 inches (3531 mm). Exterior bearing wall and interior braced wall heights shall not exceed a stud height of 10 feet (3048 mm).

2308.2.3 Allowable Loads. Loads shall be in accordance with Chapter 16 and shall not exceed the following:

1. Average dead loads shall not exceed 15 psf (718 N/m^2) for combined roof and ceiling, exterior walls, floors and partitions.

Exceptions:

1. Subject to the limitations of Section 2308.6.10.2, stone or masonry veneer up to the lesser of 5 inches (127 mm) thick or 50 psf (2395 N/m^2) and installed in accordance with Chapter 14 is permitted to a height of 30 feet (9144 mm) above a noncombustible foundation, with an additional 8 feet (2438 mm) permitted for gable ends.

2. Concrete or masonry fireplaces, heaters and chimneys shall be permitted in accordance with the provisions of this code.

2. Live loads shall not exceed 40 psf (1916 N/m^2) for floors.
3. Ground snow loads shall not exceed 50 psf (2395 N/m^2).

2308.2.4 Allowable wind speed. V_{asd} as determined in accordance with Section 1609.3.1 shall not exceed 100 miles per hour (mph) (44 m/s) (3-second gust).

Exceptions:

1. V_{asd} as determined in accordance with Section 1609.3.1 shall not exceed 110 mph (48.4 m/s) (3-second gust) for buildings in Exposure Category B that are not located in a *hurricane-prone region*.
2. Where V_{asd} as determined in accordance with Section 1609.3.1 exceeds 100 mph (3-second gust), the provisions of either AF&PA WFCM or ICC 600 are permitted to be used. Wind speeds in Figures 1609A, 1609B, and 1609C shall be converted in accordance with Section 1609.3.1 for use with AF&PA WFCM or ICC 600.

2308.2.5 Allowable roof span. Roof trusses and rafters shall not span more than 40 feet (12 192 mm) between points of vertical support.

2308.2.6 Risk Category limitation. The use of the provisions for *conventional light-frame construction* in this section shall not be permitted for *Risk Category* IV buildings, as determined by Section 1604.5, assigned to *Seismic Design Category* B, C, D or E.

2308.2.7 Portions exceeding limitations of conventional light-frame construction. When portions of a building of otherwise conventional light-frame construction exceed the limits of Section 2308.2, those portions and the supporting load path shall be designed in accordance with accepted engineering practice and the provisions of this code. For the purposes of this section, the term “portions” shall mean parts of buildings containing volume and area such as a room or a series of rooms. The extent of such design need only demonstrate compliance of the non-conventionally light-framed elements with other applicable provisions of this code and shall be compatible with the performance of the conventional light-framed system.

2308.3 Foundations and footings. Foundations and footings shall be designed and constructed in accordance with Chapter 18 . Connections to foundations and footings shall comply with this section.

2308.3.1 Foundation plates or sills. Foundation plates or sills resting on concrete or masonry foundations shall comply with Section 2304.3.1. Foundation plates or sills shall be bolted or anchored to the foundation with not less than 1/2-inch-diameter (12.7 mm) steel bolts or *approved* anchors spaced to provide equivalent anchorage as the steel bolts. Along *braced wall lines* in structures assigned to *Seismic Design Category* E, steel bolts with a minimum nominal diameter of 5/8 inch (15.9 mm) or approved anchor straps load rated in accordance with Section 1706.1 and spaced to provide equivalent anchorage shall be used. Bolts shall be embedded at least 7 inches (178 mm) into concrete or masonry.

Bolts shall be spaced not more than 6 feet (1829 mm) apart and there shall be a minimum of two bolts or anchor straps per piece with one bolt or anchor strap located not more than 12 inches (305 mm) or less than 4 inches (102 mm) from each end of each piece. Bolts in *braced wall lines* in structures over two stories above grade shall be spaced not more the 4 feet (1219 mm) o.c.. A properly sized nut and washer shall be tightened on each bolt to the plate.

2308.3.2 Braced wall line sill plate anchorage in *Seismic Design Category* D and E. Sill plates along *braced wall lines* shall be anchored with anchor bolts with steel plate washers between the foundation sill plate and the nut, or *approved* anchor straps load rated in accordance with Section 1706.1. Such washers shall be a minimum of 0.229 inch by 3 inches by 3 inches (5.82 mm by 76 mm by 76 mm) in size. The hole in the plate washer is permitted to be diagonally slotted with a width of up to 3/16 inch (4.76 mm) larger than the bolt diameter and a slot length not to exceed 1-3/4 inches (44 mm), provided a standard cut washer is placed between the plate washer and the nut.

2308.4 Floor framing. Floor framing shall comply with this section.

2308.4.1 Girders. Girders for single-story construction or girders supporting loads from a single floor shall not be less than 4 inches by 6 inches (102 mm by 152 mm) for spans 6 feet (1829 mm) or less, provided that girders are spaced not more than 8 feet (2438 mm) o.c. Spans for built-up 2-inch girders shall be in accordance with Table 2308.4.1(1) or 2308.4.1(2). Other girders shall be designed to support the loads specified in this code. Girder end joints shall occur over supports.

Where a girder is spliced over a support, an adequate tie shall be provided. The ends of beams or girders supported on masonry or concrete shall not have less than 3 inches (76 mm) of bearing.

~~TABLE 2308.9.5~~ TABLE 2308.4.1(1)
HEADER AND GIRDER SPANS[a] FOR EXTERIOR BEARING WALLS
(Maximum Spans for Douglas Fir-Larch, Hem-Fir, Southern Pine and Spruce-Pine-Fir[b] and Required Number of Jack Studs)

(Portions of table not shown remain unchanged)

~~TABLE 2308.9.6~~ TABLE 2308.4.1(2)
HEADER AND GIRDER SPANS[a] FOR INTERIOR BEARING WALLS
(Maximum Spans for Douglas Fir-Larch, Hem-Fir, Southern Pine and Spruce-Pine-Fir[b] and Required Number of Jack Studs)

(Portions of table not shown remain unchanged)

2308.4.2 Floor joists. Floor joists shall comply with this section.

2308.4.2.1 Span. Spans for floor joists shall be in accordance with Tables 2308.4.2.1(1) or 2308.4.2.1(2) or the *AF&PA Span Tables for Joists and Rafters*.

2308.4.2.2 Bearing. The ends of each joist shall not have less than 1-1/2 inches (38 mm) of bearing on wood or metal, or not less than 3 inches (76 mm) on masonry, except where supported on a 1-inch by 4-inch (25.4 mm by 102 mm) ribbon strip and nailed to the adjoining stud,

2308.4.2.3 Framing details. Joists shall be supported laterally at the ends and at each support by solid blocking except where the ends of the joists are nailed to a header, band or rim joist or to an adjoining stud or by other means. Solid blocking shall not be less than 2 inches (51mm) in thickness and the full depth of the joist. Joist framing from opposite sides of a beam, girder or partition shall be lapped at least 3 inches (76 mm) or the opposing joists shall be tied together in an approved manner. Joists framing into the side of a wood girder shall be supported by framing anchors or on ledger strips not less than 2 inches by 2 inches (51 mm by 51 mm).

TABLE ~~2308.8(1)~~ 2308.4.2.1(1)
FLOOR JOIST SPANS FOR COMMON LUMBER SPECIES
(Residential Sleeping Areas, Live Load = 30 psf, L/Δ = 360)
(Portions of table not shown remain unchanged)

TABLE ~~2308.8(2)~~ 2308.4.2.1(2)
FLOOR JOIST SPANS FOR COMMON LUMBER SPECIES
(Residential Living Areas, Live Load = 40 psf, L/Δ = 360)

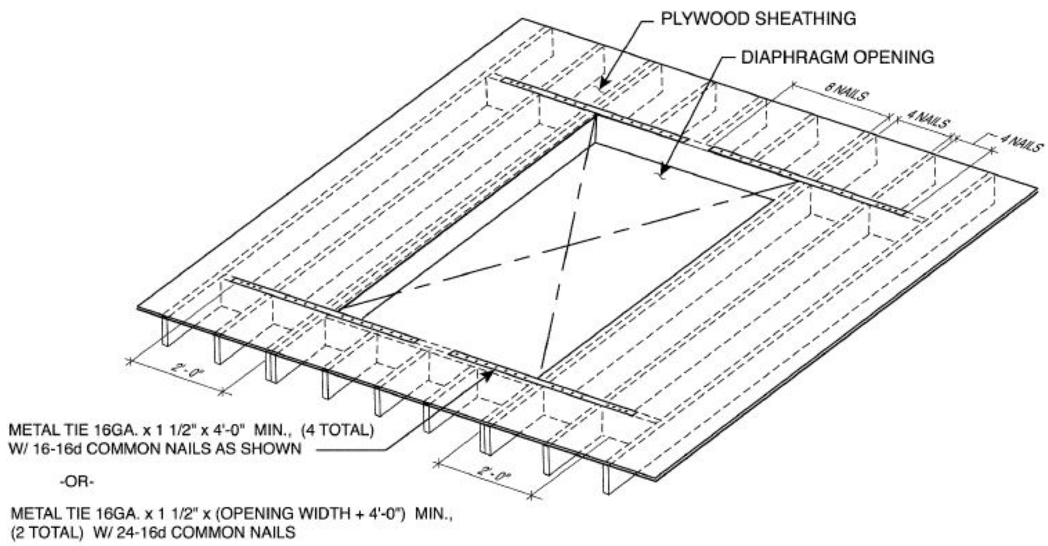
(Portions of table not shown remain unchanged)

2308.4.2.4 Notches and holes. Notches on the ends of joists shall not exceed one-fourth the joist depth. Notches in the top or bottom of joists shall not exceed one sixth the depth and shall not be located in the middle third of the span. Holes bored in joists shall not be within 2 inches (51 mm) of the top or bottom of the joist and the diameter of any such hole shall not exceed one-third the depth of the joist.

2308.4.3 Engineered wood products. Engineered wood products shall be installed in accordance with manufacturer's recommendations. Cuts, notches and holes bored in trusses, structural composite lumber, structural glue-laminated members or I-joists are not permitted except where permitted by the manufacturer's recommendations or where the effects of such alterations are specifically considered in the design of the member by a *registered design professional*.

2308.4.4 Framing around openings. Trimmer and header joists shall be doubled, or of lumber of equivalent cross section, where the span of the header exceeds 4 feet (1219 mm). The ends of header joists more than 6 feet (1829 mm) long shall be supported by framing anchors or joist hangers unless bearing on a beam, partition or wall. Tail joists over 12 feet (3658 mm) long shall be supported at the header by framing anchors or on ledger strips not less than 2 inches by 2 inches (51 mm by 51 mm).

2308.4.4.1 Openings in floor diaphragms in *Seismic Design Categories* B, C, D and E. Openings in horizontal diaphragms with a dimension perpendicular to the joist that is greater than 4 feet (1219 mm) shall be constructed with metal ties and blocking in accordance with this section and Figure 2308.4.4.1(1). Metal ties shall not be less than 0.058 inch [1.47 mm (16 galvanized gage)] thick by 1-1/2 inches (38 mm) wide with a minimum yield stress of 33,000 psi (227 Mpa). Blocking shall be provided 2 feet minimum beyond headers. Ties shall be attached to blocking with eight 16d common nails on each side of the header-joist intersection.



For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm.

FIGURE 2308.4.4.1(1)
OPENINGS IN FLOOR AND ROOF DIAPHRAGMS

Openings in floor diaphragms in *Seismic Design Categories D and E* shall not exceed a dimension greater than 50 percent of the distance between braced wall lines or an area greater than 25 percent of the area between orthogonal pairs of braced wall lines [see Figure 2308.4.4.1(2)], or shall be designed in accordance with accepted engineering practice.

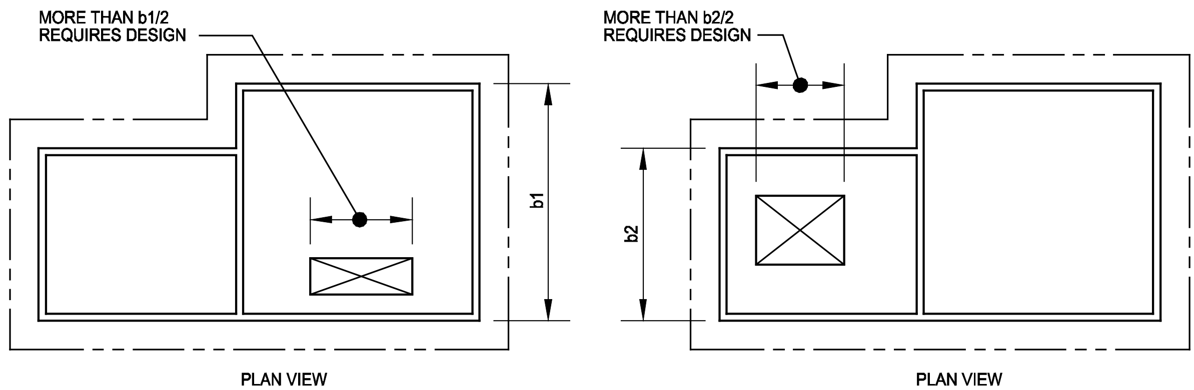


FIGURE 2308.4.4.1(2)
OPENING LIMITATIONS FOR FLOOR AND ROOF DIAPHRAGMS

2308.4.4.2 Vertical offsets in floor diaphragms in *Seismic Design Categories* D and E. Portions of a floor level shall not be vertically offset such that the framing members on either side of the offset cannot be lapped or tied together in an *approved* manner in accordance with Figure 2308.4.4.2.

Exception: Framing supported directly by foundations need not be lapped or tied directly together.

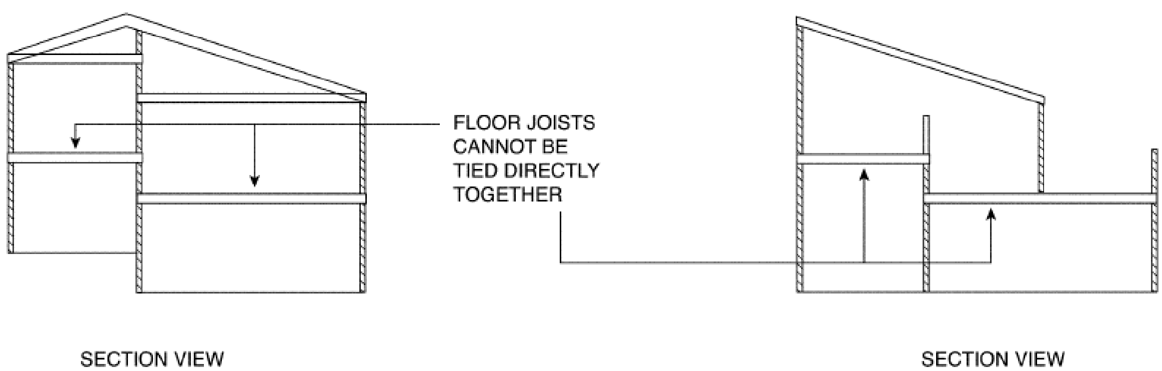


FIGURE 2308.4.4.2
PORTIONS OF FLOOR LEVEL OFFSET VERTICALLY

2308.4.5 Joists supporting bearing partitions. Bearing partitions parallel to joists shall be supported on beams, girders, doubled joists, walls or other bearing partitions. Bearing partitions perpendicular to joists shall not be offset from supporting girders, walls or partitions more than the joist depth unless such joists are of sufficient size to carry the additional load.

2308.4.6 Lateral support. Floor and ceiling framing with a nominal depth-to-thickness ratio greater than or equal to 5:1 shall have one edge held in line for the entire span. Where the nominal depth-to- thickness ratio of the framing member exceeds 6:1, there shall be one line of bridging for each 8 feet (2438 mm) of span, unless both edges of the member are held in line. The bridging shall consist of not less than 1-inch by 3-inch (25 mm by 76 mm) lumber, double nailed at each end, of equivalent metal bracing of equal rigidity, full-depth solid blocking or other *approved* means. A line of bridging shall also be required at supports where equivalent lateral support is not otherwise provided.

2308.4.7 Structural floor sheathing. Structural floor sheathing shall comply with the provisions of Section 2304.7.1.

2308.4.8 Under-floor ventilation. For under-floor ventilation, see Section 1203.3.

2308.4.9 Floor framing supporting *braced wall panels*. When *braced wall panels* are supported by cantilevered floors or are setback from the floor joist support the floor framing shall comply section 2308.6.7.

2308.4.10 Anchorage of exterior means of egress components in *Seismic Design Category D and E*. Exterior egress balconies, exterior exit stairways and similar *means of egress* components in structures assigned to *Seismic Design Category D or E* shall be positively anchored to the primary structure at not over 8 feet (2438 mm) o.c. or shall be designed for lateral forces. Such attachment shall not be accomplished by use of toenails or nails subject to withdrawal.

2308.5 Wall construction. Walls of *conventional light-frame construction* shall be in accordance with this section.

2308.5.1 Stud size, height and spacing. The size, height and spacing of studs shall be in accordance with Table 2308.5.1

Studs shall be continuous from a support at the sole plate to a support at the top plate to resist loads perpendicular to the wall. The support shall be a foundation or floor, ceiling or roof diaphragm or shall be designed in accordance with accepted engineering practice.

> **Exception:** Jack studs, trimmer studs and cripple studs at openings in walls that comply with Table 2308.4.1(1) or 2308.4.1(2).

2308.5.2 Framing details. Studs shall be placed with their wide dimension perpendicular to the wall. Not less than three studs shall be installed at each corner of an *exterior wall*.

Exceptions:

1. In interior nonbearing walls and partition, studs are permitted to be set with the long dimension parallel to the wall.
2. At corners, two studs are permitted, provided wood spacers or backup cleats of 3/8-inch-thick (9.5 mm) wood structural panel, 3/8-inch (9.5 mm) Type M “Exterior Glue” particleboard, 1-inch-thick (25 mm) lumber or other *approved* devices that will serve as an adequate backing for the attachment of facing materials are used. Where fire-resistance ratings or shear values are involved, wood spacers, backup cleats or other devices shall not be used unless specifically *approved* for such use.

TABLE 2308.5.1
SIZE, HEIGHT AND SPACING OF WOOD STUDS[c]

STUD SIZE (inches)	BEARING WALLS				NONBEARING WALLS	
	Laterally unsupported stud height[a] (feet)	Supporting roof and ceiling only	Supporting one floor, roof and ceiling	Supporting two floors, roof and ceiling	Laterally unsupported stud height[a] (feet)	Spacing (inches)
		Spacing (inches)				
2 × 3[b]	NP	NP	NP	NP	10	16
2 × 4	10	24	16	NP	14	24
3 × 4	10	24	24	16	14	24
2 × 5	10	24	24	NP	16	24
2 × 6	10	24	24	16	20	24

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm.
NP=Not Permitted
a. Listed heights are distances between points of lateral support placed perpendicular to the plane of the wall. Increases in unsupported height are permitted where justified by an analysis.
b. Shall not be used in exterior walls.
c. Utility-grade studs shall not be spaced more than 16 inches (406 mm) o.c., or support more than a roof and ceiling, or exceed 8 feet (2438 mm) in height for exterior walls and load-bearing walls or 10 feet (3048 mm) for interior non-load-bearing walls.

2308.5.3 Plates and sills. Studs shall have plates and sills according to this section.

2308.5.3.1. Bottom plate or sill. Studs shall have full bearing on a plate or sill. Plates or sills shall not be less than 2 inches (51 mm) nominal in thickness and have a width at least equal to the width of the wall studs.

2308.5.3.2 Top plates. Studs shall be capped with double top plates installed to provide overlapping at corners and at intersections with other partitions. End joints in double top plates shall be offset at least 48 inches (1219 mm), and shall be nailed in accordance with Table 2304.9.1. Plates shall be a nominal 2 inches (51 mm) in depth and have a width at least equal to the width of the studs.

> **Exception:** A single top plate is permitted, provided the plate is adequately tied at joints, corners and intersecting walls by at least the equivalent of 3-inch by 6-inch (76 mm by 152 mm) by 0.036-inch-thick (0.914 mm) galvanized steel connector that is nailed to each wall or segment of wall by six 8d nails or equivalent, provided the rafters, joists or trusses are centered over the studs with a tolerance of not more than 1 inch (25 mm).

Where bearing studs are spaced at 24-inch (610 mm) intervals and top plates are less than two 2- inch by 6-inch (51 mm by 152 mm) or two 3-inch by 4- inch (76 mm by 102 mm) members and where the floor joists, floor trusses or roof trusses that they support are spaced at more than 16-inch (406 mm) intervals, such joists or trusses shall bear within 5 inches (127 mm) of the studs beneath or a third plate shall be installed.

2308.5.4 Nonbearing walls and partitions. In nonbearing walls and partitions, studs shall be spaced not more than 28 inches (711 mm) o.c. and in interior nonbearing walls and partitions, are permitted to be set with the long dimension parallel to the wall. Interior nonbearing partitions shall be capped with no less than a single top plate installed to provide overlapping at corners and at intersections with other walls and partitions. The plate shall be continuously tied at joints by solid blocking at least 16 inches (406 mm) in length and equal in size to the plate or by 1/2-inch by 1-1/2-inch (12.7 mm by 38 mm) metal ties with spliced sections fastened with two 16d nails on each side of the joint.

2308.5.5 Openings in walls and partitions. Openings in exterior and interior walls and partitions shall comply with sections 2308.5.5.1 through 2308.5.5.3

2308.5.5.1 Openings in exterior bearing walls. Headers shall be provided over each opening in exterior bearing walls. The size and spans in Table 2308.4.1(1) are permitted to be used for one- and two-family *dwellings*. Headers for other buildings shall be designed in accordance with Section 2301.2, Item 1 or 2. Headers shall be of two pieces of nominal 2-inch (51mm) framing lumber set on edge as permitted by Table 2308.4.1(1) and nailed together in accordance with Table 2304.9.1 or of solid lumber of equivalent size.

Wall studs shall support the ends of the header in accordance with Tables 2308.4.1(1). Each end of a lintel or header shall have a bearing length of not less than 1-1/2 inches (38 mm) for the full width of the lintel.

2308.5.5.2 Openings in interior bearing partitions. Headers shall be provided over each opening in interior bearing partitions as required in Section 2308.5.5.1 The spans in Table 2308.4.1(2) are permitted to be used. Wall studs shall support the ends of the header in accordance with Table 2308.4.1(1) or 2308.4.1(2), as appropriate.

2308.5.5.3 Openings in interior nonbearing partitions. Openings in nonbearing partitions are permitted to be framed with single studs and headers. Each end of a lintel or header shall have a bearing length of not less than 11/2 inches (38 mm) for the full width of the lintel.

2308.5.6 Cripple walls. Foundation cripple walls shall be framed of studs not less in size than the studding above with a minimum length of 14 inches (356 mm), 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 2308.6.5 for cripple wall bracing.

2308.5.7 Bridging. Unless covered by interior or *exterior wall coverings* or sheathing meeting the minimum requirements of this code, stud partitions or walls with studs having a height-to-least-thickness ratio exceeding 50 shall have bridging not less than 2 inches (51 mm) in thickness and of the same width as the studs fitted snugly and nailed thereto to provide adequate lateral support. Bridging shall be placed in every stud cavity and at a frequency such that no stud so braced shall have a height-to-least-thickness ratio exceeding 50 with the height of the stud measured between horizontal framing and bridging or between bridging, whichever is greater.

2308.5.8 Pipes in walls. Stud partitions containing plumbing, heating or other pipes shall be so framed and the joists underneath so spaced as to give proper clearance for the piping. Where a partition containing such piping runs parallel to the floor joists, the joists underneath such partitions shall be doubled and spaced to *permit* the passage of such pipes and shall be bridged. Where plumbing, heating or other pipes are placed in or partly in a partition, necessitating the cutting of the soles or plates, a metal tie not less than 0.058 inch (1.47 mm) (16 galvanized gage) and 11/2 inches (38 mm) wide shall be fastened to each plate across and to each side of the opening with not less than six 16d nails.

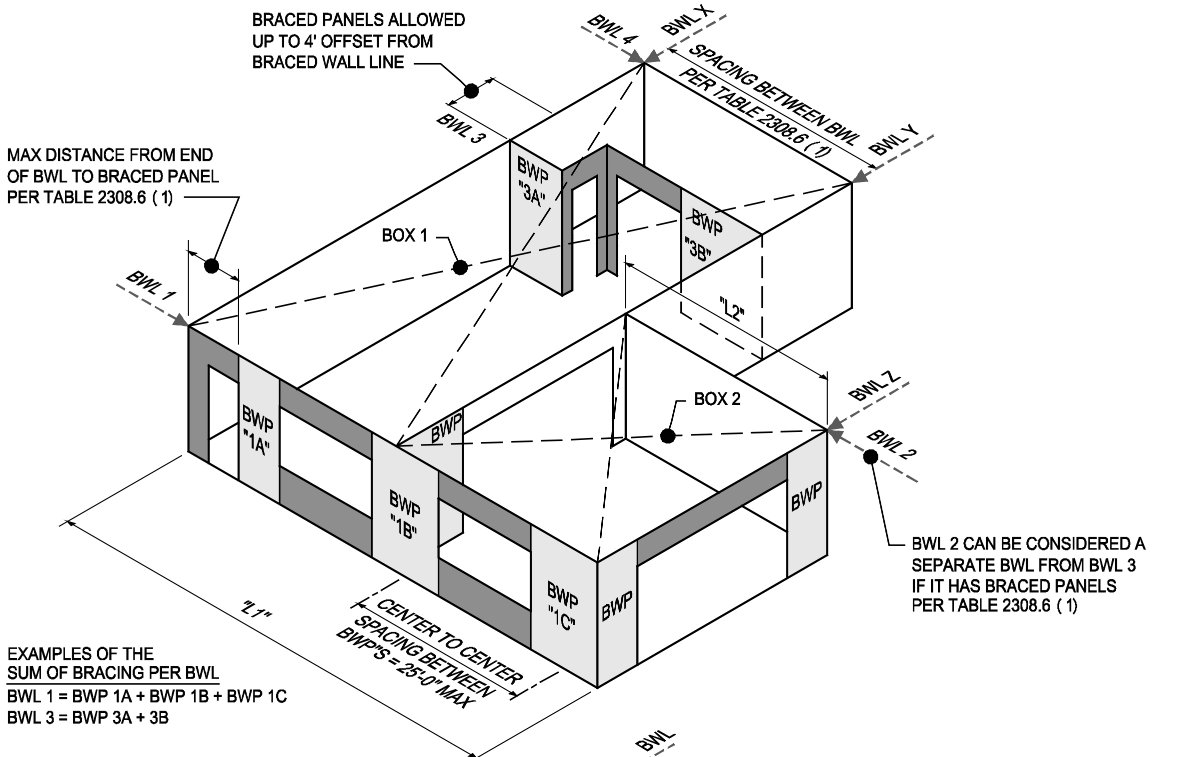
2308.5.9 Cutting and notching. In exterior walls and bearing partitions, any wood stud is permitted to be cut or notched to a depth not exceeding 25 percent of its width. Cutting or notching of studs to a depth not greater than 40 percent of the width of the stud is permitted in nonbearing partitions supporting no loads other than the weight of the partition.

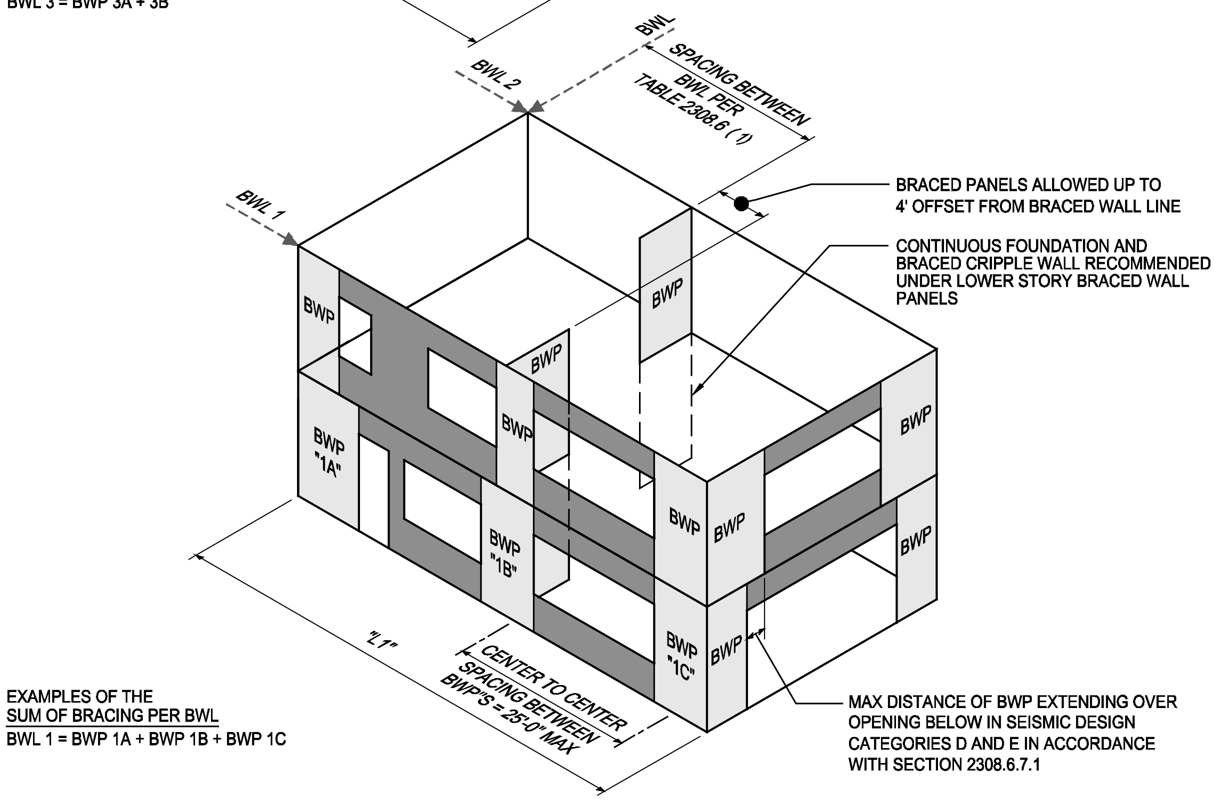
2308.5.10 Bored holes. A hole not greater in diameter than 40 percent of the stud width is permitted to be bored in any wood stud. Bored holes not greater than 60 percent of the width of the stud are permitted in nonbearing partitions or in any wall where each bored stud is doubled, provided not more than two such successive doubled studs are so bored. In no case shall the edge of the bored hole 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.

2308.6 Wall Bracing. Buildings shall be provided with exterior and interior braced wall lines as described in Sections 2308.6.1 through 2308.6.9.2.

2308.6.1 *Braced wall lines*. For the purpose of determining the amount and location of bracing required along each story level of a building, *braced wall lines* shall be designated as straight lines through the building plan in both the longitudinal and transverse direction and placed in accordance with Table 2308.6.1 and Figure 2308.6.1. *Braced wall line* spacing shall not exceed the distance specified in Table 2308.6.1. In structures assigned to Seismic Design Category D or E, braced wall lines shall intersect perpendicularly to each other.

2308.6.2 *Braced wall panels*. *Braced wall panels* shall be placed along *braced wall lines* in accordance with Table 2308.6.1 and Figure 2308.6(1) and specified in Table 2308.6.2(1). A *braced wall panel* must be located at each end of the braced wall line and at the corners of intersecting *braced wall lines* or may begin within the maximum distance from the end of the *braced wall line* in accordance with Table 2308.6(1). *Braced wall panels* in a *braced wall line* shall not be offset from each other by more than 4 feet (1219 mm). Braced wall panels shall be clearly indicated on the plans.

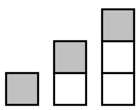
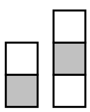
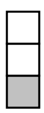
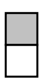
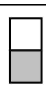
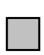




<u>For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm.</u>
<u>BWL = BRACED WALL LINE, BWP = BRACED WALL PANEL</u>

<u>Figure 2308.6(1)</u>
<u>BASIC COMPONENTS OF THE LATERAL BRACING SYSTEM</u>

TABLE 2308.1
WALL BRACING REQUIREMENTS

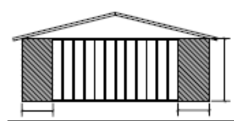
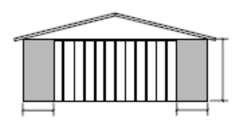
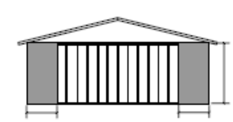
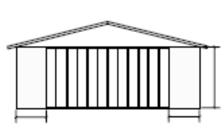
Seismic Design Category	**Story Condition** (See section 2308.2)	**Maximum spacing of braced wall lines**	**Braced panel location, spacing (o.c.) and minimum percentage (x)** — **Bracing Method** — **LIB**	**DWB WSP**	**SFB PBS PCP HPS GB,** [c,d]	**Maximum distance of braced wall panels from each end of braced wall line**
A and B		35'-0"	Each end and ≤25'-0" o.c.	Each end and ≤25'-0" o.c.	Each end and ≤25'-0" o.c.	12'-6"
		35'-0"	Each end and ≤25'-0" o.c.	Each end and ≤25'-0" o.c.	Each end and ≤25'-0" o.c.	12'-6"
		35'-0"	NP	Each end and ≤25'-0" o.c.	Each end and ≤25'-0" o.c.	12'-6"
C		35'-0"	NP	Each end and ≤25'-0" o.c.	Each end and ≤25'-0" o.c.	12'-6"
		35'-0"	NP	Each end and ≤25'-0" o.c. (min 25% of wall length) [e]	Each end and ≤25'-0" o.c. (min 25% of wall length) [e]	12'-6"
D and E		25'-0"	NP	Sds < 0.50: Each end and ≤25'-0" o.c. (min 21% of wall length) [e]	Sds < 0.50: Each end and ≤25'-0" o.c. (min 43% of wall length) [e]	8'-0"
				0.5 ≤ Sds < 0.75: Each end and ≤25'-0" o.c. (min 32% of wall length) [e]	0.5 ≤ Sds < 0.75: Each end and ≤25'-0" o.c. (min 59% of wall length) [e]	
				0.75 ≤Sds ≤ 1.00: Each end and ≤25'-0" o.c. (min 37% of wall length) [e]	0.75 ≤Sds ≤ 1.00: Each end and ≤25'-0" o.c. (min 75% of wall length) [e]	
				Sds > 1.00: Each end and ≤25'-0" o.c. (min 48% of wall length) [e]	Sds > 1.00: Each end and ≤25'-0" o.c. (min 100% of wall length) [e]	

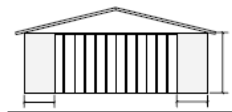
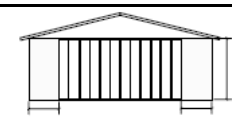
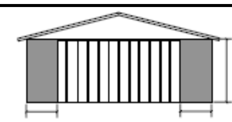
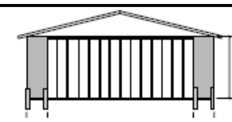
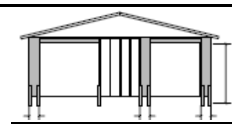
For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm.
NP = Not Permitted

a. This table specifies minimum requirements for *braced wall panels* along interior or exterior *braced wall lines*.
b. See Section 2308.6.2 for full description of bracing methods.
c. Gypsum wallboard applied to framing supports that are spaced at 16 inches on center.
d. The required lengths shall be doubled for gypsum board applied to only one face of a braced wall panel.
e. Percentage shown represents the minimum amount of bracing required along the building length (or wall length if the structure has an irregular shape)

2308.6.3 Braced wall panel methods. Construction of *braced wall panels* shall be by one or a combination of the methods in Table 2308.6.3(1). *Braced wall panel* length shall be in accordance with Section 2308.6.4 or 2308.6.5.

TABLE 2308.6.3(1)
BRACING METHODS

METHODS, MATERIAL	**MINIMUM THICKNESS**	**FIGURE**	**CONNECTION CRITERIA [a]**	
			Fasteners	**Spacing**
LIB[a] Let-in-bracing	1x4 wood or approved metal straps attached at 45° to 60° angles to studs at maximum of 16" o.c.		Per Fastener Table 2304.9.1, item 20	Wood: per stud plus top and bottom plates
			Metal strap: installed per manufacturer's installation recommendations	Metal strap: installed per manufacturer's installation recommendations
DWB Diagonal wood boards	$^{3}/_{4}$" thick (1" nominal) x 6" minimum width to studs at maximum of 24" o.c.		Per Fastener Table 2304.9.1, item 21 or 22	Per stud
WSP Wood structural panel	$^{3}/_{8}$" Per TABLE 2308.6.3(2) or 2308.6.3(3)		Per Fastener Table 2304.9.1, item 31	6" edges 12" field
SFB Structural fiberboard sheathing	$^{1}/_{2}$" Per TABLE 2308.6.3(4)		Per Fastener Table 2304.9.1, item 33	3" edges 6" field
GB Gypsum board (Double sided)	$^{1}/_{2}$" by a minimum of 4 feet wide to studs at maximum of 24" o.c.		Exterior and interior sheathing: with 5d cooler nails (1-5/8" x 0.086") or 1¼" screws (type W or S) for ½" gypsum board or $1^{5}/_{8}$" screws (type	For all braced wall panel locations: 7" o.c. along panel edges (including top and bottom plates) and

METHODS, MATERIAL	MINIMUM THICKNESS	FIGURE	CONNECTION CRITERIA [a]	
			W or S) for $^{5}/_{8}$" gypsum board.	7" o.c.in the field
PBS Particle-board sheathing	$^{3}/_{8}$" or $^{1}/_{2}$" per Table 2308.9.3(4) to studs at maximum of 16" o.c.		6d common (2" long x0.113" dia.) nails for $^{3}/_{8}$" thick sheathing or 8d common (2½" long x 0.131" dia.) nails for $^{1}/_{2}$" thick sheathing	3" edges 6" field
PCP Portland cement plaster	See Section 2510 to studs at maximum of 16" o.c.		1½" long, 11 gage, $^{7}/_{16}$" dia. head nails or $^{7}/_{8}$" long, 16 gage staples	6" o.c. on all framing members
HPS Hardboard panel siding	$^{7}/_{16}$" TABLE 2308.6.3(5)		Per Fastener Table 2308.9.1	4" edges 8" field
ABW Alternate braced wall.	$^{3}/_{8}$"		See Figure 2308.6.5(1) and Section 2308.6.5.1	See Figure 2308.6.3(1)
PFH Portal frame with hold-downs	$^{3}/_{8}$		See Figure 2308.6.5(2) and Section 2308.6.5.2	See Figure 2308.6.3(2)

For SI: 1 foot 305 mm

a. Method LIB shall have gypsum board fastened to at least one side with nails or screws.

TABLE 2308.6.3(2)
EXPOSED PLYWOOD PANEL SIDING

MINIMUM THICKNESS[a] (inch)	MINIMUM NUMBER OF PLIES	STUD SPACING (inches) Plywood siding applied directly to studs or over sheathing
$^{3}/_{8}$	3	16[b]
$^{1}/_{2}$	4	24

For SI: 1 inch = 25.4 mm.

a. Thickness of grooved panels is measured at bottom of grooves.

b. Spans are permitted to be 24 inches if plywood siding applied with face grain perpendicular to studs or over one of the following: (1) 1-inch board sheathing, (2) $^{7}/_{16}$-inch wood structural panel sheathing or (3) $^{3}/_{8}$-inch wood structural panel sheathing with strength axis (which is the long direction of the panel unless otherwise marked) of sheathing perpendicular to studs.

TABLE 2308.6.3(3)
WOOD STRUCTURAL PANEL WALL SHEATHING[b]
(Not Exposed to the Weather, Strength Axis Parallel or Perpendicular to Studs Except as Indicated Below)

MINIMUM THICKNESS (inch)	PANEL SPAN RATING	STUD SPACING (inches)		
		Siding nailed to studs	Nailable sheathing	
			Sheathing parallel to studs	Sheathing perpendicular to studs
$^3/_8$, $^{15}/_{32}$, $^1/_2$	16/0, 20/0, 24/0, 32/16 Wall—24″ o.c.	24	16	24
$^7/_{16}$, $^{15}/_{32}$, $^1/_2$	24/0, 24/16, 32/16 Wall—24″ o.c.	24	24[a]	24

For SI: 1 inch = 25.4 mm.

a. Plywood shall consist of four or more plies.

b. Blocking of horizontal joints shall not be required except as specified in Sections 2306.3 and 2308.12.4.

TABLE 2308.6.3(4)
ALLOWABLE SPANS FOR PARTICLEBOARD WALL SHEATHING
(Not Exposed to the Weather, Long Dimension of the Panel Parallel or Perpendicular to Studs)

GRADE	THICKNESS (inch)	STUD SPACING (inches)	
		Siding nailed to studs	Sheathing under coverings specified in Section 2308.9.3 parallel or perpendicular to studs
M-S "Exterior Glue" and M-2 "Exterior Glue"	$^3/_8$	16	—
	$^1/_2$	16	16

For SI: 1 inch = 25.4 mm.

TABLE 2308.6.3(5)
HARDBOARD SIDING

SIDING	MINIMUM NOMINAL THICKNESS (inch)	2 × 4 FRAMING MAXIMUM SPACING	NAIL SIZE[a, b, d]	NAIL SPACING	
				General	Bracing panels[c]
1. Lap siding					
Direct to studs	$^3/_8$	16″ o.c.	8d	16″ o.c.	Not applicable
Over sheathing	$^3/_8$	16″ o.c.	10d	16″ o.c.	Not applicable
2. Square edge panel siding					
Direct to studs	$^3/_8$	24″ o.c.	6d	6″ o.c. edges; 12″ o.c. at intermediate supports	4″ o.c. edges; 8″ o.c. at intermediate supports
Over sheathing	$^3/_8$	24″ o.c.	8d	6″ o.c. edges; 12″ o.c. at intermediate supports	4″ o.c. edges; 8″ o.c. at intermediate supports
3. Shiplap edge panel siding					
Direct to studs	$^3/_8$	16″ o.c.	6d	6″ o.c. edges; 12″ o.c. at intermediate supports	4″ o.c. edges; 8″ o.c. at intermediate supports
Over sheathing	$^3/_8$	16″ o.c.	8d	6″ o.c. edges; 12″ o.c. at intermediate supports	4″ o.c. edges; 8″ o.c. at intermediate supports

For SI: 1 inch = 25.4 mm.

a. Nails shall be corrosion resistant.

b. Minimum acceptable nail dimensions:

	Panel Siding (inch)	Lap Siding (inch)
Shank diameter	0.092	0.099
Head diameter	0.225	0.240

c. Where used to comply with Section 2308.9.3.

d. Nail length must accommodate the sheathing and penetrate framing $1^1/_2$ inches.

2308.6.4 Length of *braced wall panels*. For Methods DWB, WSP, SFB, PBS, PCP and HPS each panel must be at least 48 inches (1219 mm) in length, covering three stud spaces where studs are spaced 16 inches (406 mm) apart and covering two stud spaces where studs are spaced 24 inches (610 mm) apart. *Braced wall panels* less than the required 48" length shall not contribute towards the amount of bracing required. *Braced wall panels* longer than the required length shall be credited for their actual length. For Method GB, each panel must be at least 96 inches (2438 mm) in length where applied to one side of the studs or 48 inches (1219 mm) where applied to both sides.

All vertical joints of panel sheathing shall occur over studs and adjacent panel joints shall be nailed to common framing members. Horizontal joints shall occur over blocking or other framing equal in size to the studding except where waived by the installation requirements for the specific sheathing materials. Sole plates shall be nailed to the floor framing in accordance with Section 2308.3.2 and top plates shall be connected to the framing above in accordance with Section 2308.5.3. Where joists are perpendicular to braced wall lines above, blocking shall be provided under and in line with the *braced wall panels.*

2308.6.5 Alternative bracing. An Alternate Braced Wall (ABW) or a Portal Frame with Hold-downs (PFH) described in this section is permitted to substitute for a 48" braced wall panel of methods DWB, WSP, SFB, PBS, PCP or HPS. For method GB, each 96- inch (2438 mm) section (applied to one face) or 48-inch (1219 mm) section (applied to both faces) or portion thereof required by Table 2308.6.1 is permitted to be replaced by one panel constructed in accordance with method ABW or PFH.

2308.6.5.1. Alternate Braced Wall (ABW). An ABW shall be constructed in accordance with this section and Figure 2308.6.5.1. In one-story buildings, each panel shall have a length of not less than 2 feet 8 inches (813 mm) and a height of not more than 10 feet (3048 mm). Each panel shall be sheathed on one face with 3/8- inch-minimum-thickness (9.5 mm) wood structural panel sheathing nailed with 8d common or galvanized box nails in accordance with Table 2304.9.1 and blocked at wood structural panel edges. Two anchor bolts installed in accordance with Section 2308.3.1 shall be provided in each panel. Anchor bolts shall be placed at each panel outside quarter points. Each panel end stud shall have a hold-down device fastened to the foundation, capable of providing an *approved* uplift capacity of not less than 1,800 pounds (8006 N). The hold-down device shall be installed in accordance with the manufacturer's recommendations. The ABW shall be supported directly on a foundation or on floor framing supported directly on a foundation that is continuous across the entire length of the braced wall line. This foundation shall be reinforced with not less than one No. 4 bar top and bottom. Where the continuous foundation is required to have a depth greater than 12 inches (305 mm), a minimum 12-inch by 12-inch (305 mm by 305 mm) continuous footing or turned down slab edge is permitted at door openings in the braced wall line. This continuous footing or turned down slab edge shall be reinforced with not less than one No. 4 bar top and bottom. This reinforcement shall be lapped 15 inches (381 mm) with the reinforcement required in the continuous foundation located directly under the braced wall line.

When the ABW is installed at the first *story* of two-story buildings, the wood structural panel sheathing shall be provided on both faces, three anchor bolts shall be placed at one-quarter points, and tie-down device uplift capacity shall not be less than 3,000 pounds (13 344 N).

2308.6.5.2 Portal Frame with Hold-downs (PFH). A PFH shall be constructed in accordance with this section and Figure 2308.6.5.2. The adjacent door or window opening shall have a full-length header.

In one-story buildings, each panel shall have a length of not less than 16 inches (406 mm) and a height of not more than 10 feet (3048 mm). Each panel shall be sheathed on one face with a single layer of 3/8 inch (9.5 mm) minimum thickness wood structural panel sheathing nailed with 8d common or galvanized box nails in accordance with Figure 2308.6.5.2. The wood structural panel sheathing shall extend up over the solid sawn or glued-laminated header and shall be nailed in accordance with Figure 2308.6.5. A built-up header consisting of at least two 2 × 12s and fastened in accordance with Item 24 of Table 2304.9.1 shall be permitted to be used. A spacer, if used, shall be placed on the side of the built-up beam opposite the wood structural panel sheathing. The header shall extend between the inside faces of the first full-length outer studs of each panel. The clear span of the header between the inner studs of each panel shall be not less than 6 feet (1829 mm) and not more than 18 feet (5486 mm) in length. A strap with an uplift capacity of not less than 1,000 pounds (4,400 N) shall fasten the header to the inner studs opposite the sheathing. One anchor bolt not less than 5/8 inch (15.9 mm) diameter and installed in accordance with Section 2308.3.1 shall be provided in the center of each sill plate. The studs at each end of the panel shall have a hold-down device fastened to the foundation with an uplift capacity of not less than 4,200 pounds (18 480 N).

Where a panel is located on one side of the opening, the header shall extend between the inside face of the first full-length stud of the panel and the bearing studs at the other end of the opening. A strap with an uplift capacity of not less than 1,000 pounds (4400 N) shall fasten the header to the bearing studs. The bearing studs shall also have a hold-down device fastened to the foundation with an uplift capacity of not less than 1,000 pounds (4400 N). The hold-down devices shall be an embedded strap type, installed in accordance with the manufacturer's recommendations. The PFH panels shall be supported directly on a foundation that is continuous across the entire length of the braced wall line. This foundation shall be reinforced with not less than one No. 4 bar top and bottom. Where the continuous foundation is required to have a depth greater than 12 inches (305 mm), a minimum 12-inch by 12-inch (305 mm by 305 mm) continuous footing or turned down slab edge is permitted at door openings in the braced wall line. This continuous footing or turned down slab edge shall be reinforced with not less than one No. 4 bar top and bottom. This reinforcement shall be lapped not less than 15 inches (381 mm) with the reinforcement required in the continuous foundation located directly under the braced wall line.

When a PFH is installed at the first *story* of two-story buildings, each panel shall have a length of not less than 24 inches (610 mm).

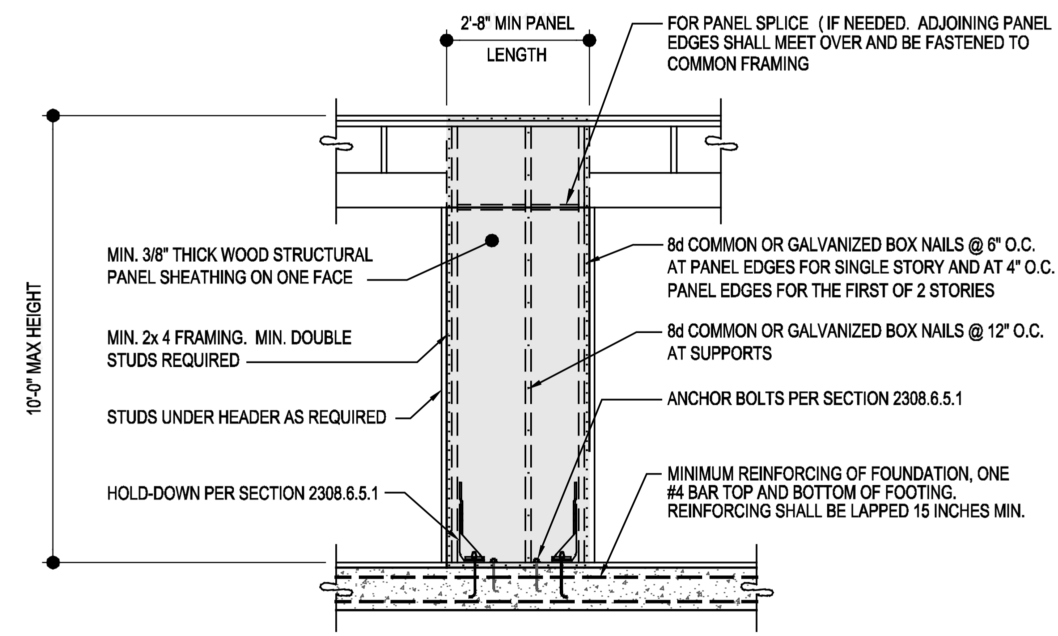


FIGURE 2308.6.5.1
ALTERNATE BRACED WALL PANEL (ABW)

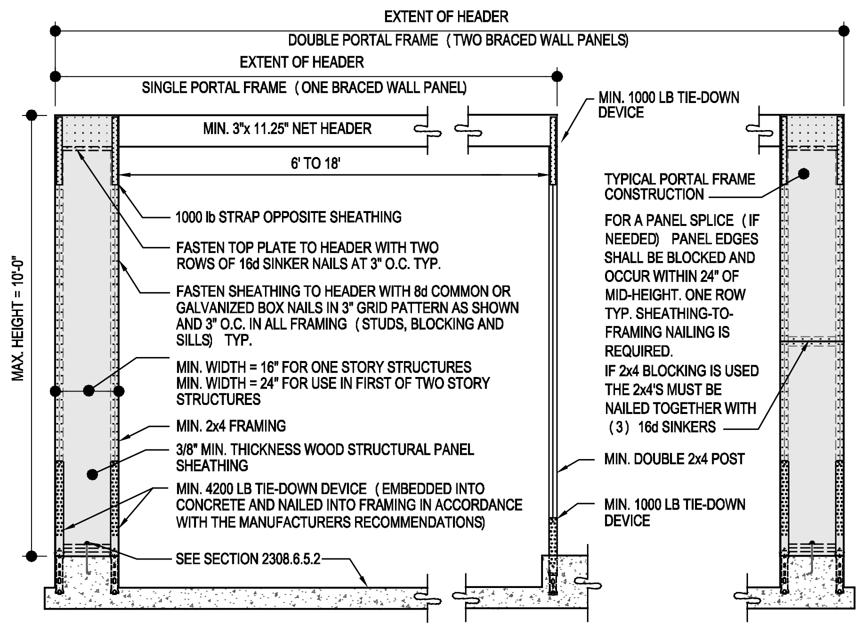


Figure 2308.6.5.2
PORTAL FRAME WITH HOLD-DOWNS (PFH)

2308.6.5 Cripple wall bracing. Cripple walls shall be braced in accordance with the following.

2308.6.5.1 Cripple wall bracing in *Seismic Design Category* A, B and C.. For the purposes of this section, cripple walls having a stud height exceeding 14 inches (356 mm) shall be considered a *story* and shall be braced in accordance with Table 2308.6(1). Spacing of edge nailing for required cripple wall bracing shall not exceed 6 inches (152mm) o.c. along the foundation plate and the top plate of the cripple wall. Nail size, nail spacing for field nailing and more restrictive boundary nailing requirements shall be as required elsewhere in the code for the specific bracing material used.

2308.6.5.2 Cripple wall bracing in *Seismic Design Category* D and E For the purposes of this section, cripple walls having a stud height exceeding 14 inches (356 mm) shall be considered a *story* and shall be braced 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) o.c.

2308.6.6 Connections of *braced wall panels*. Braced wall panel joints shall occur over studs or blocking. *Braced wall panels* shall be fastened to studs, top and bottom plates and at panel edges. *Braced wall panels* shall be applied to nominal 2-inch-wide [actual 1-1/2 inch (38 mm)] or larger stud framing.

2308.6.6.1 Bottom plate connection. Braced wall line bottom plates shall be connected to joists or full-depth blocking below in accordance with Table 2304.9.1, Item 6, or to foundations in accordance with Section 2308.3.3.

2308.6.6.2 Top plate connection. Where joists and/or rafters are used, braced wall line top plates shall be fastened over the full length of the braced wall line to joists, rafters, rim boards or blocking above in accordance with Table 2304.9.1, as applicable, based on the orientation of the joists or rafters to the braced wall line. Blocking at joists with walls above shall be equal to the depth of the joist at the braced wall line. Blocking at rafters need not be full depth but shall extend to within 2 inches (51 mm) from the roof sheathing above. Blocking shall be a minimum of 2 inches (51 mm) nominal thickness and shall be fastened to the braced wall line top plate as specified in Table 2304.9.1, Item 11. Notching or drilling of holes in blocking in accordance with the requirements of Section 2308.8.2 or Section 2308.10.4.2 shall be permitted.

At exterior gable end walls braced wall panel sheathing in the top *story* shall be extended and fastened to roof framing where the spacing between parallel exterior braced wall lines is greater than 50 feet (15 240 mm).

Where roof trusses are used and are installed perpendicular to an exterior braced wall line, lateral forces shall be transferred from the roof diaphragm to the braced wall over the full length of the braced wall line by blocking of the ends of the trusses or by other *approved* methods providing equivalent lateral force transfer. Blocking shall be minimum 2 inches (51 mm) nominal thickness and shall extend to within 2 inches (51 mm) from the roof sheathing above and shall be fastened to the braced wall line top plate as specified in Table 2304.9.1. Notching or drilling of holes in blocking in accordance with the requirements of Section 2308.4.2.4 or Section 2308.7.4 shall be permitted.

2308.6.6.3 Sill anchorage. Where foundations are required by Section 2308.6.7, braced wall line sills shall be anchored to concrete or masonry foundations. Such anchorage shall conform to the requirements of Section 2308.3. The anchors shall be distributed along the length of the braced wall line. Other anchorage devices having equivalent capacity are permitted.

2308.6.6.4 Anchorage to all-wood foundations. Where all-wood foundations are used, the force transfer from the braced wall lines shall be determined based on calculation and shall have a capacity greater than or equal to the connections required by Section 2308.3.

2308.6.7 Braced wall line and diaphragm support. Braced wall lines and floor and roof diaphragms shall be supported in accordance to this section.

2308.6.7.1 Foundation requirements. Braced wall lines shall be supported by continuous foundations.

> **Exception:** For structures with a maximum plan dimension not over 50 feet (15 240 mm), continuous foundations are required at exterior walls only.

For structures in *Seismic Design Category D and E,* exterior *braced wall panels* shall be in the same plane vertically with the foundation or the braced wall line shall be designed in accordance with accepted engineering practice according to section 2308.1.1

Exceptions:

1. Exterior *braced wall panels* may be located up to 4 feet from the foundation below when supported by a floor constructed in accordance with all the following:
   - 1.1 Cantilevers or setbacks shall not exceed four times the nominal depth of the floor joists
   - 1.2. Floor joists shall be 2 inches by 10 inches (51 mm by 254 mm) or larger and spaced not more than 16 inches (406 mm) o.c.
   - 1.3. The ratio of the back span to the cantilever shall be at least 2:1.
   - 1.4. Floor joists at ends of *braced wall panels* shall be doubled.
   - 1.5. A continuous rim joist shall be connected to the ends of cantilevered joists. The rim joist is permitted to be spliced using a metal tie not less than 0.058 inch (1.47 mm) (16 galvanized gage) and 11/2 inches (38 mm) wide fastened with six 16d common nails on each side. The metal tie shall have a minimum yield stress of 33,000 psi (227 MPa).

1.6. Joists at setbacks or the end of cantilevered joists shall not carry gravity loads from more than a single *story* having uniform wall and roof loads, nor carry the reactions from headers having a span of 8 feet (2438 mm) or more.

2. The end of a required braced wall panel shall be allowed to extend not more than 1 foot (305 mm) over an opening in the wall below. This requirement is applicable to *braced wall panels* offset in plane and to *braced wall panels* offset out of plane as permitted by the exception to Item 1 above in this section.

   Exception: *Braced wall panels* are permitted to extend over an opening not more than 8 feet (2438 mm) in width where the header is a 4-inch by 12-inch (102 mm by 305 mm) or larger member.

2308.6.7.2 Floor and roof diaphragm support in *Seismic Design Category D and E*. In structures assigned to *Seismic Design Category D or E,* floor and roof diaphragms shall be laterally supported by braced wall lines on all edges and connected in accordance with Section 2308.3.2 [see Figure 2308.6.7.2(1)].

Exception: Portions of roofs or floors that do not support *braced wall panels* above are permitted to extend up to 6 feet (1829 mm) beyond a braced wall line [see Figure 2308.6.7.2(2)] provided that the framing members are connected to the braced wall line below in accordance with Section 2308.6.6.

2308.6.7.3 Stepped footings in Seismic Design Category B, C, D and E . Where the height of a required braced wall panel extending from foundation to floor above varies more than 4 feet (1219 mm), the following construction shall be used:

1. Where the bottom of the footing is stepped and the lowest floor framing rests directly on a sill bolted to the footings, the sill shall be anchored as required in Section 2308.3.3.
2. Where the lowest floor framing rests directly on a sill bolted to a footing not less than 8 feet (2438 mm) in length along a line of bracing, the line shall be considered to be braced. The double plate of the cripple stud wall beyond the segment of footing extending to the lowest framed floor shall be spliced to the sill plate with metal ties, one on each side of the sill and plate. The metal ties shall not be less than 0.058 inch [1.47 mm (16 galvanized gage)] by 11/2 inches (38 mm) wide by 48 inches (1219 mm) with eight 16d common nails on each side of the splice location (see Figure 2308.6.7.3(1). The metal tie shall have a minimum yield stress of 33,000 pounds per square inch (psi) (227 MPa).
3. Where cripple walls occur between the top of the footing and the lowest floor framing, the bracing requirements for a *story* shall apply.

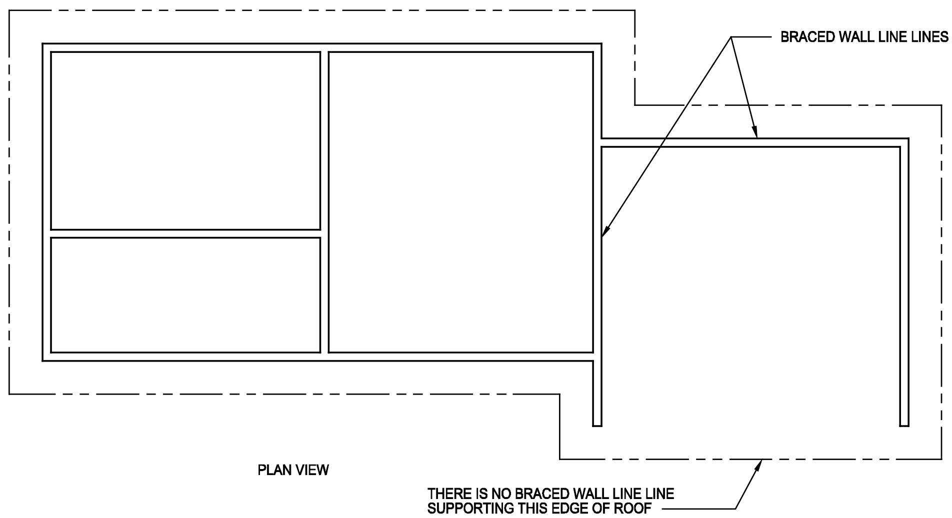


FIGURE 2308.6.7.2(1)
ROOF IN SDC D OR E NOT SUPPORTED ON ALL EDGES

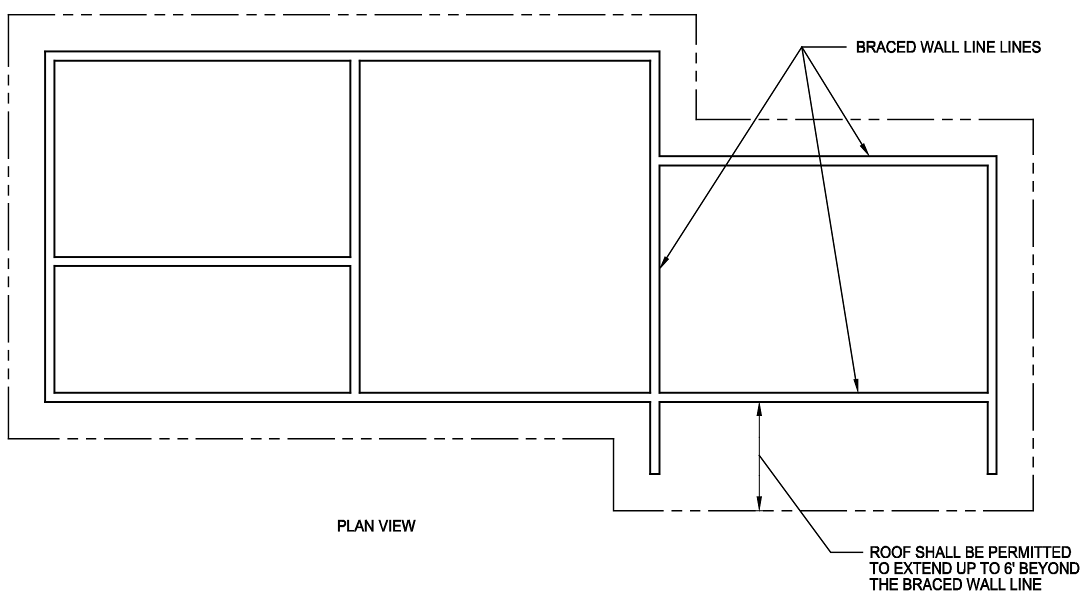
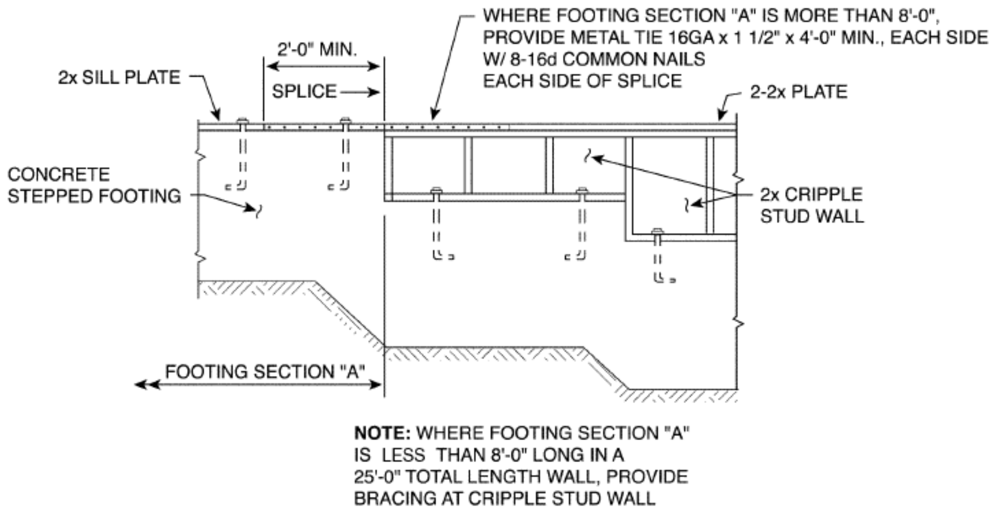


FIGURE 2308.6.7.2(2)
ROOF EXTENSION IN SDC D OR E BEYOND BRACED WALL LINE



For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm.

FIGURE 2308.6.7.3(1)
STEPPED FOOTING CONNECTION DETAILS

2308.6.8 Attachment of sheathing. Fastening of braced wall panel sheathing shall not be less than that prescribed in Tables 2308.6(1) and 2304.9.1. Wall sheathing shall not be attached to framing members by adhesives.

2308.6.9 Limitations of concrete or masonry veneer. Concrete or masonry veneer shall comply with Chapter 14 and this section.

2308.6.9.1 Limitations of concrete or masonry veneer in *Seismic Design Categories* B or C.. Concrete or masonry walls and stone or masonry veneer shall not extend above a basement.

Exceptions:

1. In structures assigned to *Seismic Design Category* B, stone and masonry veneer is permitted to be used in the first two stories above grade plane or the first three stories above grade plane where the lowest story has concrete or masonry walls, provided that structural use panel wall bracing is used and the length of bracing provided is one and one-half times the required length as determined in Table 2308.9.3(1).
2. Stone and masonry veneer is permitted to be used in the first story above grade plane or the first two stories above grade plane where the lowest story has concrete or masonry walls.
3. Stone and masonry veneer is permitted to be used in both stories of buildings with two stories above grade plane, provided the following criteria are met:
   3.1. Type of brace per Section 2308.9.3 shall be WSP and the allowable shear capacity in accordance with Section 2306.3 shall be a minimum of 350 plf (5108 N/m).
   3.2. *Braced wall panels* in the second *story* shall be located in accordance with Section 2308.9.3 and not more than 25 feet (7620 mm) on center, and the total length of braced wall panels shall be not less than 25 percent of the braced wall line length. *Braced wall panels* in the first *story* shall be located in accordance with Section 2308.9.3 and not more than 25 feet (7620 mm) on center, and the total length of *braced wall panels* shall be not less than 45 percent of the braced wall line length.
   3.3. Hold-down connectors shall be provided at the ends of each *braced wall panel* for the second *story* to first *story* connection with an allowable capacity of 2,000 pounds (8896 N). Hold-down connectors shall be provided at the ends of each *braced wall panel* for the first story to foundation connection with an allowable capacity of 3,900

pounds (17 347 N). In all cases, the hold-down connector force shall be transferred to the foundation.

3.4. Cripple walls shall not be permitted.

2308.6.9.2 Limitations of concrete or masonry in *Seismic Design Categories D and E* Concrete or masonry walls and stone or masonry veneer shall not extend above a basement.

Exception: In structures assigned to *Seismic Design Category* D, stone and masonry veneer is permitted to be used in the first story above grade plane, provided the following criteria are met:

1. Type of brace in accordance with Section 2308.9.3 shall be WSP and the allowable shear capacity in accordance with Section 2306.3 shall be a minimum of 350 plf (5108 N/m).
2. The bracing of the first *story* shall be located at each end and at least every 25 feet (7620 mm) o.c. but not less than 45 percent of the braced wall line.
3. Hold-down connectors shall be provided at the ends of braced walls for the first floor to foundation with an allowable capacity of 2,100 pounds (9341 N).
4. Cripple walls shall not be permitted.

2308.7 Roof and ceiling framing. The framing details required in this section apply to roofs having a minimum slope of three units vertical in 12 units horizontal (25-percent slope) or greater. Where the roof slope is less than three units vertical in 12 units horizontal (25-percent slope), members supporting rafters and ceiling joists such as ridge board, hips and valleys shall be designed as beams.

2308.7.1 Ceiling joist spans. Allowable spans for ceiling joists shall be in accordance with Table 2308.7.1(1) or 2308.7.1(2). For other grades and species, refer to the *AF&PA Span Tables for Joists and Rafters*.

~~TABLE 2308.10.2(1)~~ TABLE 2308.7.1(1)
CEILING JOIST SPANS FOR COMMON LUMBER SPECIES
(Uninhabitable Attics Without Storage, Live Load = 10 pounds psf, L/Δ = 240)

(Portions of Table not shown remain unchanged)

~~TABLE 2308.10.2(2)~~ TABLE 2308.7.1(2)
CEILING JOIST SPANS FOR COMMON LUMBER SPECIES
(Uninhabitable Attics With Limited Storage, Live Load = 20 pounds per square foot, L/Δ = 240)

(Portions of Table not shown remain unchanged)

2308.7.2 Rafter spans. Allowable spans for rafters shall be in accordance with Table 2308.7.2(1), 2308.7.2(2), 2308.7.2(3), 2308.7.2(4), 2308.7.2(5) or 2308.7.2(6). For other grades and species, refer to the *AF&PA Span Tables for Joists and Rafters*.

~~TABLE 2308.10.3(1)~~ TABLE 2308.7.2(1)
RAFTER SPANS FOR COMMON LUMBER SPECIES
(Roof Live Load = 20 pounds per square foot, Ceiling Not Attached to Rafters, L/Δ = 180)

(Portions of Table not shown remain unchanged)

~~TABLE 2308.10.3(2)~~ TABLE 2308.7.2(2)
RAFTER SPANS FOR COMMON LUMBER SPECIES
(Roof Live Load = 20 pounds per square foot, Ceiling Attached to Rafters, L/Δ = 240)

(Portions of Table not shown remain unchanged)

~~TABLE 2308.10.3(3)~~ TABLE 2308.7.2(3)
RAFTER SPANS FOR COMMON LUMBER SPECIES
(Ground Snow Load = 30 pounds per square foot, Ceiling Not Attached to Rafters, L/Δ = 180)

(Portions of Table not shown remain unchanged)

~~TABLE 2308.10.3(4)~~ TABLE 2308.7.2(4)
RAFTER SPANS FOR COMMON LUMBER SPECIES
(Ground Snow Load = 50 pounds per square foot, Ceiling Not Attached to Rafters, L/Δ = 180)

(Portions of Table not shown remain unchanged)

~~TABLE 2308.10.3(5)~~ TABLE 2308.7.2(5)
RAFTER SPANS FOR COMMON LUMBER SPECIES
(Ground Snow Load = 30 pounds per square foot, Ceiling Attached to Rafters, L/Δ = 240)

(Portions of Table not shown remain unchanged)

~~TABLE 2308.10.3(6)~~ TABLE 2308.7.2(6)
RAFTER SPANS FOR COMMON LUMBER SPECIES
(Ground Snow Load = 50 pounds per square foot, Ceiling Attached to Rafters, L/Δ = 240)

(Portions of Table not shown remain unchanged)

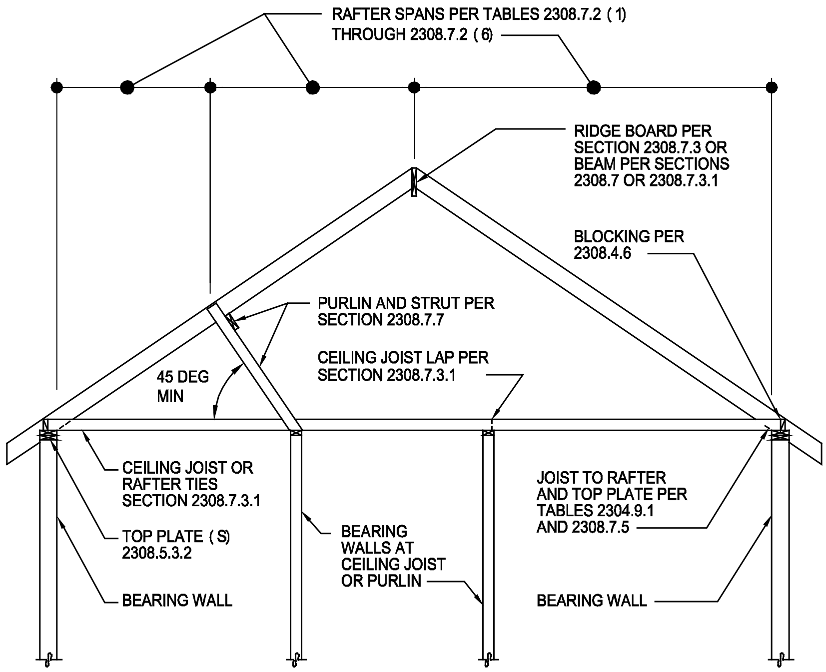
2308.7.3 Ceiling joist and rafter framing. Rafters shall be framed directly opposite each other at the ridge. There shall be a ridge board at least 1-inch (25 mm) nominal thickness at ridges and not less in depth than the cut end of the rafter. At valleys and hips, there shall be a single valley or hip rafter not less than 2-inch (51 mm) nominal thickness and not less in depth than the cut end of the rafter.

2308.7.3.1 Ceiling joist and rafter connections. Ceiling joists and rafters shall be nailed to each other and the assembly shall be nailed to the top wall plate in accordance with Tables 2304.9.1 and 2308.7.5. Ceiling joists shall be continuous or securely joined where they meet over interior partitions and be fastened to adjacent rafters in accordance with Tables 2304.9.1 and 2308.7.3.1 to provide a continuous rafter tie across the building where such joists are parallel to the rafters. Ceiling joists shall have a bearing surface of not less than 1-1/2 inches (38 mm) on the top plate at each end.

Where ceiling joists are not parallel to rafters, an equivalent rafter tie shall be installed in a manner to provide a continuous tie across the building, at a spacing of not more than 4 feet (1219 mm) o.c. The connections shall be in accordance with Tables 2308.7.3.1 and 2304.9.1, or connections of equivalent capacities shall be provided. Where ceiling joists or rafter ties are not provided at the top of the rafter support walls, the ridge formed by these rafters shall also be supported by a girder conforming to Section 2308.2.7. Rafter ties shall be spaced not more than 4 feet (1219 mm) o.c.

Rafter tie connections shall be based on the equivalent rafter spacing in Table 2308.7.3.1. Rafter/ceiling joist connections and rafter/tie connections shall be of sufficient size and number to prevent splitting from nailing.

Roof framing member connection to braced wall lines shall be in accordance with 2308.6.6.2.



For SI: 1 inch = 25.4 mm, 1 foot = 305 mm, 1 degree = 0.018 rad.
Note: Where ceiling joints run perpendicular to the rafter, rafter ties shall be installed per section 2308.7.3.1

FIGURE 2308.7
ROOF CEILING FRAMING

~~TABLE 2308.10.4.1~~ TABLE 2308.7.3.1
RAFTER TIE CONNECTIONS[g]

(Portions of Table not shown remain unchanged)

2308.7.4 Notches and holes. Notching at the ends of rafters or ceiling joists shall not exceed one-fourth the depth. Notches in the top or bottom of the rafter or ceiling joist shall not exceed one-sixth the depth and shall not be located in the middle one-third of the span, except that a notch not exceeding one-third of the depth is permitted in the top of the rafter or ceiling joist not further from the face of the support than the depth of the member. Holes bored in rafters or ceiling joists shall not be within 2 inches (51 mm) of the top and bottom and their diameter shall not exceed one-third the depth of the member.

2308.7.5 Wind uplift. The roof construction shall have rafter and truss ties to the wall below. Resultant uplift loads shall be transferred to the foundation using a continuous load path. The rafter or truss to wall connection shall comply with Tables 2304.9.1 and 2308.7.5

~~TABLE 2308.10.1~~ TABLE 2308.7.5
REQUIRED RATING OF APPROVED UPLIFT CONNECTORS (pounds)[a, b, c, e, f, g, h]

(Portions of Table not shown remain unchanged)

2308.7.6 Framing around openings. Trimmer and header rafters shall be doubled, or of lumber of equivalent cross section, where the span of the header exceeds 4 feet (1219 mm). The ends of header rafters more than 6 feet (1829 mm) long shall be supported by framing anchors or rafter hangers unless bearing on a beam, partition or wall.

2308.7.6.1 Openings in roof diaphragms in *Seismic Design Categories* B, C, D and E. Openings in horizontal diaphragms with a dimension perpendicular to the joist that is greater than 4 feet (1219 mm) shall be constructed with metal ties and blocking in accordance with this section and Figure 2308.4.4.1(1). Metal ties shall not be less than 0.058 inch [1.47 mm (16 galvanized gage)] thick by 1-1/2 inches (38 mm) wide with a minimum yield stress of 33,000 psi (227 Mpa). Blocking shall be provided 2 feet minimum beyond headers. Ties shall be attached to blocking with eight 16d common nails on each side of the header-joist intersection.

2308.7.7 Purlins. Purlins to support roof loads are permitted to be installed to reduce the span of rafters within allowable limits and shall be supported by struts to bearing walls. The maximum span of 2-inch by 4-inch (51 mm by 102 mm) purlins shall be 4 feet (1219 mm). The maximum span of the 2-inch by 6-inch (51 mm by 152 mm) purlin shall be 6 feet (1829 mm), but in no case shall the purlin be smaller than the supported rafter. Struts shall not be smaller than 2-inch by 4-inch (51 mm by 102 mm) members. The unbraced length of struts shall not exceed 8 feet (2438 mm) and the minimum slope of the struts shall not be less than 45 degrees (0.79 rad) from the horizontal.

2308.7.8 Blocking. Roof rafters and ceiling joists shall be supported laterally to prevent rotation and lateral displacement in accordance with the provisions of Section 2308.8.5 and connected to braced wall lines per Section 2308.6.6.2.

2308.7.9 Engineered wood products. Prefabricated wood I-joists, structural glued-laminated timber and structural composite lumber shall not be notched or drilled except where permitted by the manufacturer's recommendations or where the effects of such alterations are specifically considered in the design of the member by a *registered design professional*.

2308.7.10 Roof sheathing. Roof sheathing shall be in accordance with Tables 2304.7(3) and 2304.7(5) for wood structural panels, and Tables 2304.7(1) and 2304.7(2) for lumber and shall comply with Section 2304.7.2.

2308.7.11 Joints. Joints in lumber sheathing shall occur over supports unless *approved* end-matched lumber is used, in which case each piece shall bear on at least two supports.

2308.7.12 Roof planking. Planking shall be designed in accordance with the general provisions of this code.

In lieu of such design, 2-inch (51 mm) tongue-and groove planking is permitted in accordance with Table 2308.10.9. Joints in such planking are permitted to be randomly spaced, provided the system is applied to not less than three continuous spans, planks are center matched and end matched or splined, each plank bears on at least one support, and joints are separated by at least 24 inches (610 mm) in adjacent pieces.

2308.7.13 Wood trusses. Wood trusses shall be designed in accordance with Section 2303.4. Connection to braced wall lines shall be in accordance with Section 2308.6.6.2.

2308.7.14 Attic ventilation. For *attic* ventilation, see Section 1203.2.

Reason: This proposal is intended to completely replace the existing section 2308 "Conventional Light-Frame Construction" with a re-formatted version. This proposal is not intended to introduce any new requirements into, nor remove any requirements from, the existing section 2308.

As a result of many code cycles, Section 2308 has become fragmented and is not organized in a logical manner and is difficult to use. With this proposal, Section 2308 is formatted to begin with general requirements then proceed to foundations, floor framing, wall framing, wall bracing and roof-ceiling construction in that order. The additional requirements for *Seismic Design Categories* in the 2012 IBC Sections 2308.11 and 2308.12 (SDC B/C and SDC D/E respectively) have been merged into the appropriate new sections based on the type of construction such as floor framing, wall bracing and roof framing.

Terminology has been coordinated throughout the section such as the terms, "conventional light-frame construction", "braced wall line" and "braced wall panel".

This proposal is intended to be non-technical and separate proposals have been submitted to address technical items in section 2308.

In order to make the prescriptive provisions of the IBC more closely resemble the format of the similar provisions in the IRC, much of the wall bracing terminology is replicated from the IRC, namely:

- The requirements for braced wall line spacing were put into a single table format based on Seismic Design Category rather than scattered throughout all of Section 2308.
- The wall bracing methods were compiled into a table similar to the IRC, including abbreviations for the methods, rather

than referring to them by a number. The fasteners specified in this table were cross-referenced to the fastener table 2308.9.3.1 where applicable.

- For the section, “Alternate bracing” a figure (copied from the IRC) was introduced, but no technical changes were made.
- Similarly, for Section 2308.9.3.2, “Alternate bracing wall panel adjacent to a door or window opening” was renamed since it aligned perfectly with the Portal Frame with Hold-downs method (PFH) in the IRC. The figure was already in the IBC, so the title was changed to reflect the new name.

Comparison of the proposed 2015 to the existing 2012

<table>
<tr><th>Proposed 2015</th><th>2012 IBC</th></tr>
<tr><td>2308 Conventional Light-Frame Construction</td><td>2308 Conventional Light-Frame Construction</td></tr>
<tr><td>2308.1 General. The requirements of this section are intended for conventional light-frame construction. Other construction methods are permitted to be used, provided a satisfactory design is submitted showing compliance with other provisions of this code. Interior non-load-bearing partitions, ceilings and curtain walls of conventional light-frame construction are not subject to the limitations of <s>this</s> section 2308.2.</td><td>2308.1 General. As shown modified to the left</td></tr>
<tr><td></td><td>2308.1.1 Portions exceeding limitations of conventional construction. Moved to 2308.2.8</td></tr>
<tr><td>2308.2 Limitations</td><td>2308.2 Limitations. Included reference to items in 2308.11 (SDC B and C) and 2308.12 (SDC D and E). Those items have been moved here and elsewhere in the section as noted.</td></tr>
<tr><td>2308.2.1 Stories. The height limitations in the table are from:

2308.2.1 Stories. Structures of conventional light-frame construction shall be limited in story height according to the following:
<table>
<tr><th>Seismic Design Category</th><th>Allowable Story above grade plane</th></tr>
<tr><td>A and B</td><td>Three stories</td></tr>
<tr><td>C</td><td>Two Stories</td></tr>
<tr><td>D and E [a]</td><td>One story</td></tr>
</table>
a. For the purposes of this section, for buildings assigned to Seismic Design Category D or E, unless cripple walls are solid blocked and do not exceed 14 inches in height, cripple walls shall be considered to be a story.</td><td>2308.2 Limitations. Buildings are permitted to be constructed in accordance with the provisions of conventional light-frame construction, subject to the following limitations, and to further limitations of Sections 2308.11 and 2308.12.
1. Buildings shall be limited to a maximum of three stories above grade plane. For the purposes of this section, for buildings assigned to Seismic Design Category D or E, cripple stud walls shall be considered to be a story.
Exception: Solid blocked cripple walls not exceeding 14 inches (356 mm) in height need not be considered a story.

2308.11.1 Number of stories. Structures of conventional light-frame construction and assigned to Seismic Design Category C shall not exceed two stories above grade plane.

2308.12.1 Number of stories. Structures of conventional light-frame construction and assigned to Seismic Design Category D or E shall not exceed one story above grade plane.</td></tr>
<tr><td>2308.2.2 Allowable floor-to-floor height</td><td>Moved from 2308.2, item 2</td></tr>
<tr><td>2308.2.3 Allowable Loads</td><td>Moved from 2308.2, item 3</td></tr>
<tr><td>2308.2.4 Allowable wind speed</td><td>Moved from 2308.2, item 4</td></tr>
<tr><td>2308,2,5 Allowable roof span</td><td>Moved from 2308.2, item 5</td></tr>
<tr><td>2308.2.6 Risk Category limitation</td><td>Moved from 2308.2, item 6. SDC “F” was deleted since the provisions of 2308 are not allowed in SDC F.</td></tr>
<tr><td>2308.2.8 Portions exceeding limitations of conventional light-frame construction</td><td>Moved from 2308.1.1 and unchanged. The last sentence was moved here from the last sentence of 2308.4.2. The rest of 2308.4.2 was redundant.</td></tr>
<tr><td>2308.3 Foundations and footings. Foundations and footings shall be as specified in Chapter 18.</td><td>Moved from 2308.6</td></tr>
<tr><td>2308.3.1 Foundation plates or sills</td><td>Moved from 2308.12.9</td></tr>
<tr><td>2308.3.2 Sill plate anchorage in Seismic Design Category D and E.</td><td>2308.12.8 Sill plate anchorage</td></tr>
<tr><td>2308.4 Floor framing</td><td></td></tr>
<tr><td>2308.4.1 Girders</td><td>Moved from 2308.7</td></tr>
<tr><td>2308.4.2 Floor joists</td><td></td></tr>
<tr><td>2308.4.2.1 Span</td><td>Moved from 2308.8</td></tr>
</table>

2308.4.2.2 Bearing	Moved from 2308.8.1. Switched first sentence to end of paragraph
2308.4.2.3 Framing details	Moved from 2308.8.2. Notches portion removed and placed in section 2308.4.2.4
2308.4.2.4 Notches and holes	Moved from 2308.8.2
2308.4.3 Engineered wood products	Moved from 2308.8.2.1. First sentence is new.
2308.4.4 Framing around openings	Moved from 2308.8.3
2308.4.4.1 Openings in horizontal diaphragms in SDC B, C, D and E	From 2308.11.3.3 The text of this section has been re-arranged for clarity. The first sentence states that a tie and blocking are required. Then, the tie is described followed by the blocking.
2308.4.5 Joists supporting bearing partitions	Moved from 2308.8.4
2308.4.6 Lateral support	Moved from 2308.8.5. Changed "Floor, attic and roof…." to "Floor and ceiling…"
2308.4.7 Structural floor sheathing	Moved from 2308.8.6
2308.4.8 Under-floor ventilation	Moved from 2308.8.7
2308.4.9 Floor framing supporting braced wall panels	Reference to existing requirements from 2308.12.6 that have been moved to 2308.6.7
2308.4.10 Anchorage of exterior means of egress components in Seismic Design Category D or E	Moved from 2308.12.7
2308.5 Wall Construction	
2308.5.1 Stud size, height and spacing	Moved from 2308.9.1.
2308.5.2 Framing details	Moved from 2308.9.2 Exception #1 from 2308.9.2.3 Exception #2 from 2308.9.2
Table 2308.5.1	From existing Table 2308.9.1 Footnote "c" is from existing language in section 2308.9.1
2308.5.3 Plates and sills	
2308.5.3.1 Bottom plate or sill	From 2308.9.2.4
2308.5.3.2 Top plates	From 2308.9.2.1
2308.5.4 Nonbearing walls and partitions	From 2308.9.2.3
2308.5.5 Openings in walls and partitions	From 2308.9.5.
2308.5.5.1 Openings in exterior bearing walls "Wall studs shall support……"	From 2308.9.5.1 From 2308.9.5.2
2308.5.5.2 Openings in interior bearing partitions	From 2308.9.6
2308.5.5.2 Openings in interior nonbearing partitions	From 2308.9.7.
2308.5.6 Cripple walls	From 2308.9.4
2308.5.7 Bridging	From 2308.9.9
2308.5.8 Pipes in walls	From 2308.9.8
2308.5.9 Cutting and notching	From 2308.9.10
2308.5.10 Bored holes	From 2308.9.11
2308.6 Wall bracing	
2308.6.1 Braced wall line spacing Refers to new Table 2308.6.1 that contains spacing information from:	 BWL at 35' o.c. from 2308.3.1 BWL in SDC D/E at 25' o.c. from 2308.12.3
2308.6.2 Location of braced panels	From 2308.9.3. Distance of panel from end of wall line (12 ½ feet) was moved to Table 2308.6.1 along with SDC D and E limitation of 8 feet from 2308.12.4
2308.6.3 Braced wall panel methods New Table 2308.6.3(1)	From 2308.9.3. items 1 through 8 are re-located into Table 2308.6.3.(1) and renamed;

	1 LIB Let In Bracing 2 DWB Diagonal Wood Boards 3 WSP Wood Structural Panels 4 SFB Structural Fiberboard Sheathing 5 GB Gypsum Board 6 PBS Particle Board Sheathing 7 PCP Portland Cement Plaster 8 HPS Hardboard Panel Siding The two "Alternative bracing" options from 2308.9.3.1 are incorporated into Table 2308.6.3(1) as items 9 and 10 9 Alt bracing from 2308.9.3.1 ABW (Alternate Braced Wall) 10 Alt bracing wall panel adjacent to a door or window opening PFH (Portal Frame w/ Hold-downs)
2308.6.4 Length of braced wall panels	From 2308.9.3
2308.6.5 Alternative bracing	From 2308.9.3.1
2308.6.5.1 Alternate Braced Wall (ABW)	From 2308.9.3.1
2308.6.5.2 Portal Frame w/ Hold-downs (PFH)	From 2308.9.3.2 "Alternate bracing wall panel adjacent to a door or window opening"
2308.6.6 Cripple wall bracing	From 2308.9.4.1
2308.6.6.1 Cripple wall bracing in Seismic Design Category A, B and C	From 2308.9.4.1 and 2308.9.4.2
2308.6.6.2 Cripple wall bracing in Seismic Design Category D and E	From 2308.12.4
2308.6.7 Connections of braced wall panels	From 2308.12.4
2308.6.6.1 Bottom plate connection	From 2308.3.2.1
2308.6.6.2 Top plate connection	From 2308.3.2.2
2308.6.6.3 Sill anchorage	From first portion of 2308.3.3. The remainder of 2308.3.3 is moved to 2308.3.1 "Foundation Plates and Sills"
2308.6.6.4 Anchorage to all-wood foundations	From 2308.3.3.1
2308.6.7 Braced wall line support	
2308.6.7.1 Foundation requirements Cantilever floor provisions Braced panel over beam below	From 2308.3.4 From 2308.12.6, Item 1 (re-worded) From 2308.12.6, Item 3 (re-worded and shown in Fig. 2308.6(1)
2308.6.7.2 Floor and roof diaphragm support in Seismic Design Category D and E	From 2308.12.6, item 2
2308.6.7.3 Stepped footings in SDC B,C,D and E	From 2308.11.3.2
2308.6.8 Attachment of sheathing	From 2308.12.5
2308.6.9 Limitation of concrete or masonry veneer	
2308.6.9.1 Concrete or masonry veneer in Seismic Design Category B and C	From 2308.11.2
2308.6.9.2 Concrete or masonry veneer in Seismic Design Category D and E	From 2308.12.2
2308.7 Roof and ceiling framing	From 2308.10. Figure 2308.7 is new and is similar to the Figure in the IRC
2308.7.1 Ceiling joist spans	From 2308.10.2
2308.7.2 Rafter spans	From 2308.10.3
2308.7.3 Ceiling joist and rafter framing	From 2308.10.4
2308.7.3 Ceiling joist and rafter connections	From 2308.10.4
2308.7.4 Notches and holes	From 2308.10.4.2
2308.7.5 Wind uplift	From 2308.10.1
2308.7.6 Framing around openings	From 2308.10.4.3

2308.7.6 Openings in roof diaphragms in SDC B, C, D and E	From 2308.11.3.3 The text of this section has been re-arranged for clarity. The first sentence states that a tie and blocking are required. Then, the tie is described followed by the blocking.
2308.7.7 Purlins	From 2308.10.5
2308.7.9 Engineered wood products	From 2308.10.7
2308.7.10 Roof sheathing	From 2308.10.8
2308.7.11 Joints	From 2308.10.8.1
2308.7.12 Roof planking	From 2308.10.9
2308.7.13 Trusses	From 2308.10.10
2308.7.14 Attic ventilation	From 2308.10.11

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

2308-S-RICE.doc

Public Hearing Results

Committee Action: Disapproved

Committee Reason: The committee feels this is a good reorganization of convention construction requirements, but with the number of editorials issues this disapproval will assure that they get done. Proponent is encouraged to work with FEMA and AWC on a public comment.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment 1:

Chuck Bajnai, Chesterfield County, VA, representing ICC Building Code Action Committee, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

~~2308.2.7~~ **2308.1.1 Portions exceeding limitations of conventional light-frame construction.** When portions of a building of otherwise conventional light-frame construction exceed the limits of Section 2308.2, those portions and the supporting load path shall be designed in accordance with accepted engineering practice and the provisions of this code. For the purposes of this section, the term "portions" shall mean parts of buildings containing volume and area such as a room or a series of rooms. The extent of such design need only demonstrate compliance of the non-conventionally light-framed elements with other applicable provisions of this code and shall be compatible with the performance of the conventional light-framed system.

2308.1.2 Connections and fasteners. Connectors and fasteners used in conventional construction shall comply with the requirements of Section 2304.9.

2308.2.3 Allowable Loads. Loads shall be in accordance with Chapter 16 and shall not exceed the following:

1. Average dead loads shall not exceed 15 psf (718 N/m^2) for combined roof and ceiling, exterior walls, floors and partitions.

   Exceptions:

   1. Subject to the limitations of Section ~~2308.6.10.2~~ 2308.6.9.2, stone or masonry veneer up to the lesser of 5 inches (127 mm) thick or 50 psf (2395 N/m^2) and installed in accordance with Chapter 14 is permitted to a height of 30 feet (9144 mm) above a noncombustible foundation, with an additional 8 feet (2438 mm) permitted for gable ends.
   2. Concrete or masonry fireplaces, heaters and chimneys shall be permitted in accordance with the provisions of this code.

2. Live loads shall not exceed 40 psf (1916 N/m^2) for floors.
3. Ground snow loads shall not exceed 50 psf (2395 N/m^2).

2308.3.1 Foundation plates or sills: Foundation plates or sills resting on concrete or masonry foundations shall comply with Section 2304.3.1. Foundation plates or sills shall be bolted or anchored to the foundation with not less than ½-inch-diameter 912.7 mm) steel bolts or approved anchors spaced to provide equivalent anchorage as the steel bolts. ~~Along braced wall lines in structures assigned to Seismic Design Category E, steel bolts with a minimum nominal diameter of 5/8 inch (15.9 mm) or approved anchor straps load rated in accordance with Section 1706.1 and spaced to provide equivalent anchorage shall be used.~~ Bolts shall be embedded at least 7 inches (178 mm) into concrete or masonry. Bolts shall be spaced not more than 6 feet (1829 mm) apart and there shall be a minimum of two bolts or anchor straps per piece with one bolt or anchor strap located not more than 12 inches (305 mm) or less than 4 inches (102 mm) from each end of each piece. ~~Bolts in braced wall lines in structures over two stories above grade shall be spaced not more the 4 feet (1219 mm) o.c..~~ A properly sized nut and washer shall be tightened on each bolt to the plate.

<u>Exceptions:</u>

1. <u>Along braced wall lines in structures assigned to Seismic Design Category E, steel bolts with a minimum nominal diameter of 5/8 inch (15.9 mm) or approved anchor straps load rated in accordance with Section 1711.1 and spaced to provide equivalent anchorage shall be used.</u>
2. <u>Bolts in braced wall lines in structures over two stories above grade shall be spaced not more than 4 feet (1219 mm) o.c.</u>

2308.4.4.1 Openings in floor diaphragms in Seismic Design Categories B, C, D, and E: Openings in horizontal diaphragms <u>in Seismic Design Categories B, C, D and E</u> with a dimension perpendicular to the joist that is greater than 4 feet (1219 mm) shall be constructed with metal ties and blocking in accordance with this section and Figure 2308.4.4.1(1). Metal ties shall not be less than 0.058 inch [1.47 mm (16 galvanized gage)] thick by 1-1/2 inches (38 mm) wide with a minimum yield stress of 33,000 psi (227 Mpa). Blocking shall be provided 2 feet minimum beyond headers. Ties shall be attached to blocking with eight 16d common nails on each side of the header-joist intersection.

Openings in floor diaphragms in Seismic Design Categories D and E ~~shall not exceed a dimension greater than 50 percent~~ <u>shall not have any dimension exceeding 50 percent</u> of the distance between braced wall lines or an area greater than 25 percent of the area between orthogonal pairs of braced wall lines [see Figure 2308.4.4.1(2)], or <u>the portion of the structure containing the opening</u> shall be designed in accordance with accepted engineering practice <u>to resist the forces specified in Chapter 16, to the extent such irregular opening affects the performance of the conventional framing system.</u>

2308.4.4.2 Vertical offsets in floor diaphragms in Seismic Design Categories D and E: <u>In Seismic Design Categories D and E,</u> portions of a floor level shall not be vertically offset such that the framing members on either side of the offset cannot be lapped or tied together in an approved manner in accordance with Figure 2308.4.4.2 <u>unless the portion of the structure containing the irregular offset is designed in accordance with accepted engineering practice.</u>

> **<u>Exception:</u>** <u>Framing supported directly by foundations need not be lapped or tied directly together.</u>

2308.5.3.2 Top plates: <u>Bearing and exterior wall</u> studs shall be capped with double top plates installed to provide overlapping at corners and at intersections with other partitions. End joints in double top plates shall be offset at least 48 inches (1219 mm), and shall be nailed in accordance with Table 2304.9.1. Plates shall be a nominal 2 inches (51 mm) in depth and have a width at least equal to the width of the studs.

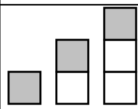
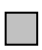
> **Exception:** A single top plate is permitted, provided the plate is adequately tied at joints, corners and intersecting walls by at least the equivalent of 3-inch by 6-inch (76 mm by 152 mm) by 0.036-inch-thick (0.914 mm) galvanized steel connector that is nailed to each wall or segment of wall by six 8d nails or equivalent, provided the rafters, joists or trusses are centered over the studs with a tolerance of not more than 1 inch (25 mm).

Where bearing studs are spaced at 24-inch (610 mm) intervals and top plates are less than two 2- inch by 6-inch (51 mm by 152 mm) or two 3-inch by 4- inch (76 mm by 102 mm) members and where the floor joists, floor trusses or roof trusses that they support are spaced at more than 16-inch (406 mm) intervals, such joists or trusses shall bear within 5 inches (127 mm) of the studs beneath or a third plate shall be installed.

2308.5.6 Cripple walls. *(No change to first two sentences.)* See Section ~~2308.6.5~~ <u>2308.6.6</u> for cripple wall bracing.

2308.6.1 Braced wall lines: For the purpose of determining the amount and location of bracing required along each story of a building, braced wall lines shall be designated as straight lines through the building plan in both the longitudinal and transverse direction and placed in accordance with Table 2308.6.1 and Figure ~~2308.6.1~~ <u>2308.6(1).</u> *(no change to the rest of the section)*

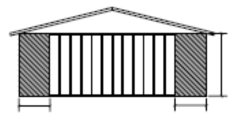
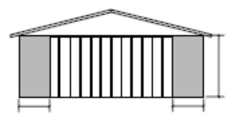
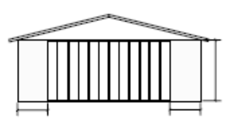
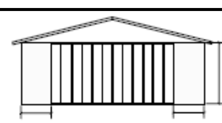
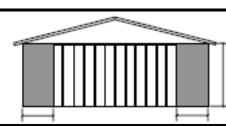
Table ~~2308.1~~ 2308.6.1
WALL BRACING REQUIREMENTS

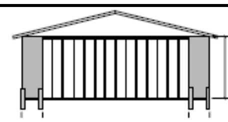
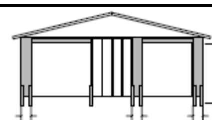
Seismic Design Category	Story Condition (See section 2308.2)	Maximum spacing of braced wall lines	Braced panel location, spacing (o.c.) and minimum percentage (x)			Maximum distance of braced wall panels from each end of braced wall line
			Bracing Method			
			LIB	DWB WSP	SFB PBS PCP HPS GB, ~~c~~,d	
A and B		35'-0"	Each end and ≤25'-0" o.c.	Each end and ≤25'-0" o.c.	Each end and ≤25'-0" o.c.	12'-6"
		35'-0"	Each end and ≤25'-0" o.c.	Each end and ≤25'-0" o.c.	Each end and ≤25'-0" o.c.	12'-6"
		35'-0"	NP	Each end and ≤25'-0" o.c.	Each end and ≤25'-0" o.c.[c]	12'-6"
C		35'-0"	NP	Each end and ≤25'-0" o.c.	Each end and ≤25'-0" o.c.	12'-6"
		35'-0"	NP	Each end and ≤25'-0" o.c. (min 25% of wall length) [e]	Each end and ≤25'-0" o.c. (min 25% of wall length) [e,c]	12'-6"
D and E		25'-0"	NP	Sds < 0.50: Each end and ≤25'-0" o.c. (min 21% of wall length) [e]	Sds < 0.50: Each end and ≤25'-0" o.c. (min 43% of wall length) [e]	8'-0"
				0.5 ≤ Sds < 0.75: Each end and ≤25'-0" o.c. (min 32% of wall length) [e]	0.5 ≤ Sds < 0.75: Each end and ≤25'-0" o.c. (min 59% of wall length) [e]	
				0.75 ≤Sds ≤ 1.00: Each end and ≤25'-0" o.c. (min 37% of wall length) [e]	0.75 ≤Sds ≤ 1.00: Each end and ≤25'-0" o.c. (min 75% of wall length) [e]	
				Sds > 1.00: Each end and ≤25'-0" o.c. (min 48% of wall length) [e]	Sds > 1.00: Each end and ≤25'-0" o.c. (min 100% of wall length) [e]	

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm.
NP = Not Permitted

a. This table specifies minimum requirements for *braced wall panels* along interior or exterior *braced wall lines*.
b. See Section 2308.6.2 for full description of bracing methods.
c. For method GB, gypsum wallboard applied to framing supports that are spaced at 16 inches on center.
d. The required lengths shall be doubled for gypsum board applied to only one face of a braced wall panel.
e. Percentage shown represents the minimum amount of bracing required along the building length (or wall length if the structure has an irregular shape)

TABLE 2308.6.3(1)
BRACING METHODS

METHODS, MATERIAL	MINIMUM THICKNESS	FIGURE	CONNECTION CRITERIA [a]	
			Fasteners	**Spacing**
LIB [a] Let-in-bracing	1x4 wood or approved metal straps attached at 45° to 60° angles to studs at maximum of 16” o.c.		Per Fastener Table 2304.9.1~~, item 20~~	Wood: per stud plus top and bottom plates
			Metal strap: installed per manufacturer’s installation recommendations	Metal strap: installed per manufacturer’s installation recommendations
DWB Diagonal wood boards	$^3/_4$" thick (1” nominal) x 6” minimum width to studs at maximum of 24” o.c.		Per Fastener Table 2304.9.1~~, item 21 or 22~~	Per stud
WSP Wood structural panel	$^3/_8$” Per TABLE 2308.6.3(2) or 2308.6.3(3)		Per Fastener Table 2304.9.1~~, item 31~~	6" edges 12" field
SFB Structural fiberboard sheathing	$^1/_2$" Per TABLE ~~2308.6.3(4)~~ 2304.9.1 to studs at maximum 16" o.c.		Per Fastener Table 2304.9.1~~, item 33~~	3" edges 6" field
GB Gypsum board (Double sided)	$^1/_2$" by a minimum of 4 feet wide to studs at maximum of 24” o.c.		Exterior and interior sheathing: with 5d cooler nails (1-5/8” x 0.086”) or 1¼” screws (type W or S) for ½” gypsum board or 1$^5/_8$” screws (type W or S) for $^5/_8$” gypsum board.	For all braced wall panel locations: 7" o.c. along panel edges (including top and bottom plates) and 7" o.c.in the field
PBS Particle-board sheathing	$^3/_8$" or $^1/_2$" per Table ~~2308.9.3(4)~~ 2308.6.3(4) to studs at maximum of 16” o.c.		6d common (2” long x0.113" dia.) nails for $^3/_8$” thick sheathing or 8d common (2½" long x 0.131" dia.) nails for $^1/_2$” thick sheathing	3" edges 6" field
PCP Portland cement plaster	See Section 2510 to studs at maximum of 16” o.c.		1½" long, 11 gage, $^7/_{16}$" dia. head nails or $^7/_8$" long, 16 gage staples	6" o.c. on all framing members
HPS Hardboard panel siding	$^7/_{16}$" TABLE 2308.6.3(5)		Per Fastener Table ~~2308.9.1~~ 2304.9.1	4" edges 8" field

METHODS, MATERIAL	MINIMUM THICKNESS	FIGURE	CONNECTION CRITERIA [a]	
ABW Alternate braced wall.	$^3/_8$"		See Figure 2308.6.5(1) and Section 2308.6.5.1	See Figure ~~2308.6.3(1)~~ 2308.6.5(1)
PFH Portal frame with hold-downs	$^3/_8$		See Figure 2308.6.5(2) and Section 2308.6.5.2	See Figure ~~2308.6.3(2)~~ 2308.6.5(2)

For SI: 1 foot 305 mm

a. Method LIB shall have gypsum board fastened to at least one side with nails or screws.

TABLE 2308.6.3(3)
WOOD STRUCTURAL PANEL WALL SHEATHING

(No change to table)

a. *(no change)*
b. Blocking of horizontal joints shall not be required except as specified in Sections ~~2306.3 and 2308.12.4~~ 2308.6.4.

TABLE 2308.6.3(5)
HARDBOARD SIDING

(No change to table)

a. *(no changes)*
b. *(no changes)*
c. Where used to comply with Section ~~2308.9.3~~ 2308.6.
d. *(no changes)*

2308.6.4 Length of braced wall panels. For Methods DWB, WSP, SFB, PBS, PCP and HPS each panel must be at least 48 inches (1219 mm) in length, covering three stud spaces where studs are spaced 16 inches (406 mm) apart and covering two stud spaces where studs are spaced 24 inches (610 mm) apart. *Braced wall panels* less than the required 48" length shall not contribute towards the amount of bracing required. *Braced wall panels* longer than the required length shall be credited for their actual length. For Method GB, each panel must be at least 96 inches (2438 mm) in length where applied to one side of the studs or 48 inches (1219 mm) where applied to both sides.

All vertical joints of panel sheathing shall occur over studs and adjacent panel joints shall be nailed to common framing members. Horizontal joints shall occur over blocking or other framing equal in size to the studding except where waived by the installation requirements for the specific sheathing materials. Sole plates shall be nailed to the floor framing in accordance with Section ~~2308.3.2~~ 2308.6.7 and top plates shall be connected to the framing above in accordance with Section 2308.5.3. Where joists are perpendicular to braced wall lines above, blocking shall be provided under and in line with the braced wall panels.

2308.6.5.2 Portal Fame with Hold-downs (PFH). A PFH shall be constructed in accordance with this section and Figure 2308.6.5.2. The adjacent door or window opening shall have a full-length header.

In one-story buildings, each panel shall have a length of not less than 16 inches (406 mm) and a height of not more than 10 feet (3048 mm). Each panel shall be sheathed on one face with a single layer of 3/8 inch (9.5 mm) minimum thickness wood structural panel sheathing nailed with 8d common or galvanized box nails in accordance with Figure 2308.6.5.2. The wood structural panel sheathing shall extend up over the solid sawn or glued-laminated header and shall be nailed in accordance with Figure ~~2308.6.5~~ 2308.6.5.2. A built-up header consisting of at least two 2 × 12s and fastened in accordance with Item 24 of Table 2304.9.1 shall be permitted to be used. A spacer, if used, shall be placed on the side of the built-up beam opposite the wood structural panel sheathing. The header shall extend between the inside faces of the first full-length outer studs of each panel. The clear span of the header between the inner studs of each panel shall be not less than 6 feet (1829 mm) and not more than 18 feet (5486 mm) in length. A strap with an uplift capacity of not less than 1,000 pounds (4,400 N) shall fasten the header to the inner studs opposite the sheathing. One anchor bolt not less than 5/8 inch (15.9 mm) diameter and installed in accordance with Section 2308.3.1 shall be provided in the center of each sill plate. The studs at each end of the panel shall have a hold-down device fastened to the foundation with an uplift capacity of not less than 4,200 pounds (18 480 N).

Where a panel is located on one side of the opening, the header shall extend between the inside face of the first full-length stud of the panel and the bearing studs at the other end of the opening. A strap with an uplift capacity of not less than 1,000 pounds (4400 N) shall fasten the header to the bearing studs. The bearing studs shall also have a hold-down device fastened to the foundation with an uplift capacity of not less than 1,000 pounds (4400 N). The hold-down devices shall be an embedded strap type, installed in accordance with the manufacturer's recommendations. The PFH panels shall be supported directly on a foundation that is continuous across the entire length of the braced wall line. This foundation shall be reinforced with not less than one No. 4 bar top and bottom. Where the continuous foundation is required to have a depth greater than 12 inches (305 mm), a minimum 12-inch by 12-inch (305 mm by 305 mm) continuous footing or turned down slab edge is permitted at door openings in the braced wall line. This continuous footing or turned down slab edge shall be reinforced with not less than one No. 4 bar top and bottom. This reinforcement shall be lapped not less than 15 inches (381 mm) with the reinforcement required in the continuous foundation located directly under the braced wall line.

When a PFH is installed at the first *story* of two-story buildings, each panel shall have a length of not less than 24 inches (610 mm).

2308.6.5.1 Cripple wall bracing in Seismic Design Category A, B and C. For the purposes of this section, cripple walls in Seismic Design Categories A, B, and C having a stud height exceeding 14 inches (356 mm) shall be considered a *story* and shall be braced in accordance with Table 2308.6(1). Spacing of edge nailing for required cripple wall bracing shall not exceed 6 inches (152mm) o.c. along the foundation plate and the top plate of the cripple wall. Nail size, nail spacing for field nailing and more restrictive boundary nailing requirements shall be as required elsewhere in the code for the specific bracing material used.

2308.6.5.2 Cripple wall bracing in Seismic Design Category D and E. For the purposes of this section, cripple walls in Seismic Design Category D and E having a stud height exceeding 14 inches (356 mm) shall be considered a story and shall be braced in accordance with Table ~~2308.6(1)~~ 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)~~ 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) o.c.

2308.6.6.1 Bottom plate connection. Braced wall line bottom plates shall be connected to joists or full-depth blocking below in accordance with Table 2304.9.1, ~~Item 6~~ or to foundations in accordance with Section ~~2308.3.3~~ 2308.3.1.

2308.6.6.2 Top plate connection. Where joists and/or rafters are used, braced wall line top plates shall be fastened over the full length of the braced wall line to joists, rafters, rim boards or blocking above in accordance with Table 2304.9.1, as applicable, based on the orientation of the joists or rafters to the braced wall line. Blocking at joists with walls above shall be equal to the depth of the joist at the braced wall line. Blocking at rafters need not be full depth but shall extend to within 2 inches (51 mm) from the roof sheathing above. Blocking shall be a minimum of 2 inches (51 mm) nominal thickness and shall be fastened to the braced wall line top plate as specified in Table 2304.9.1~~, Item 11~~. Notching or drilling of holes in blocking in accordance with the requirements of Section ~~2308.8.2~~ 2308.4.2.4 or Section ~~2308.10.4.2~~ 2308.7.4 shall be permitted.

At exterior gable end walls braced wall panel sheathing in the top *story* shall be extended and fastened to roof framing where the spacing between parallel exterior braced wall lines is greater than 50 feet (15 240 mm).

Where roof trusses are used and are installed perpendicular to an exterior braced wall line, lateral forces shall be transferred from the roof diaphragm to the braced wall over the full length of the braced wall line by blocking of the ends of the trusses or by other *approved* methods providing equivalent lateral force transfer. Blocking shall be minimum 2 inches (51 mm) nominal thickness and shall extend to within 2 inches (51 mm) from the roof sheathing above and shall be fastened to the braced wall line top plate as specified in Table 2304.9.1. Notching or drilling of holes in blocking in accordance with the requirements of Section 2308.4.2.4 or Section 2308.7.4 shall be permitted.

2308.6.7.1 Foundation requirements. *(no change to first sentence and Exception.)* For structures in Seismic Design Category D and E, exterior braced wall panels shall be in the same plane vertically with the foundation or the ~~braced wall line~~ portion of the structure containing the offset shall be designed in accordance with accepted engineering practice according to section 2308.1.1

For structures in *Seismic Design Category D and E,* exterior *braced wall panels* shall be in the same plane vertically with the foundation or the ~~braced wall line~~ portion of the structure containing the offset shall be designed in accordance with accepted engineering practice according to section 2308.1.1

Exceptions:

1. Exterior *braced wall panels* may be located up to 4 feet from the foundation below when supported by a floor constructed in accordance with all the following:
   - 1.1 Cantilevers or setbacks shall not exceed four times the nominal depth of the floor joists
   - 1.2. Floor joists shall be 2 inches by 10 inches (51 mm by 254 mm) or larger and spaced not more than 16 inches (406 mm) o.c.
   - 1.3. The ratio of the back span to the cantilever shall be at least 2:1.
   - 1.4. Floor joists at ends of *braced wall panels* shall be doubled.
   - 1.5. A continuous rim joist shall be connected to the ends of cantilevered joists. The rim joist is permitted to be spliced using a metal tie not less than 0.058 inch (1.47 mm) (16 galvanized gage) and 11/2 inches (38 mm) wide fastened with six 16d common nails on each side. The metal tie shall have a minimum yield stress of 33,000 psi (227 MPa).
   - 1.6. Joists at setbacks or the end of cantilevered joists shall not carry gravity loads from more than a single *story* having uniform wall and roof loads, nor carry the reactions from headers having a span of 8 feet (2438 mm) or more.
2. The end of a required braced wall panel shall be allowed to extend not more than 1 foot (305 mm) over an opening in the wall below. This requirement is applicable to *braced wall panels* offset in plane and to *braced wall panels* offset out of plane as permitted by the exception to Item 1 above in this section.

Exception: *Braced wall panels* are permitted to extend over an opening not more than 8 feet (2438 mm) in width where the header is a 4-inch by 12-inch (102 mm by 305 mm) or larger member

2308.6.7.2 Floor and roof diaphragms support in Seismic Design Category D and E. In structures assigned to *Seismic Design Category D or E,* floor and roof diaphragms shall be laterally supported by braced wall lines on all edges and connected in accordance with Section ~~2308.3.2~~ 2308.6.7 [see Figure 2308.6.7.2(1)].

> **Exception:** Portions of roofs or floors that do not support *braced wall panels* above are permitted to extend up to 6 feet (1829 mm) beyond a braced wall line [see Figure 2308.6.7.2(2)] provided that the framing members are connected to the braced wall line below in accordance with Section 2308.6.6.

2308.6.7.3 Stepped footings in Seismic Design Category B, C, D, and E. In Seismic Design Category B, C, D, and E, where the height of a required braced wall panel extending from foundation to floor above varies more than 4 feet (1219 mm), the following construction shall be used:

1. Where the bottom of the footing is stepped and the lowest floor framing rests directly on a sill bolted to the footings, the sill shall be anchored as required in Section ~~2308.3.3~~ 2308.3.
2. Where the lowest floor framing rests directly on a sill bolted to a footing not less than 8 feet (2438 mm) in length along a line of bracing, the line shall be considered to be braced. The double plate of the cripple stud wall beyond the segment of footing extending to the lowest framed floor shall be spliced to the sill plate with metal ties, one on each side of the sill and plate. The metal ties shall not be less than 0.058 inch [1.47 mm (16 galvanized gage)] by 11/2 inches (38 mm) wide by 48 inches (1219 mm) with eight 16d common nails on each side of the splice location (see Figure 2308.6.7.3(1). The metal tie shall have a minimum yield stress of 33,000 pounds per square inch (psi) (227 MPa).
3. Where cripple walls occur between the top of the footing and the lowest floor framing, the bracing requirements for a *story* shall apply.

2308.6.8 Attachment of sheathing. Fastening of braced wall panel sheathing shall not be less than that prescribed in Tables ~~2308.6(1)~~ 2308.6.1 and 2304.9.1. Wall sheathing shall not be attached to framing members by adhesives.

2308.6.9.1 Limitations of concrete and masonry veneer in Seismic Design Categories B or C. In Seismic Design Categories B and C, concrete or masonry walls and stone or masonry veneer shall not extend above a basement.

Exceptions:

1. In structures assigned to *Seismic Design Category* B, stone and masonry veneer is permitted to be used in the first two stories above grade plane or the first three stories above grade plane where the lowest story has concrete or masonry walls, provided that structural use panel wall bracing is used and the length of bracing provided is one and one-half times the required length as determined in Table ~~2308.9.3(1)~~ 2308.6.1.

2. Stone and masonry veneer is permitted to be used in the first story above grade plane or the first two stories above grade plane where the lowest story has concrete or masonry walls.

3. Stone and masonry veneer is permitted to be used in both stories of buildings with two stories above grade plane, provided the following criteria are met:

    3.1. Type of brace per Section ~~2308.9.3~~ 2308.6.1 shall be WSP and the allowable shear capacity in accordance with Section 2306.3 shall be a minimum of 350 plf (5108 N/m).

    3.2. *Braced wall panels* in the second *story* shall be located in accordance with Section ~~2308.9.3~~ 2308.6.1 and not more than 25 feet (7620 mm) on center, and the total length of braced wall panels shall be not less than 25 percent of the braced wall line length. *Braced wall panels* in the first *story* shall be located in accordance with Section ~~2308.9.3~~ 2308.6.1 and not more than 25 feet (7620 mm) on center, and the total length of *braced wall panels* shall be not less than 45 percent of the braced wall line length.
    3.3. Hold-down connectors shall be provided at the ends of each *braced wall panel* for the second *story* to first *story* connection with an allowable capacity of 2,000 pounds (8896 N). Hold-down connectors shall be provided at the ends of each *braced wall panel* for the first story to foundation connection with an allowable capacity of 3,900 pounds (17 347 N). In all cases, the hold-down connector force shall be transferred to the foundation.
    3.4. Cripple walls shall not be permitted.

2308.6.9.2 Limitations of concrete or masonry in Seismic Design Categories D and E: In Seismic Design Categories D and E, concrete or masonry walls and stone or masonry veneer shall not extend above a basement.

> **Exception**: In structures assigned to *Seismic Design Category* D, stone and masonry veneer is permitted to be used in the first story above grade plane, provided the following criteria are met:

1. Type of brace in accordance with Section ~~2308.9.3~~ 2308.6.1 shall be WSP and the allowable shear capacity in accordance with Section 2306.3 shall be a minimum of 350 plf (5108 N/m).
2. The bracing of the first *story* shall be located at each end and at least every 25 feet (7620 mm) o.c. but not less than 45 percent of the braced wall line.
3. Hold-down connectors shall be provided at the ends of braced walls for the first floor to foundation with an allowable capacity of 2,100 pounds (9341 N).

4. Cripple walls shall not be permitted.

2308.7.8 Blocking. Roof rafters and ceiling joists shall be supported laterally to prevent rotation and lateral displacement in accordance with the provisions of Section ~~2308.8.5~~ 2308.4.6 and connected to braced wall lines per Section 2308.6.6.2.

2308.7.12 Roof planking. Planking shall be designed in accordance with the general provisions of this code.

In lieu of such design, 2-inch (51 mm) tongue-and groove planking is permitted in accordance with Table ~~2308.10.9~~ 2308.7.12. Joints in such planking are permitted to be randomly spaced, provided the system is applied to not less than three continuous spans, planks are center matched and end matched or splined, each plank bears on at least one support, and joints are separated by at least 24 inches (610 mm) in adjacent pieces.

TABLE ~~2308.10.9~~ 2308.7.12
ALLOWABLE SPANS FOR 2-INCH TONGUE-AND-GROOVE DECKING

(No change to table contents)

2308.8 Design of elements. Combining of engineered elements or systems and conventionally specified elements or systems shall be permitted subject to the following limits.

2308.8.1 Elements exceeding limitations of conventional construction. When a building of otherwise conventional construction contains structural elements exceeding the limits of Section 2308.2, these elements and the supporting load path shall be designed in accordance with accepted engineering practice and the provisions of this code.

2308.8.2 Structural elements or systems not described herein. When a building of otherwise conventional construction contains structural elements or systems not described in Section 2308, these elements or systems shall be designed in accordance with accepted engineering practice and the provisions of this code. The extent of such design need only demonstrate compliance of the nonconventional elements with other applicable provisions of this code and shall be compatible with the performance of the conventionally framed system.

(Portions of proposal not shown remain unchanged)

Commenter's Reason: This public comment is submitted by the ICC Building Code Action Committee (BCAC). The BCAC was established by the ICC Board of Directors to pursue opportunities to improve and enhance an assigned International Code or portion thereof. This includes both the technical aspects of the codes as well as the code content in terms of scope and application of referenced standards. Since its inception in July, 2011, the BCAC has held 5 open meetings and numerous workgroup calls which included members of the BCAC as well as any interested party to discuss and debate the proposed changes and the public comments. Related documentation and reports are posted on the BCAC website at: http://www.iccsafe.org/cs/BCAC/Pages/default.aspx.

This proposed modification to S273 addresses concerns that were raised at the Code Development Hearings. The original proposal, S273, was the product of countless hours by the Building Code Action Committee members. The proposal was not completely free of editorial errors by the deadline for submission.

The committee strongly supported the proposal and felt that it was a good reorganization of section 2308 but that it needed corrections. At the time of the Code Development Hearings in May, most, if not all, of the concerns had been identified and preliminarily addressed. This modification, the work of the BCAC members, addresses the necessary corrections and editorial changes and is submitted with the full support of the BCAC and we request approval of this modification. Due to the overall length of the original code change, this Public Comment shows only the specific items that required correction.
The inconsistencies and mistakes in the original proposal that are cleaned up by this public comment, as explained below:

Section 2308.2.3: corrects an incorrect section reference.

Section 2308.1.1 (new): location as a subsection of 2308.1, as it is in the current code, is appropriate to avoid a circular reference to 2308.2 in the text, and the more prominent location in 2308.1 is appropriate.

Section 2308.1.2 (new): This requirement for connectors and fasteners was inadvertently omitted from the code change; it is inserted here.

Section 2308.3.1: relocates two sentences as exceptions for clear application of the rest of the requirements of the section.

2308.4.4.1: Since titles are editorial the seismic design category needs to be contained in the text of the section; and clarity of wording for floor opening limitations. It is not the opening that needs to be designed but the portion of the structure containing an opening that exceeds the limitations for size.

2308.4.4.2: The Seismic Design Categories need to be named in the text, and the alternative for design of the portion of the structure containing an irregular offset in the current code should be preserved.

2308.5.3.2: This clarification is necessary due to the reorganization of the sections.

2308.5.6: Corrects an incorrect reference.

2308.6.1: Corrects an incorrect Figure number reference.

Table 2308.1: Corrects an incorrect Table number from Table 2308.1 to 2308.6.1; the text has it correct. Also, corrects the footnote “c” placement and text to reflect the current application in current Table 2308.9.3(1).

Table 2308.6.3(1): eliminating reference to items numbers of Table 2304.9.1 in the “Connection Criteria” column will preclude cross-referencing difficulties if the fastener table is changed; the item numbers are not necessary for a correct reference. Several incorrect numbers references are corrected. Also, an existing 16-inch stud spacing limitation is inserted for the Structural Fiberboard method (SFB).

Table 2308.6.3(3): deletes a non-existent section reference and replaces the other with the correct section reference.

Table 2308.6.3(5): corrects an incorrect reference.

2308.6.4: corrects an incorrect section reference.

Section 2308.6.5.2: corrects an incorrect reference.

2308.6.5.1: the Seismic Design Categories need to be named in the text since titles are editorial.

2308.6.5.2: corrects an incorrect table number in two places.

2308.6.6.1: eliminates an unnecessary use of a table item number designation to preclude correlation problems later; also corrects an incorrect reference.

2308.6.6.2: eliminates an unnecessary use of a table item number designation to preclude correlation problems later; also corrects two incorrect references.

2308.6.7.1: reworded to make it clear that it is not the braced wall line that needs to be designed but the portion of the structure containing the offset which causes the structure to be “irregular” in regard to the limitations.

2308.6.7.2: corrects an incorrect section reference.

2308.6.7.3: the Seismic Design Categories need to be named in the text since titles are editorial; also corrects an incorrect section reference.

2308.6.8: corrects an incorrect table reference.

2308.6.9.1: the Seismic Design Categories need to be named in the text since titles are editorial; also corrects incorrect references in four places.

2308.6.9.2: the Seismic Design Categories need to be named in the text since titles are editorial; also corrects an incorrect reference.

2308.7.8: corrects an incorrect reference.

2308.7.12: coordinates the number of the referenced table with the section number.

Table 2308.7.12: coordinates the number of the table with the section that references it.

2308.8 (new): This section currently appears in 2012 IBC Section but was omitted from the proposal. It addresses individual engineered elements within the building and therefore differs from proposed 2308.1.1 which addresses entire portions of structures. These provisions are important for guidance regarding engineered elements and systems within a conventionally framed structure, and should be retained.

The following renumbering to be done by staff editorial.

~~2308.6.5~~ <u>2308.6.6</u> Cripple wall bracing.
~~2308.6.5.1~~ <u>2308.6.6.1</u> Cripple wall bracing in Seismic Design Category A, B and C.
~~2308.6.5.2~~ <u>2308.6.6.2</u> Cripple wall bracing in Seismic Design Category D and E.
~~2308.6.6~~ <u>2308.6.7</u> Connections of braced wall panels.
~~2308.6.6.1~~ <u>2308.6.7.1</u> Bottom plate connection.
~~2308.6.6.2~~ <u>2308.6.7.2</u> Top plate connection.
~~2308.6.6.3~~ <u>2308.6.7.3</u> Sill anchorage.
~~2308.6.6.4~~ <u>2308.6.7.4</u> Anchorage to all-wood foundations.
~~2308.6.7~~ <u>2308.6.8</u> Braced wall line and diaphragm support.
~~2308.6.7.1~~ <u>2308.6.8.1</u> Foundation requirements.

~~2308.6.7.2~~ 2308.6.8.2 Floor and roof diaphragm support in Seismic Design Category D and E.
Figure ~~2308.6.7.2(1)~~ 2308.6.8.2(1) ROOF IN SDC D OR E NOT SUPPORTED ON ALL EDGES
Figure ~~2308.6.7.2(2)~~ 2308.6.8.2(2) ROOF EXTENSION IN SDC D OR E BEYOND BRACED WALL LINE
~~2308.6.7.3~~ 2308.6.8.3 Stepped footings in Seismic Design Category B, C, D, and E.
Figure ~~2308.6.7.3(1)~~ 2308.6.8.3(1) STEPPED FOOTING CONNECTION DETAILS
~~2308.6.8~~ 2308.6.9 Attachment of sheathing.
~~2308.6.9~~ 2308.6.10 Limitations of concrete or masonry veneer.
~~2308.6.9.1~~ 2308.6.10.1 Limitations of concrete or masonry veneer in Seismic Design Categories B or C.
~~2308.6.9.2~~ 2308.6.10.2 Limitations of concrete or masonry in Seismic Design Categories D and E.

Corresponding update, to Section references are staff edits.

Public Comment 2:

Larry Wainright, Qualtim, representing Structural Building Components Association (SBCA), requests Approval as Submitted.

Commenters Reason: While the language in not perfect as described by the committee, the proposed changes are a vast improvement in the organization and clarity of this section. Over the past several years, SBCRI has conducted a great deal of research into the requirements of the IBC, section 2308 and the design capacity of wall assemblies built to those provisions. For engineered design, section 2306, references SDPWS. The design capacities in SDPWS are those obtained from E72 type tests with full restraint at the ends of the shearwall and a load beam at the top of the wall. Section 2308 requires neither of these conditions. Full scale testing using section 2308 provisions has shown that the capacity is significantly lower and there is reliance on systems effects to achieve the assumed shear strength. The Table below is an example what the assumed system effect is once all of the buildings construction details have been completed (i.e additional strength from the addition of interior partitions, windows and doors, corner framing, interior gypsum, etc.) It is only with this type of transparency that the provisions of section 2308 make any sense. We urge your support of the proposal as written to help provide clarity for all users of the code. This is a well thought out and reasonable rewrite.

Simplified Nominal Unit Shear Capacities for Braced Wall Lines.

Sheathing Material	Fastener	Fastener Spacing	Any Species Stud Framing		
			Tested capacity[1]	System Effects Factor	Nominal Unit Shear capacity for use in design.
3/8", 7/16" or 15/32" WSP @16" and 24" o.c framing.	6d (2" x 0.113" nails) or 8d (2 1/2 x 0.131"	6:12	350	1.80	630
3/8", 7/16" or 15/32" WSP @16" and 24" o.c framing (with 1/2" gypsum on interior face of wall.	6d (2" x 0.113") or 8d (2 1/2 x 0.131"nails and Types S or W drywall screws.	6:12 WSP & 16:16 for GWB	450	1.80	810
3/8", 7/16" or 15/32" WSP @16" and 24" o.c framing. - Seismic	6d (2" x 0.113" nails) or 8d (2 1/2 x 0.131"	6:12	330[2]	1.45[3]	475

[1]SBCRI full scale testing with anchor bolt restraint per section 2308 provisions.
[2]This value is based on a 5% reduction in tested capacities with Cyclic testing in accordance with research by Dolan, Toothman and Seaders.
[3]Factor to correlate SBCRI tested capacities with Anchor bolt restraint to SDPWS seismic values.

Full details of this research referenced above can be found at http://sbcri.info/bcters.php

Public Comment 3:

John Gruber, P.E., Sheppard Engineering, P.C., representing self, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

2308.1.1 Design Values. The provisions of section 2308 are based on the design values as shown in Table 2308.1.1.

Table 2308.1.1- Simplified Nominal Unit Shear Capacities for Braced Wall Lines.

Simplified Nominal Unit Shear Capacities for Braced Wall Lines.					
Sheathing Material	**Fastener**	**Fastener Spacing**	**Any Species Stud Framing**		
			Tested capacity[1]	**System Effects Factor**	**Nominal Unit Shear capacity for use in design.**
3/8", 7/16" or 15/32" WSP @16" and 24" o.c framing.	6d (2" x 0.113" nails) or 8d (2 1/2 x 0.131"	6:12	350	1.80	630
3/8", 7/16" or 15/32" WSP @16" and 24" o.c framing (with 1/2" gypsum on interior face of wall.	6d (2" x 0.113") or 8d (2 1/2 x 0.131"nails and Types S or W drywall screws.	6:12 WSP & 16:16 for GWB	450	1.80	810
3/8", 7/16" or 15/32" WSP @16" and 24" o.c framing. - Seismic	6d (2" x 0.113" nails) or 8d (2 1/2 x 0.131"	6:12	330[2]	1.45[3]	475

[1]SBCRI full scale testing with anchor bolt restraint per section 2308 provisions.
[2]This value is based on a 5% reduction in tested capacities with Cyclic testing in accordance with research by Dolan, Toothman and Seaders.
[3]Factor to correlate SBCRI tested capacities with Anchor bolt restraint to SDPWS seismic values.

(Portions of proposal not shown remain unchanged)

Commenters Reason: Over the past several years, SBCRI has conducted a great deal of research into the requirements of the IBC, section 2308 and the design capacity of wall assemblies built to those provisions. For engineered design, section 2306, references SDPWS. The design capacities in SDPWS are those obtained from E72 type tests with full restraint at the ends of the shearwall and a load beam at the top of the wall. Section 2308 requires neither of these conditions. Full scale testing using section 2308 provisions has shown that the capacity is significantly lower. Table 2308.1.1 simply adds transparency to this section to show what the assumed system effect is once all of the buildings construction detail has been completed (i.e. additional strength from the addition of interior partitions, windows and doors, corner framing, interior gypsum, etc.) Full details of this research can be found at http://sbcri.info/bcters.php

S273-12
Final Action: AS AM AMPC____ D

S276-12
2308.2

Proposed Change as Submitted

Proponent: Charles S. Bajnai, Chesterfield County (bajnaic@chesterfield.gov), VA, Ed Keith, American Plywood Association, representing Chesterfield County, VA, Robert Rice, OBOA, representing Chesterfield County, VA

Revise as follows:

2308.2 Limitations. Buildings are permitted to be constructed in accordance with the provisions of *conventional light-frame construction*, subject to the following limitations, and to further limitations of Sections 2308.11 and 2308.12.

3.1. Average dead loads shall not exceed 15 psf (718 N/m2) for combined roof and ceiling, exterior walls, floors and partitions.

Exceptions:

1. Subject to the limitations of Sections 2308.11.2 and 2308.12.2, stone or masonry veneer up to the lesser of 5 inches (127 mm) thick or 50 psf (2395 N/m^2) and installed in accordance with Chapter 14 is permitted to a height of 30 feet (9144 mm) above a noncombustible foundation, with an additional 8 feet (2438 mm) permitted for gable ends.
2. Concrete or masonry fireplaces, heaters and chimneys shall be permitted in accordance with the provisions of this code. 3.2. Live loads shall not exceed 40 psf (1916 N/m^2) for floors.

3.2. Live loads shall not exceed 40 psf (1916 N/m^2) for floors <u>of conventional light-frame construction</u>.

3.3. Ground snow loads shall not exceed 50 psf (2395 N/m^2).

(Portions of text not shown remain unchanged)

Reason: The limitation of 40 psf live load for floors from Table 1607.1 makes Section 2308, Conventional Light- Frame Construction, essentially restricted to residential construction.

This code change proposal is intended to clarify that the 40 psf live load for floors applies to all stories constructed of conventional light-frame construction.

This new exemption would allow Section 2308, Conventional Light-Frame Construction to apply to live/work structures, and one story offices, retail spaces, assembly spaces, schools, etc

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

2308.2-BAJNAI-RICE.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: This code change potentially extends the application of the conventional construction provisions to buildings not originally intended.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Chuck Bajnai, Chesterfield County, VA, representing ICC Building Code Action Committee requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

2308.2 Limitations. Buildings are permitted to be constructed in accordance with the provisions of *conventional light-frame construction*, subject to the following limitations, and to further limitations of Sections 2308.11 and 2308.12.

3.1. Average dead loads shall not exceed 15 psf (718 N/m2) for combined roof and ceiling, exterior walls, floors and partitions.

Exceptions:

1. Subject to the limitations of Sections 2308.11.2 and 2308.12.2, stone or masonry veneer up to the lesser of 5 inches (127 mm) thick or 50 psf (2395 N/m^2) and installed in accordance with Chapter 14 is permitted to a height of 30 feet (9144 mm) above a noncombustible foundation, with an additional 8 feet (2438 mm) permitted for gable ends.
2. Concrete or masonry fireplaces, heaters and chimneys shall be permitted in accordance with the provisions of this code. 3.2. Live loads shall not exceed 40 psf (1916 N/m^2) for floors.

3.2 Live loads shall not exceed 40 psf (1916 N/m^2) for floors of conventional light-frame construction. **.** <u>Floor live load shall be allowed to exceed 40 psf (1916 N/m2) where the floor is constructed on grade.</u>

3.3. Ground snow loads shall not exceed 50 psf (2395 N/m^2).

Commenter's Reason: This public comment is submitted by the ICC Building Code Action Committee (BCAC). The BCAC was established by the ICC Board of Directors to pursue opportunities to improve and enhance an assigned International Code or portion thereof. This includes both the technical aspects of the codes as well as the code content in terms of scope and application of referenced standards. Since its inception in July, 2011, the BCAC has held 5 open meetings and numerous workgroup calls which included members of the BCAC as well as any interested party to discuss and debate the proposed changes and the public comments. Related documentation and reports are posted on the BCAC website at: http://www.iccsafe.org/cs/BCAC/Pages/default.aspx.

There was much commotion on the floor when this code change proposal was being discussed. The proponents and opponents were confused as to who should be testifying. After the vote, both the opponents and proponents agreed that they liked this code change because it made sense to allow Section 2308 to be more widely applicable.

The committee disapproved this proposal because they thought that easing the limitations would potentially extend the application of Section 2308 to buildings not originally intended. They were right on, but they missed the beauty of this code change. With the current limitation of 40 psf floor live load –Section 2308 is seldom used! The live load table, Table 1607.1, only allows live load of 40 psf for the following applications: catwalks, patient rooms in hospitals, cell blocks, classrooms and habitable areas and stairs in residences. The proposed code change was intended to make 2308 useable: for small commercial structures or live work structures built with first floor slab on grade.

I did intend to extend the application and make 2308 more versatile. My original intention might not have been clear enough, so this public comment comes out and says that higher live loads can only be applied to a <u>floor is constructed on grade</u>. The public comment should make it clearer that someone cannot build with 100 psf live loads on conventional light frame construction applied to the second floor.

S276-12

Final Action: AS AM AMPC____ D

S280-12
2308.2.2

<u>Proposed Change as Submitted</u>

Proponent: Robert Rice, C.B.O., Josephine County, OR, representing Oregon Building Officials Association (structdesigner@yahoo.com)

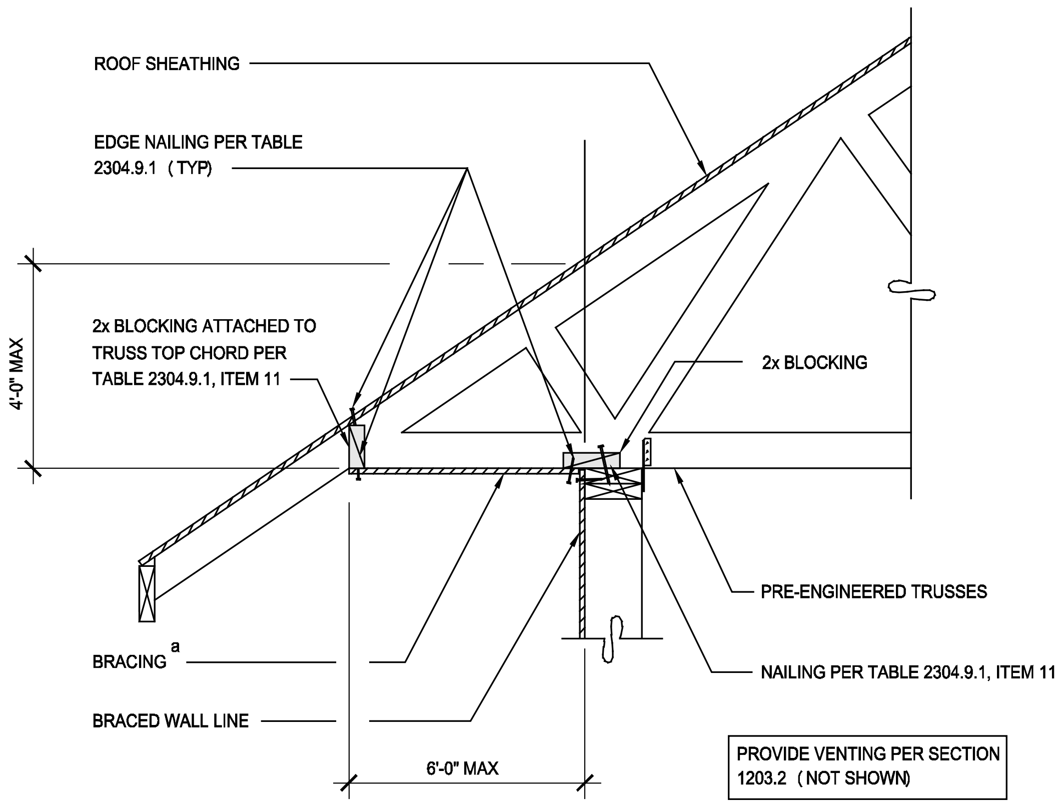
Revise as follows:

2308.3.2.2 Top plate connection. Where joists and/or rafters are used, braced wall line top plates shall be fastened over the full length of the braced wall line to joists, rafters, rimboards or blocking above in accordance with Table 2304.9.1, Items 11, 12, 15 or 19, as applicable, based on the orientation of the joists or rafters to the braced wall line. Blocking at joists with walls above shall be equal to the depth of the joist at the braced wall line. Blocking at rafters need not be full depth but shall extend to within 2 inches (51 mm) from the roof sheathing above. Blocking shall be a minimum of 2 inches (51 mm) nominal thickness and shall be fastened to the braced wall line top plate as specified in Table 2304.9.1, Item 11. Notching or drilling of holes in blocking in accordance with the requirements of Section 2308.8.2 or Section 2308.10.4.2 shall be permitted.

At exterior gable end walls braced wall panel sheathing in the top *story* shall be extended and fastened to roof framing where the spacing between parallel exterior braced wall lines is greater than 50 feet (15 240 mm).

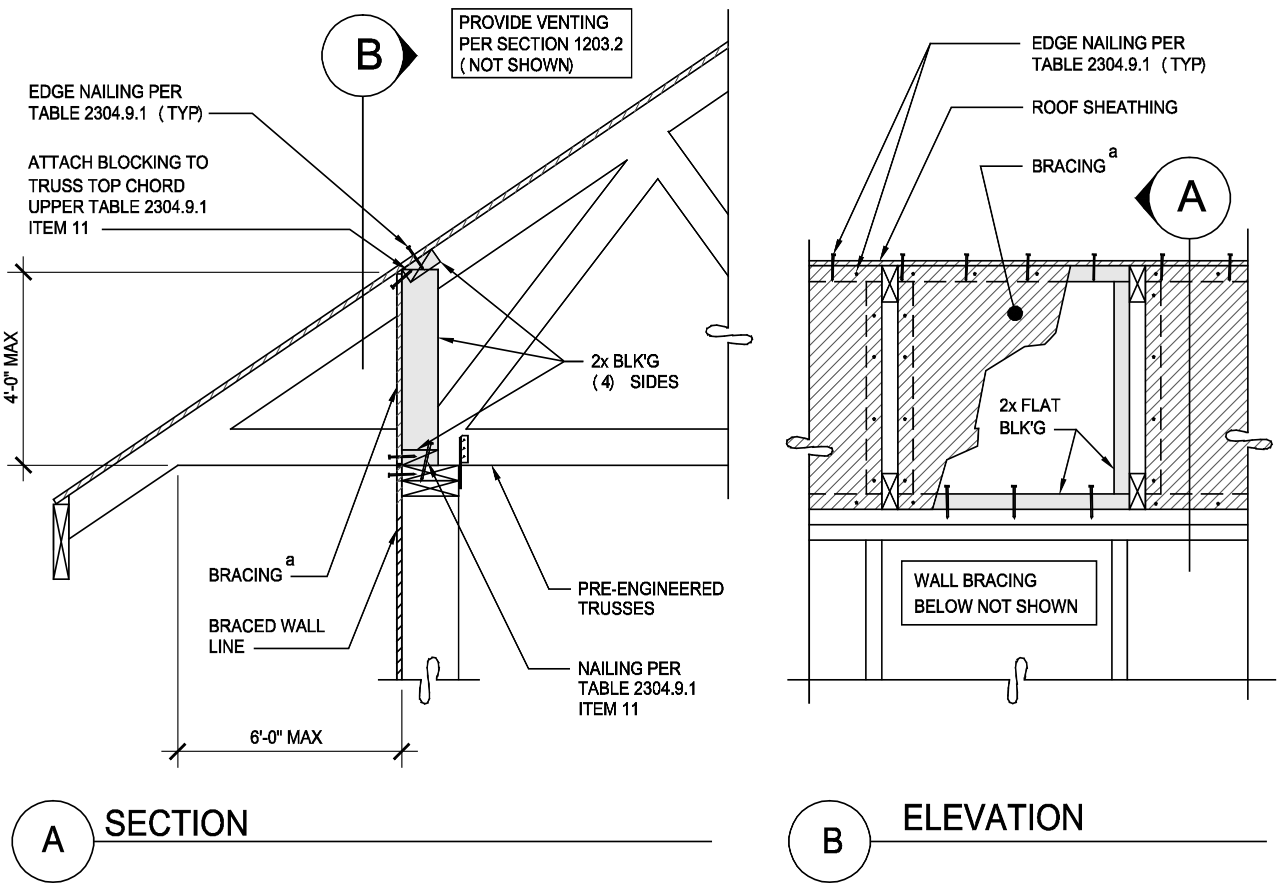
Where roof trusses are used and are installed perpendicular to an exterior braced wall line, lateral forces shall be transferred from the roof diaphragm to the braced wall over the full length of the braced wall line by blocking of the ends of the trusses or by other *approved* methods providing equivalent lateral force transfer. Blocking shall be minimum 2 inch (51 mm) nominal thickness and shall extend to within 2 inches (51 mm) from the roof sheathing above and shall be fastened to the braced wall line top plate as specified in Table 2304.9.1, Item 11. Notching or drilling of holes in blocking in accordance with the requirements of Section 2308.8.2 or Section 2308.10.4.2 shall be permitted.

> <u>**Exception**. Where the roof sheathing is greater than 9-1/4 inches (235 mm) above the top plate solid blocking is not required when the framing members are connected in accordance with one of the following methods:</u>
>
> 1. <u>In accordance with Figure 2308.3.2 (1)</u>
> 2. <u>In accordance with Figure 2308.3.2 (2)</u>
> 3. <u>With full height engineered blocking panels designed for values listed in American Forest and Paper Association (AF&PA) Wood Frame Construction Manual for One- and Two-Family Dwellings (WFCM).</u>
> 4. <u>Designed in accordance with accepted engineering methods.</u>



For SI: 1 inch = 25.4 mm
a. Methods of bracing shall be as described in Section 2308.9.3, method 2,3,4,6,7 or 8

FIGURE 2308.3.2(1)
BRACED WALL LINE TOP PLATE CONNECTION



For SI: 1 inch = 25.4 mm
a. Methods of bracing shall be as described in Section 2308.9.3, method 2,3,4,6,7 or 8

FIGURE 2308.3.2 (2)
BRACED WALL PANEL TOP PLATE CONNECTION

TABLE 2304.9.1
FASTENING SCHEDULE

CONNECTION	FASTENING[a]	LOCATION
1. Joist to sill or girder	3 - 8d common (2 1/2" x 0.131") 3 – 3 x 0.131 nails 3 – 3" x 14 gage staples	toenail
2. Bridging or blocking to joist, rafter or truss	2 - 8d common (2 1/2" 0.131") 2 – 3 x 0.131" nails 2 – 3" x 14 gage staples	toenail each end
11. Blocking between joists, ~~or~~ rafters or truss to top plate	3 - 8d common (2 1/2" x 0.131") 3 – 3" x 0.131 nails 3 – 3" 14 gage staples	toenail
Blocking between rafters or truss not at the wall top plate, to rafter or truss	2 - 8d common (2 1/2" x 0.131") 2 – 3" x 0.131" nails 2 – 3" 14 gage staples	toenail each end
	2 - 16d common (3 1/2" x 0.162") 3 – 3" x 0.131" nails 3 – 3" x 14 gage staples	endnail

(Portions of table not shown remain unchanged)

Reason: The 2012 IBC has fairly clear wording in Section 2308.3.2 that when the Conventional Light-Frame Construction provisions are used that the diaphragms need to be connected to the braced wall line to resist wind and seismic (lateral) forces and states.

The prescriptive provisions of "conventional light-frame construction" as provided for in section 2308 are very limited in scope. In section 2308.2 they are limited to:

1. Three stories max (two stories max in SDC C, one story in SDC D and above)
2. Max floor to floor height of 11'-7"
3. Max dead loads of 15 psf
4. Floor live load of 40 psf max
5. Ground snow of 50 psf max
6. Wind speeds of 100 max
7. Roof truss span of 40 feet max between vertical supports
8. Not allowed to be used for Occupancy Category IV buildings in SDC B,C,D,E
9 More restrictive requirements for SDC B,C, D and E defined in 2308.11.
10 Even more restrictive requirements specifically for SDC D and E
11. Limited by "irregular structures" definitions in 2308.12.6
12. Braced wall line spacing 35 feet max each direction, each floor.
13. In SDC D and E max spacing is 25 feet. (IRC allow exception up to 50 feet)

In other words, due to the limitations listed above as well as the other limitations in the code not listed here, the structures that are built with the provisions of section 2308 are small, light-framed buildings that do not have the significant lateral loading that other buildings do.

The alternate provisions in the exceptions are intended to address the increasingly common occurrence of cantilevered/high-heel trusses. This occurs due to insulation requirements and to provide a cantilevered portion of roof to be an exterior covered porch. The current provisions of this section of code do not cover this common condition. The current code language requires that "Blocking shall be a minimum of 2 inches (51 mm) nominal thickness…" This does not work for heights greater than what a 2x 10 or 2x 12 will accommodate.

The current code text (IBC) states the intention of connecting the braced wall line to the roof or floor diaphragm above in section 2308.3.2. A similar version of this proposal was adopted as an Oregon amendment in 2006 for the adoption of the 2006 IBC and has worked well for many years and two more code cycles Since then, countless hours have gone into developing proposals for both the IRC and the IBC code development process. The IRC proposal was approved in Minneapolis for the 2009 code. During the process of resolving concerns and developing a consensus changes were made to the proposal. Based on engineering reports and historical data, an exception was made for low heel connections (9 ¼") in lower wind and seismic zones to not require the blocking.

This proposal does not add additional requirements to the code. This proposal clarifies that the connection needs to occur and provides prescriptive solutions when solid blocking, per the current text, is not possible or is impractical..

Per accepted engineering practice for lateral design loads, the floor and roof diaphragms transmit wind and seismic loads into the braced walls (engineered shearwalls or prescriptive braced panels). The fact that the diaphragm needs to be connected to the braced wall line to complete the load path is often not fully understood by plans examiners, inspectors and contractors. The typical requirement that is intended by the code is that full height solid blocking occur at this connection with edge nailing to the blocking and the blocking connected to the top plate of the wall to transfer the diaphragm (plf) force to the wall top plates. This is evidenced in the IBC by the exception to irregular structures stating, *"..lateral forces shall be transferred from the roof diaphragm to the braced wall by blocking of the ends of the trusses.."*. In order for the forces to be transferred there has to be a connection capable of transferring the diaphragm shear evenly to the top plates.

Without this clarification of the text it is a connection that may or may not occur based on what I have seen in the field and have discussed with code officials. The blocking that is called for in the code serves three functions. It provides closure to prevent animals, birds, etc. from entering the attic space, it prevents the trusses or rafters from "rolling over" and it transfers the diaphragm forces to the wall. Most code officials, inspectors and contractors understand the first two objectives. However, the latter is a concept that is often not fully understood. This needs to be perceived, understood and implemented in a uniform way.

In addition, rather than identify a problem without providing a solution, my proposal includes two details to accomplish this connection simply. The solutions are, in principle, fundamentally extending the roof diaphragm sheathing to the wall top plates either vertically in the truss bays or horizontally through the soffit. No design is required since it is just completing the load path with the already defined sheathing and nailing.

Without prescriptive provisions in the current code this condition would require engineering or, as stated in 2308.3.2, Exception to item 1 "..by other approved methods." would be left up to the Authority Having Jurisdiction to determine what is acceptable without any guidance or uniformity between jurisdictions.

Typically, the engineering solution would provide details similar to those included in this proposal. Therefore, the solution and construction costs would not change. Costs would be reduced by eliminating additional costs for engineering where these prescriptive solutions work.

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

2308.3.2.2-S-RICE.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: Disapproval is based on a number of deficiencies in the proposed figures, including nailing, panel uplift and continuous vent effect on load path. It would require the connections along braced wall lines that are preferred at braced wall panels only.

Assembly Action: **None**

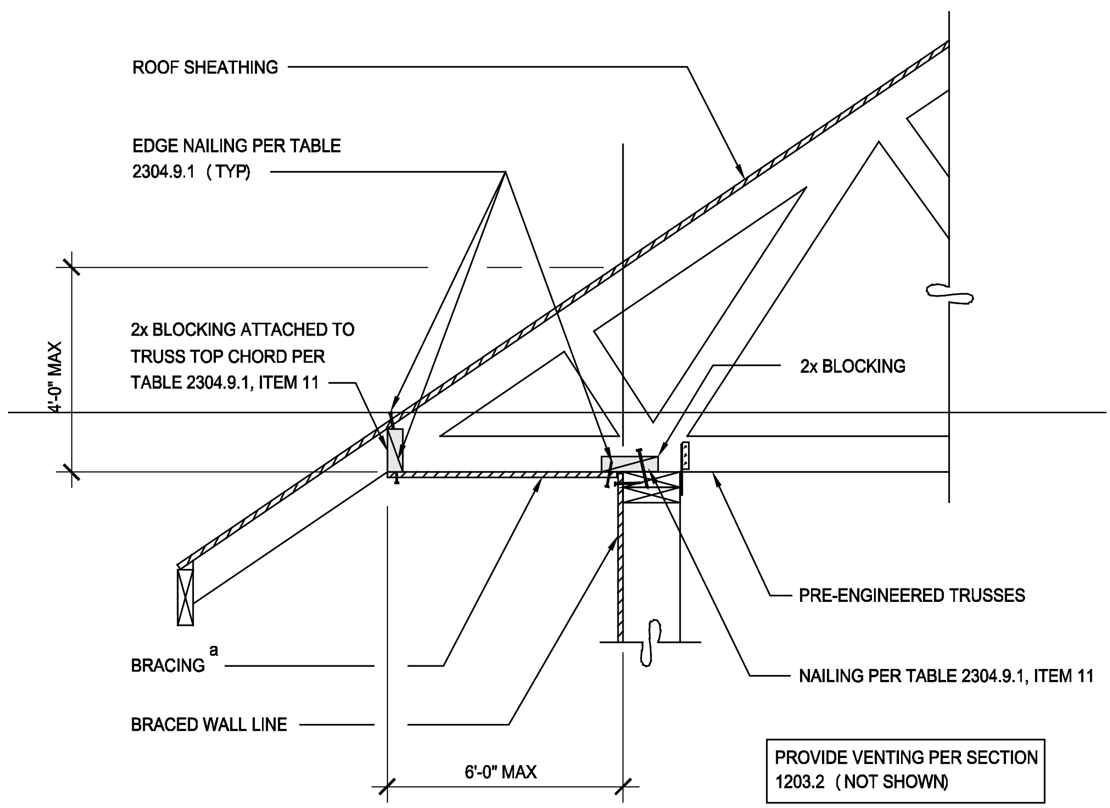
Individual Consideration Agenda

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

Public Comment:

Robert Rice, Josephine County Oregon representing Oregon Building Officials Association and J. Daniel Dolan representing FEMA Code Resource Support Committee, requests Approval as Modified by this Public Comment.

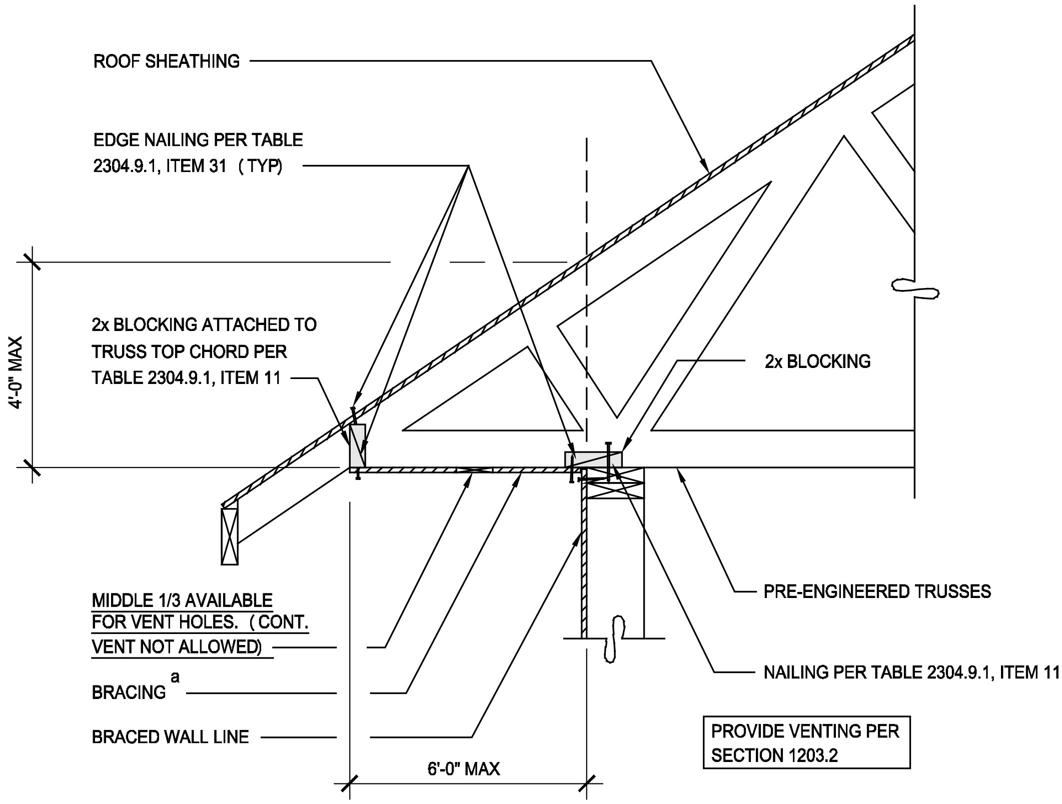
Modify the proposal as follows:



a. Methods of bracing shall be as described in Section 2308.9.3 method 2, 3, 4, 6, 7 or 8

~~For SI: 1 inch = 25.4 mm~~
~~a. Methods of bracing shall be as described in Section 2308.9.3, method 2,3,4,6,7 or 8~~

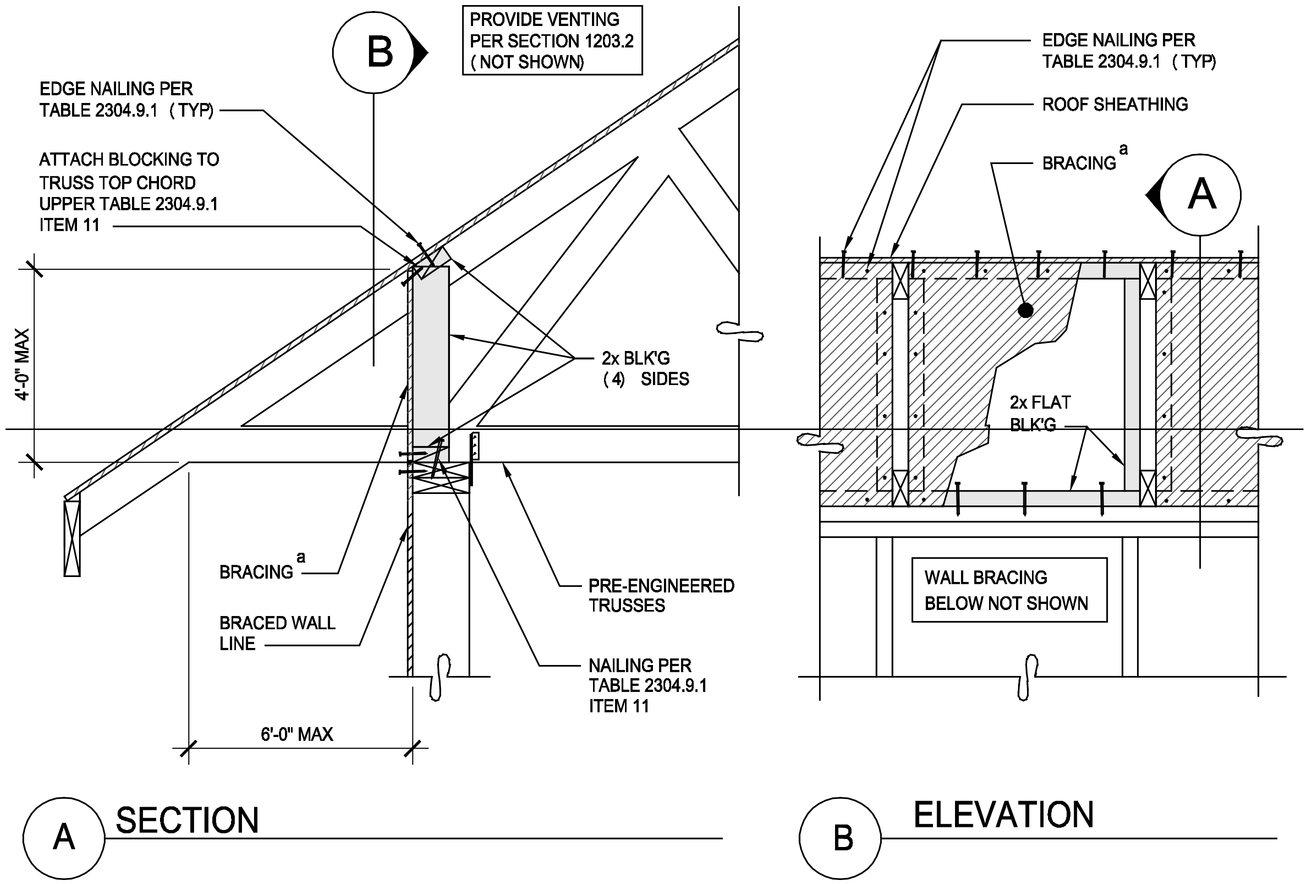
~~**FIGURE 2308.3.2(1)**~~
~~**BRACED WALL LINE TOP PLATE CONNECTION**~~



a. Methods of bracing shall be as described in Section 2308.9.3 methods 2, 3, 4, 6, 7 or 8

For SI: 1 inch = 25.4 mm

FIGURE 2308.3.2(1)
BRACED WALL LINE TOP PLATE CONNECTION

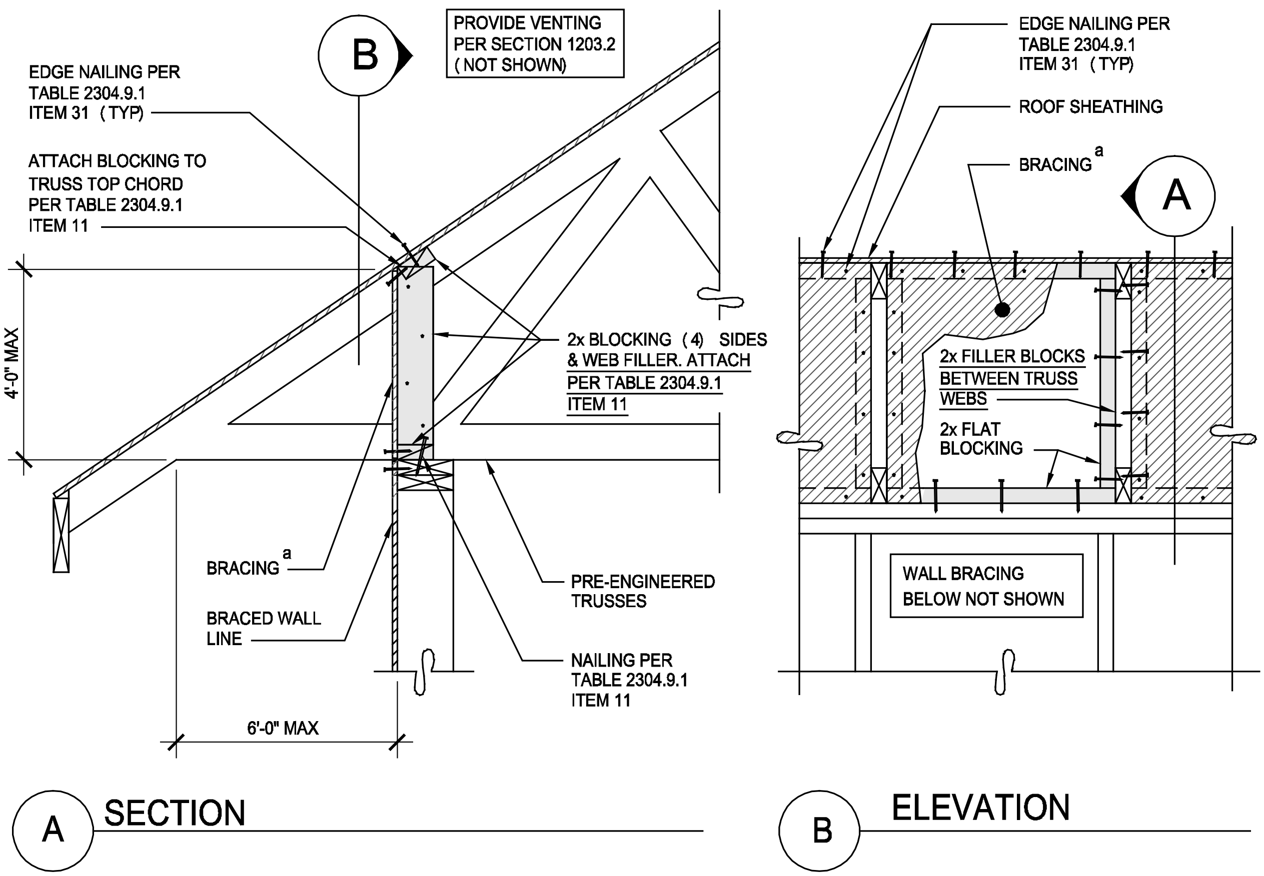


~~For SI: 1 inch = 25.4 mm~~

~~a. Methods of bracing shall be as described in Section 2308.9.3, method 2,3,4,6,7 or 8~~

~~FIGURE 2308.3.2 (2)~~

~~BRACED WALL PANEL TOP PLATE CONNECTION~~



a. Methods of bracing shall be as described in Section 2308.9.3 methods 2, 3, 4, 6, 7 or 8

<u>For SI: 1 inch = 25.4 mm</u>

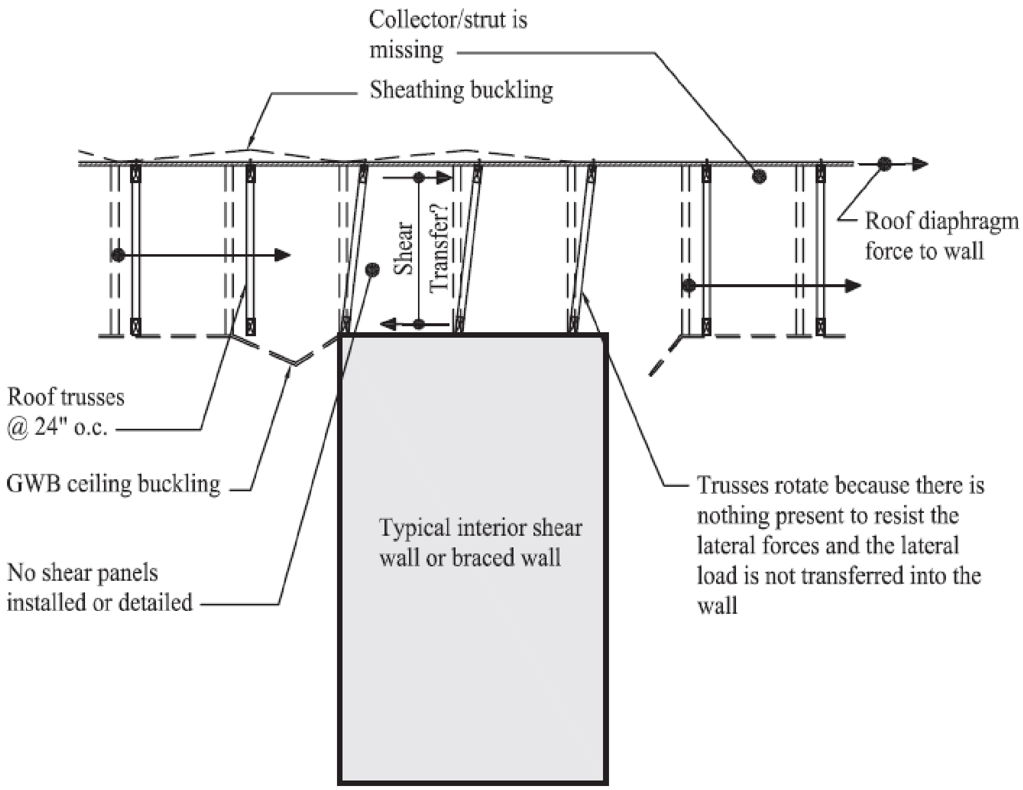
<u>FIGURE 2308.3.2 (2)</u>
<u>BRACED WALL PANEL TOP PLATE CONNECTION</u>

TABLE 2304.9.1
FASTENING SCHEDULE

CONNECTION	FASTENING[a]	LOCATION
11. Blocking between joists, rafters or truss to top plate	3 - 8d common (2 1/2" x 0.131") 3 – 3" x 0.131 nails 3 – 3" 14 gage staples	toenail
Blocking between rafters or truss not at the wall top plate, to rafter or truss	2 - 8d common (2 1/2" x 0.131") 2 – 3" x 0.131" nails 2 – 3" 14 gage staples	toenail each end
	2 - 16d common (3 1/2" x 0.162") 3 – 3" x 0.131" nails 3 – 3" x 14 gage staples	endnail
<u>Flat blocking to truss and web filler</u>	<u>16d common (3 1/2" x 0.162") @ 6" o.c.</u> <u>3" x 0.131" nails @ 6" o.c.</u> <u>3" x 14 gage staples @ 6" o.c.</u>	<u>face nail</u>

(Portions of Table and proposal not shown remain unchanged)

Commenter's Reason: The original proposal, as submitted, addresses a construction condition that is becoming much more common with an increase in the use of cantilevered and high-heel trusses. Cantilevered trusses are often incorporated to create a covered entry way for a portion of a wall line. Additionally, high-heel stub trusses are becoming more common to accommodate deeper attic insulation to meet increased energy-efficiency requirements. The concern regarding the lack of load path from the roof diaphragm to the braced wall line is illustrated in the example below from the publication, *Analysis of Irregular Shaped Structures: Diaphragms and Shearwalls, McGraw/Hill – ICC 2011,*



The details shown in the original proposal, and further modified by this public comment, provide the necessary load path from the roof diaphragm to the braced wall line. Reality is, this condition often occurs without consideration of the incomplete load path either by the plans examiner or the inspector. IBC Section 2308, *Conventional Light-Frame Construction,* is very limited in scope and only applies to smaller, lightly loaded structures with additional restrictions for structures in seismic design categories B, C, D and E as noted in the original proposals reason statement. The prescriptive details from the original proposal as modified by this public comment will ensure that the condition is addressed and provide sufficient connection for structures within the scope of section 2308.

Further, these details are consistent with engineering reports addressing light-framed wood structures such as ATC-7 *Proceedings of a Workshop on Design of Horizontal Wood Diaphragms, November 19-20, 1979* - Applied Technology Council and ICC 600-2008, *Standard for Residential Construction in High-wind Regions.* The figures shown below from those two publications essentially provide the same detail and have been used and considered accepted engineering practice for many decades.

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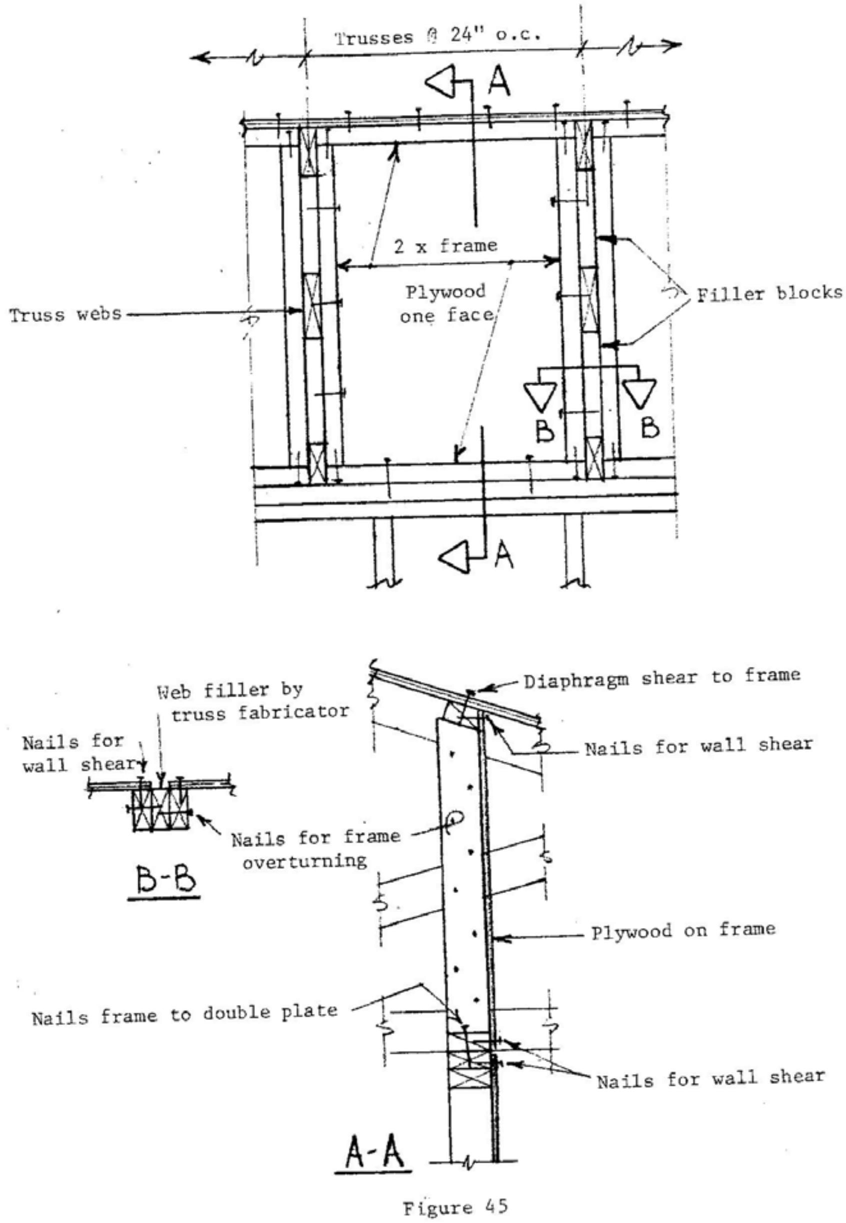


Figure 45 from ATC-7. *Proceedings of a Workshop on Design of Horizontal Wood Diaphragms, November 19-20, 1979* - Applied Technology Council

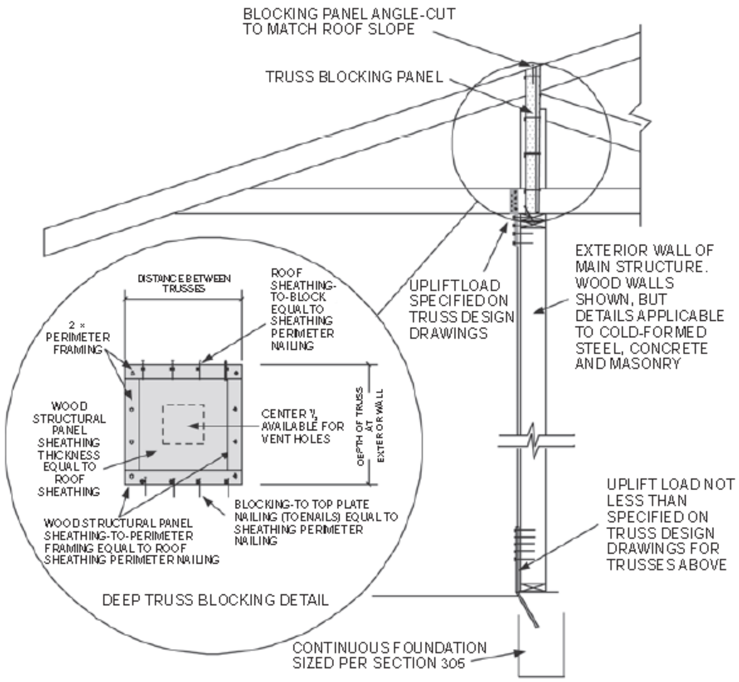


FIGURE 308(3)
SECTION A-A—OVERHANG UPLIFT RESISTANCE DESIGNED ROOF TRUSS

STANDARD FOR RESIDENTIAL CONSTRUCTION IN HIGH-WIND REGIONS—2008

Figure 308(3) from ICC 600-2008, *Standard for Residential Construction in High-wind Regions, ICC*

As stated in the original proposal, structures built per section 2308 have the following limitations;

1. *Three stories max (two stories max in SDC C, one story in SDC D and above)*
2. *Max floor to floor height of 11'-7"*
3. *Max dead loads of 15 psf*
4. *Floor live load of 40 psf max*
5. *Ground snow of 50 psf max*
6. *Wind speeds of 100 max*
7. *Roof truss span of 40 feet max between vertical supports*
8. *Not allowed to be used for Occupancy Category IV buildings in SDC B,C,D,E*
9. *More restrictive requirements for SDC B,C, D and E defined in 2308.11.*
10. *Even more restrictive requirements specifically for SDC D and E*
11. *Limited by "irregular structures" definitions in 2308.12.6*
12. *Braced wall line spacing 35 feet max each direction, each floor.*
13. *I n SDC D and E max spacing is 25 feet*

At the Code Development Hearings (CDH), concern was expressed about panel uplift at the individual panels when resisting lateral forces. This could be a concern especially if the panels or blocking only occurred at the individual *Braced Wall Panels.* IBC Section 2308.3.2.2 requires that *"….lateral forces shall be transferred from the roof diaphragm to the braced wall* ***over the full length of the braced wall line*** *by blocking of the ends of the trusses or by other approved methods providing equivalent lateral force transfer."* As shown below in a sample (Figure 9.19) from the publication, *Analysis of Irregular Shaped Structures: Diaphragms and Shearwalls*, McGraw/Hill – ICC 2012, when the panels occur continuously the net uplift force at each panel is zero as the adjacent panels counteract with a downward force. This is accomplished by adequate connection between adjacent panels and has been addressed in this public comment by the addition of 2x web filler and nailing that was not specified in the original proposal.

Figure 2308.3.2(2) has been modified in this public comment to include specific nailing requirements for the vertical blocking and calls for truss web infill blocking. The fasteners required for this connection have been added to the fastener table, Table 2304.9.1

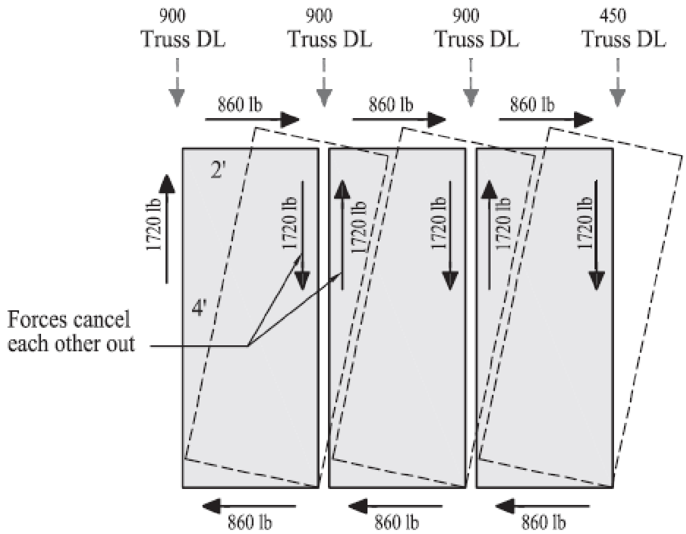


FIGURE 9.19 Individual shear panel forces.

Figure 9.19 from the publication*, Analysis of Irregular Shaped Structures: Diaphragms and Shearwalls,* McGraw/Hill – ICC 2012,

The details provided in this proposal, as with the already-required 2x solid blocking at lower depth heel areas, will satisfy the requirement of the code and the details would provide "..*equivalent lateral force transfer".*

It is important to note that, in addition to the details provided for prescriptive solutions, the text (2308.3.2.2 exceptions 3 and 4) also provide allowance for the option of engineered blocking panels provided by the truss manufacturer as well as a design in accordance with accepted engineering methods. In other words, these details are just prescriptive options. There are other options available to the code user

Another issue raised at the CDH was that, *"It would require the connections along **braced wall lines** that are preferred at **braced wall panels** only.* This is nearly opposite of the other concerns expressed and implied that it would be too strict and demanding. This concern is a moot point since, as stated above, section 2308.3.2.2 already requires that *"….lateral forces shall be transferred from the roof diaphragm to the braced wall over the **full length of the braced wall line**….*and no change in that requirement is proposed. Without these prescriptive solutions the solution to provide a complete load path would require engineering in every case. The result of the engineering would likely mimic these details.

Question arose at the CDH regarding the venting in figure 2308.3.2(1). There was concern that the figure, as shown in the original proposal, would allow a continuous vent in the soffit which would disrupt the continuity of the sheathing and may reduce the sheathings capacity to transfer the required shear force to the wall line. The modified figure in this public comment defines the allowed area for vent holes and specifically prohibits continuous venting without an engineered design.

As a footnote, the figures in this proposal refer to the current tables and bracing methods in the 2012 IBC. Upon passage of S273, the figures in this proposal that will be submitted to ICC will reflect the new table numbers and bracing methods. S273 does not change the technical requirements. Only the table and method numbers and names have changed. Both proposals have been developed by the same group of interested parties and upon approval, they will be coordinated with ICC staff to work seamlessly together.

This proposal, as amended by this public comment, adequately addresses the issues of providing a complete load path with prescriptive solutions that would other wise require additional engineering services and, in most cases, produce the same or similar details.

Bibliography:

1. Malone, R. Terry, and Robert W. Rice. *Analysis of Irregular Shaped Structures: Diaphragms and Shearwalls,* Washington DC: McGraw/Hill 2012 (Co-branded with International Code Council)
2. ATC. *Proceedings of a Workshop on Design of Horizontal Wood Diaphragms,* Redwood City, CA: Applied Technology Council.
3. ICC. ICC 600-2008, *Standard for Residential Construction in High-wind Regions.* Washington DC: International Code Council

S280-12

Final Action: AS AM AMPC____ D

S281-12

2308.7, 2308.9.1, 2308.9.5.1, 2308.9.5.2, 2308.9.6, Table 2308.9.5, Table 2308.9.6

Proposed Change as Submitted

Proponent: Paul Coats, PE, CBO, American Wood Council (pcoats@awc.org)

Revise as follows:

2308.7 Girders. Girders for single-story construction or girders supporting loads from a single floor shall not be less than 4 inches by 6 inches (102 mm by 152 mm) for spans 6 feet (1829 mm) or less, provided that girders are spaced not more than 8 feet (2438 mm) o.c. ~~Spans for built-up 2-inch (51 mm) girders shall be in accordance with Table 2308.9.5 or 2308.9.6. Other girders~~ Girders shall be designed to support the loads specified in this code. Girder end joints shall occur over supports. Where a girder is spliced over a support, an adequate tie shall be provided. The ends of beams or girders supported on masonry or concrete shall not have less than 3 inches (76 mm) of bearing.

2308.9.1 Size, height and spacing. The size, height and spacing of studs shall be in accordance with Table 2308.9.1 except that utility-grade studs shall not be spaced more than 16 inches (406 mm) o.c., or support more than a roof and ceiling, or exceed 8 feet (2438 mm) in height for exterior walls and load-bearing walls or 10 feet (3048 mm) for interior nonload-bearing walls. Studs shall be continuous from a support at the sole plate to a support at the top plate to resist loads perpendicular to the wall. The support shall be a foundation or floor, ceiling or roof diaphragm or shall be designed in accordance with accepted engineering practice.

> **Exception:** Jack studs, trimmer studs and cripple studs at openings in walls that comply with ~~Table 2308.9.5~~ Section 2308.9.5.2.

2308.9.5.1 Headers. Headers shall be provided over each opening in exterior-bearing walls. ~~The spans in Table 2308.9.5 are permitted to be used for one- and two-family *dwellings*.~~ Headers ~~for other buildings~~ shall be designed in accordance with Section 2301.2, Item 1 or 2. Headers shall be of two or more pieces of nominal 2-inch (51 mm) framing lumber set on edge ~~as permitted by Table 2308.9.5~~ and nailed together in accordance with Table 2304.9.1 or of solid lumber of equivalent size.

2308.9.5.2 Header support. Wall studs shall be designed to support the ends of the header ~~in accordance with Table 2308.9.5~~. Each end of a lintel or header shall have a length of bearing of not less than $1^1/_2$ inches (38 mm) for the full width of the lintel.

2308.9.6 Openings in interior bearing partitions. Headers shall be provided over each opening in interior bearing partitions as required in Section 2308.9.5. ~~The spans in Table 2308.9.6 are permitted to be used.~~ Wall studs shall support the ends of the header in accordance with ~~Table 2308.9.5 or 2308.9.6, as appropriate~~ Section 2308.9.5.2.

~~TABLE 2308.9.5~~
~~HEADER AND GIRDER SPANS[a] FOR EXTERIOR BEARING WALLS~~
~~(Maximum Spans for Douglas Fir-Larch, Hem-Fir, Southern Pine and Spruce-Pine-Fir[b] and Required Number of Jack Studs)~~

~~TABLE 2308.9.6~~
~~HEADER AND GIRDER SPANS[a] FOR INTERIOR BEARING WALLS~~
~~(Maximum Spans for Douglas Fir-Larch, Hem-Fir, Southern Pine and Spruce-Pine-Fir[b] and Required Number of Jack Studs)~~

Reason: Deletion of Table 2308.9.5 and Table 2308.9.6 without replacement is proposed because of limited applicability of the tabulated header spans resulting from the exclusion of detached one- and two-family dwellings from the scope of 2308 and the live load limitation of 40 psf per 2308.2. In addition, the species-based header spans are subject to being dated should design values change. Design value-based prescriptive engineered options for header spans are available from other sources. For example, header spans for conditions covered by Table 2308.9.5 and Table 2308.9.6, as well as support of headers by use of jack studs providing full bearing, can be found in the WFCM.

Specific reference to “one- and two- family dwellings” from 2308.9.5.1 is deleted to coordinate with the exclusion of detached one-and two-family dwellings from the scope of 2308. Other text sections are revised to coordinate with removal of the Tables.

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

2308.7-S-COATS.doc

Public Hearing Results

Committee Action: Disapproved

Committee Reason: The committee believes that the header span tables are needed in the conventional construction provisions. Outside of Southern Pine, there was no testimony to justify the removal of other wood species. Where there are problems the committee would like to see them fixed. Also adding requirements for “to be designed” is not appropriate for conventional construction.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Paul D. Coats, American Wood Council, requests Approval as Modified by this Public Comment.

Replace the proposal as follows:

2308.7 Girders. Girders for single-story construction or girders supporting loads from a single floor shall not be less than 4 inches by 6 inches (102 mm by 152 mm) for spans 6 feet (1829 mm) or less, provided that girders are spaced not more than 8 feet (2438 mm) o.c. ~~Spans for built-up 2-inch (51 mm) girders shall be in accordance with Table 2308.9.5 or 2308.9.6.~~ Other girders shall be designed to support the loads specified in this code. Girder end joints shall occur over supports. Where a girder is spliced over a support, an adequate tie shall be provided. The ends of beams or girders supported on masonry or concrete shall not have less than 3 inches (76 mm) of bearing.

2308.7.1 Allowable girder spans. The allowable spans of girders fabricated of dimension lumber shall not exceed the values set forth in Tables 2308.9.5 and 2308.9.6

TABLE 2308.9.5

~~**HEADER AND GIRDER SPANS[a] FOR EXTERIOR BEARING WALLS**~~

~~**(Maximum Spans for Douglas Fir-Larch, Hem-Fir, Southern Pine and Spruce-Pine-Fir[b] and Required Number of Jack Studs)**~~

GIRDER SPANS[a] AND HEADER SPANS[a] FOR EXTERIOR BEARING WALLS (Maximum spans for Douglas fir-larch, hem-fir, southern pine and spruce-pine-fir[b] and required number of jack studs)

EXTRACT TABLE R502.5(1) (except 70 psf snow load columns) of the International Residential Code

TABLE 2308.9.6

~~**HEADER AND GIRDER SPANS[a] FOR INTERIOR BEARING WALLS**~~

~~**(Maximum Spans for Douglas Fir-Larch, Hem-Fir, Southern Pine and Spruce-Pine-Fir[b] and Required Number of Jack Studs)**~~

GIRDER SPANS[a] AND HEADER SPANS[a] FOR INTERIOR BEARING WALLS (Maximum spans for Douglas fir-larch, hem-fir, southern pine and spruce-pine-fir[b] and required number of jack studs)

EXTRACT entire TABLE R502.5(2) of the International Residential Code

Commenter’s Reason: The spans in the girder tables in the IBC and IRC are identical, and buildings that qualify for conventional construction in the IBC have loading limitations commensurate with residential buildings--buildings within the scope of the IRC. It is also our intent to propose adjustments to spans in the IRC code which will automatically update the spans in these tables in the IBC.

In this way species-specific spans for girder and headers will be automatically correlated between the two codes, even though the codes themselves are developed in separate code change cycles.

To facilitate the maintenance of these tables by the IRC committee, a new section of charging text, identical to the charging text for these tables in IRC Section R502.5, has been added and the existing charging text in 2308.7 has been deleted.

Further information about this change is posted at: http://www.awc.org/Code-Officials/2012-IBC-Challenges.

Analysis. The result of this public comment if successful would be to extract the span tables from the 2012 IRC and, in order to achieve consistency between the IBC and IRC, would also include any changes made to the subject IRC span tables during the 2013 Code Change Cycle. Any changes to code change committee responsibilities in future code development cycles are not part of this code change, but are the responsibility of the ICC Code Correlation Committee.

S281-12

Final Action: AS AM AMPC____ D

S283-12

2308.8, Table 2308.8(1), Table 2308.8(2), 2308.10.2, Table 2308.10.2(1), Table 2308.10.2(2), 2308.10.3, Table 2308.10.3(1), Table 2308.10.3(2), Table 2308.10.3(3), Table 2308.10.3(4), Table 2308.10.3(5), Table 2308.10.3(6)

<u>Proposed Change as Submitted</u>

Proponent: Paul Coats, P.E. CBO, American Wood Council (pcoats@awc.org)

Revise as follows:

2308.8 Floor joists. Spans for floor joists shall be in accordance with ~~Table 2308.8(1) or 2308.8(2). For other grades and or species, refer to~~ the *AF&PA Span Tables for Joists and Rafters*.

~~**TABLE 2308.8(1)**~~
~~**FLOOR JOIST SPANS FOR COMMON LUMBER SPECIES**~~
~~**(Residential Sleeping Areas, Live Load = 30 psf, L/Δ = 360)**~~

~~**TABLE 2308.8(2)**~~
~~**FLOOR JOIST SPANS FOR COMMON LUMBER SPECIES**~~
~~**(Residential Living Areas, Live Load = 40 psf, L/Δ = 360)**~~

2308.10.2 Ceiling joist spans. Allowable spans for ceiling joists shall be in accordance with ~~Table 2308.10.2(1) or 2308.10.2(2). For other grades and species, refer to~~ the ~~*AF&PA*~~ *<u>AWC</u> Span Tables for Joists and Rafters*.

~~**TABLE 2308.10.2(1)**~~
~~**CEILING JOIST SPANS FOR COMMON LUMBER SPECIES**~~
~~**(Uninhabitable Attics Without Storage, Live Load = 10 pounds psf, L/Δ = 240)**~~

~~**TABLE 2308.10.2(2)**~~
~~**CEILING JOIST SPANS FOR COMMON LUMBER SPECIES**~~
~~**(Uninhabitable Attics With Limited Storage, Live Load = 20 pounds per square foot, L/Δ = 240)**~~

2308.10.3 Rafter spans. Allowable spans for rafters shall be in accordance with ~~Table 2308.10.3(1), 2308.10.3(2), 2308.10.3(3), 2308.10.3(4), 2308.10.3(5) or 2308.10.3(6). For other grades and species, refer to the *AF&PA*~~ <u>the *AWC*</u> *Span Tables for Joists and Rafters*.

~~**TABLE 2308.10.3(1)**~~
~~**RAFTER SPANS FOR COMMON LUMBER SPECIES**~~
~~**(Roof Live Load = 20 pounds per square foot, Ceiling Not Attached to Rafters, L/Δ = 180)**~~

~~**TABLE 2308.10.3(2)**~~
~~**RAFTER SPANS FOR COMMON LUMBER SPECIES**~~
~~**(Roof Live Load = 20 pounds per square foot, Ceiling Attached to Rafters, L/Δ = 240)**~~

~~**TABLE 2308.10.3(3)**~~
~~**RAFTER SPANS FOR COMMON LUMBER SPECIES**~~
~~**(Ground Snow Load = 30 pounds per square foot, Ceiling Not Attached to Rafters, L/Δ = 180)**~~

~~**TABLE 2308.10.3(4)**~~
~~**RAFTER SPANS FOR COMMON LUMBER SPECIES**~~
~~**(Ground Snow Load = 50 pounds per square foot, Ceiling Not Attached to Rafters, L/Δ = 180)**~~

~~**TABLE 2308.10.3(5)**~~
~~**RAFTER SPANS FOR COMMON LUMBER SPECIES**~~
~~**(Ground Snow Load = 30 pounds per square foot, Ceiling Attached to Rafters, L/Δ = 240)**~~

~~**TABLE 2308.10.3(6)**~~
~~**RAFTER SPANS FOR COMMON LUMBER SPECIES**~~
~~**(Ground Snow Load = 50 pounds per square foot, Ceiling Attached to Rafters, L/Δ = 240)**~~

Reason: Species- and grade-specific span tables are subject to becoming dated if design values for specific species or grades change, and therefore it is proposed to directly reference the AWC Span Tables for Joists and Rafters. The design value format of the tabulated spans in Span Tables for Joists and Rafters is not sensitive to design value changes for specific species and grades. Span Tables for Joists and Rafters is currently included as a reference in IBC 2306.

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

2308.8-S-COATS.do

Public Hearing Results

Committee Action: Disapproved

Committee Reason: Similar to S281-12 if there is a problem with the span table, the committee feels it should be fixed rather than removed since Sectin2308 should be a cook book approach.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Paul D. Coats, American Wood Council, requests Approval as Modified by this Public Comment.

Replace the proposal as follows:

2308.8 Floor joists. Spans for floor joists shall be in accordance with Table 2308.8(1) or 2308.8(2). For other grades and or species, refer to the *~~AF&PA~~* *AWC* *Span Tables for Joists and Rafters*.

Table 2308.8(1) ~~FLOOR JOIST SPANS FOR COMMON LUMBER SPECIES (Residential Sleeping Areas, Live Load = 30 psf, L/Δ = 360~~ FLOOR JOIST SPANS FOR COMMON LUMBER SPECIES (Residential Sleeping Areas, Live Load = 30 psf, L/Δ = 360 ~~)~~
EXTRACT Table R502.3(1) from the International Residential Code (do not extract footnote a)

Table 2308.8(2) ~~FLOOR JOIST SPANS FOR COMMON LUMBER SPECIES (Residential Living Areas, Live Load = 40 psf, L/Δ = 360)~~ FLOOR JOIST SPANS FOR COMMON LUMBER SPECIES (Residential living areas, live load = 40 psf, L/Δ = 360)
EXTRACT Table R502.3(2) from the International Residential Code (do not extract footnote b)

2308.10.2 Ceiling joist spans. ~~Allowable s~~Spans for ceiling joists shall be in accordance with Table 2308.10.2(1) or Table 2308.10.2(2). For other grades and species, and for other loading conditions, refer to the *~~AF&PA~~* *AWC* Span Tables for Joists and Rafters.

Table 2308.10.2(1) ~~CEILING JOIST SPANS FOR COMMON LUMBER SPECIES (Uninhabitable Attics Without Storage, Live Load = 10 pounds psf, L/Δ = 240)~~ CEILING JOIST SPANS FOR COMMON LUMBER SPECIES (Uninhabitable attics without storage, live load = 10 psf, L/Δ = 240)
EXTRACT Table R802.4(1) from the International Residential Code

Table 2308.10.2(2) ~~CEILING JOIST SPANS FOR COMMON LUMBER SPECIES (Uninhabitable Attics With Limited Storage, Live Load = 20 pounds per square foot, L/Δ = 240)~~ CEILING JOIST SPANS FOR COMMON LUMBER SPECIES (Uninhabitable attics with limited storage, live load = 20 psf, L/Δ = 240)
EXTRACT Table R802.4(2) from the International Residential Code

2308.10.3 Rafter spans. ~~Allowable s~~Spans for rafters shall be in accordance with Table 2308.10.3(1), 2308.10.3(2), 2308.10.3(3), 2308.10.3(4), 2308.10.3(5) or 2308.10.3(6). For other grades and species and for other loading conditions, refer to the ~~*AF&PA*~~ *AWC Span Tables for Joists and Rafters*. The span of each rafter shall be measured along the horizontal projection of the rafter.

TABLE 2308.10.3(1) ~~RAFTER SPANS FOR COMMON LUMBER SPECIES (Roof Live Load = 20 pounds per square foot, Ceiling Not Attached to Rafters, L/Δ = 180)~~ RAFTER SPANS FOR COMMON LUMBER SPECIES (Roof live load = 20 psf, ceiling not attached to rafters, L/Δ = 180)
EXTRACT Table R802.5.1(1) from the International Residential Code

TABLE 2308.10.3(2) ~~RAFTER SPANS FOR COMMON LUMBER SPECIES (Roof Live Load = 20 pounds per square foot, Ceiling Attached to Rafters, L/Δ = 240)~~ RAFTER SPANS FOR COMMON LUMBER SPECIES (Roof Live Load = 20 psf, ceiling attached to rafters, L/Δ = 240)
EXTRACT Table R803.5.1(2) from the International Residential Code

TABLE 2308.10.3(3) ~~RAFTER SPANS FOR COMMON LUMBER SPECIES (Ground Snow Load = 30 pounds per square foot, Ceiling Not Attached to Rafters, L/Δ = 180)~~ RAFTER SPANS FOR COMMON LUMBER SPECIES (Ground Snow Load = 30 psf, ceiling not attached to rafters, L/Δ = 180)
EXTRACT Table R802.5.1(3) from the International Residential Code

TABLE 2308.10.3(4) ~~RAFTER SPANS FOR COMMON LUMBER SPECIES (Ground Snow Load = 50 pounds per square foot, Ceiling Not Attached to Rafters, L/Δ = 180)~~ RAFTER SPANS FOR COMMON LUMBER SPECIES (Ground Snow Load = 50 psf, ceiling not attached to rafters, L/Δ = 180)
EXTRACT Table R802.5.1(4) from the International Residential Code

TABLE 2308.10.3(5) ~~RAFTER SPANS FOR COMMON LUMBER SPECIES (Ground Snow Load = 30 pounds per square foot, Ceiling Attached to Rafters, L/Δ = 240)~~ RAFTER SPANS FOR COMMON LUMBER SPECIES (Ground Snow Load = 30 psf, ceiling attached to rafters, L/Δ = 240)
EXTRACT Table R803.5.1(5) from the International Residential Code

TABLE 2308.10.3(6) ~~RAFTER SPANS FOR COMMON LUMBER SPECIES (Ground Snow Load = 50 pounds per square foot, Ceiling Attached to Rafters, L/Δ = 240)~~ RAFTER SPANS FOR COMMON LUMBER SPECIES (Ground Snow Load = 50 psf, ceiling attached to rafters, L/Δ = 240)
EXTRACT Table R802.5.1(6) from the International Residential Code

Commenter's Reason: The spans for joists and rafters in the conventional construction provisions of the IBC and IRC are identical, and buildings that qualify for conventional construction in the IBC have loading limitations commensurate with residential buildings--buildings within the scope of the IRC. It is also our intent to propose adjustments to spans in the IRC code which will automatically update the spans in these tables in the IBC. In this way species-specific spans for joists and rafters will be automatically correlated between the two codes, even though the codes themselves are developed in separate code change cycles.

To facilitate the maintenance of these tables by the IRC committee, the sections containing the charging text have been modified to read exactly like the corresponding sections in the IRC. Footnotes to the IRC tables that would not apply will not be extracted, as indicated in the public comment.

Further information about this change is posted at: http://www.awc.org/Code-Officials/2012-IBC-Challenges.

Analysis. The result of this public comment if successful would be to extract the span tables from the 2012 IRC and, in order to achieve consistency between the IBC and IRC, would also include any changes made to the subject IRC span tables during the 2013 Code Change Cycle. Any changes to code change committee responsibilities in future code development cycles are not part of this code change, but are the responsibility of the ICC Code Correlation Committee.

S283-12

Final Action: AS AM AMPC____ D

S287-12

202 (New), 2302, 2308.9.3 (New), 2304.6, Table 2304.6, 2304.6.1, 2304.6.2

Proposed Change as Submitted

Proponent: Paul Coats, American Wood Council, (pcoats@awc.org)

Add new text as follows:

SECTION 202
DEFINITIONS

GABLE. The triangular portion of the wall beneath a dual-slope, pitched, or mono-slope roof.

Revise as follows:

2302.1 Definitions. For the purposes of this chapter, and as used elsewhere in this code the following terms are defined in Chapter 2:

GABLE

2304.6 Exterior wall sheathing. ~~Except as provided for in Section 1405 for weatherboarding or where stucco construction that complies with Section 2510 is installed, enclosed buildings shall be sheathed with one of the materials of the nominal thickness specified in Table 2304.6 or any other *approved* material of equivalent strength or durability~~ Wall sheathing on the outside of exterior walls, including gables, and the connection of sheathing to framing shall be designed in accordance with the general provisions of this code and shall be capable of resisting wind pressures in accordance with Section 1609..

2304.6.1 Wood structural panel sheathing. Where wood structural panel sheathing is used as the exposed finish on the outside of exterior walls, it shall have an exterior exposure durability classification. Where wood structural panel sheathing is used elsewhere, but not as the exposed finish, it shall be of a type manufactured with exterior glue (Exposure 1 or Exterior). ~~Wood structural panel wall sheathing or siding used as structural sheathing shall be capable of resisting wind pressures in accordance with Section 1609. Maximum wind speeds for wood~~ Wood structural panel sheathing ~~used to resist wind pressures~~, connections, and framing spacing shall be in accordance with Table 2304.6.1 for the applicable wind speed and exposure category when used with enclosed buildings with a mean roof height not greater than 30 feet (9144 mm) and a topographic factor (K_{zt}) of 1.0.

~~**2304.6.2**~~ **2304.7 Interior paneling.** Softwood wood structural panels used for interior paneling shall conform to the provisions of Chapter 8 and shall be installed in accordance with Table 2304.9.1. Panels shall comply with DOC PS 1, DOC PS 2 or ANSI/APA PRP 210. Prefinished hardboard paneling shall meet the requirements of CPA/ANSI A135.5. Hardwood plywood shall conform to HPVA HP-1.

2308.9.3 Exterior wall sheathing. Except where stucco construction that complies with Section 2510 is installed, the outside of exterior walls, including gables, of enclosed buildings shall be sheathed with one of the materials of the nominal thickness specified in Table 2308.9.3. with fasteners in accordance with requirements of 2304.9 or fasteners designed in accordance with accepted engineering practice.

TABLE ~~2304.6~~ 2308.9.3
MINIMUM THICKNESS OF WALL SHEATHING

SHEATING TYPE	MINIMUM THICKNESS	MAXIMUM WALL STUD SPACING
Wood boards	5/8 inch	24 inches on center
Fiberboard	1/2 inch	16 inches on center
Wood structural panel	In accordance with Tables 2308.9.3(2) and 2308.9.3(3)	--
M-S “Exterior Glue” and M-2 “Exterior Glue” Particleboard	In accordance with Section 2306.3 and Table 2308.9.3(4)	--
Gypsum sheathing	½ inch	16 inches on center
~~Gypsum wallboard~~	~~½ inch~~	~~24 inches on center~~
Reinforced cement mortar	1 inch	24 inches on center

For SI: 1 inch = 25.4 mm.

Reason: (2308.9.3) This new section comes from existing Section 2304.6. The content of the current section is moved to 2308.9.3 because it contains prescriptive minimum sheathings more suitable for wind speeds in accordance with limitations of 2308. The section is clarified as being applicable to exterior wall sheathing. The term “gable” is included to clarify that exterior wall sheathing recommendations are equally applicable to the gable.

Table 2304.6 is moved and renumbered as Table 2308.9.3. Gypsum wallboard is removed from the table to make it clear the table applies to exterior wall sheathing, in accordance with the proposed Section 2308.9.3.

Section 2304.6 is rewritten to establish minimum structural performance requirements and clarify that wall sheathing on the outside of exterior walls, as well as connection of sheathing to framing, must be capable of resisting wind pressures in accordance with Section 1609. The term “gable” is included to clarify that exterior wall sheathing recommendations for out of plane wind resistance are equally applicable to the gable.

Revisions to 2304.6.1 coordinate with the minimum structural performance requirements added in the new 2304.6. Prior language covering design for out of plane wind resistance is deleted because it is addressed in new section 2304.6. Reference to Table 2304.6.1 is revised to clarify that several factors are critical for determination of the applicable maximum wind speed including fastener schedule and stud spacing.

This renumbers Section 2304.6.2 to 2304.7 to separate provisions for Interior Paneling from 2306.6 which would contain new provisions applicable to exterior wall sheathing but not to interior paneling.

A definition is added for “gable” used in proposed revisions in Item #1 and #2 to clarify that gables should be sheathed in accordance with provisions for walls.

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

2308.9.3 (NEW)-S-COATS.doc

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

2308.9.3 Exterior wall sheathing. Except where stucco construction that complies with Section 2510 is installed, the outside of exterior walls, including gables, of enclosed buildings shall be sheathed with one of the materials of the nominal thickness specified in Table 2308.9.3 with fasteners in accordance with requirements of 2304.9 or fasteners designed in accordance with accepted engineering practice. Alternatively, sheathing materials and fasteners complying with Section 2304.6 shall be permitted.

(Portions of proposal not shown are unchanged)

Committee Reason: This change clarifies the details of exterior wall sheathing. The modification recognizes that sheathing meeting the performance requirements should be a permitted alternative.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Gary J. Ehrlich, P.E., National Association of Home Builders (NAHB), requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

SECTION 202
DEFINITIONS

GABLE. The triangular portion of a ~~the~~ wall beneath the end of a dual-slope, pitched, or mono-slope roof or portion thereof and above the top plates of the story or level of the ceiling below.

2308.9.3 Exterior wall sheathing. Except where stucco construction that complies with Section 2510 is installed, the outside of exterior walls, including gables, of enclosed buildings shall be sheathed with one of the materials of the nominal thickness specified in Table 2308.9.3 with fasteners in accordance with requirements of 2304.9 or fasteners designed in accordance with accepted engineering practice. Alternatively, sheathing materials and fasteners complying with Section 2304.6 shall be permitted.

TABLE 2308.9.3
MINIMUM THICKNESS OF WALL SHEATHING

SHEATHING TYPE	MINIMUM THICKNESS	MAXIMUM WALL STUD SPACING
Diagonal w~~W~~ood boards	5/8 inch	24 inches on center
Structural f~~F~~iberboard	1/2 inch	16 inches on center
Wood structural panel	In accordance with Tables 2308.9.3(2) and 2308.9.3(3)	--
M-S “Exterior Glue” and M-2 “Exterior Glue” Particleboard	In accordance with Section 2306.3 and Table 2308.9.3(4)	--
Gypsum sheathing	½ inch	16 inches on center
Reinforced cement mortar	1 inch	24 inches on center
Hardboard panel siding	In accordance with Tables 2308.9.3(5)	--

(portions of proposal not shown remain unchanged)

Commenter’s Reason: The purpose of this public comment is to address NAHB’s concerns as identified in our testimony against the proposal.

Under Section 2308.9.3, Item #5, gypsum board sheathing a minimum of ½ inch in thickness used as bracing is permitted when attached to studs up to 24 inches on center. Table 2308.9.3(1) limits the stud spacing to 16” for the bottom story of a 3-story house in Seismic Design Category A and B, and the bottom of a two-story house in Seismic Design Category C, but otherwise 24” in spacing is permitted. There is no limit under Table 2308.12.4 for gypsum sheathing used as bracing on a one-story house in Seismic Design Categories D and E.

The definition of Gable is flawed. As written, it is not clear the end of a gambrel (“barn-shaped”) roof on a Dutch Colonial house is a gable. The shape of the end wall is actually an irregular pentagon, not a triangle. Also, if the structure is balloon-framed, the definition technically makes the entire end wall of the structure a “gable”, even the portions of the wall that are associated with the story or stories below.

Finally, Table 2308.9.3 needs to be revised to reflect all of the permitted structural sheathing methods on exterior walls and coordinate with the revisions to the wall bracing portions of Section 2308 under S273.

S287-12
Final Action: AS AM AMPC____ D

S288-12
2308.9.3

Proposed Change as Submitted

Proponent: Paul Coats, P.E., CBO, American Wood Council (pcoats@awc.org)

Revise as follows:

2308.9.3 Bracing. Braced wall lines shall consist of braced wall panels that meet the requirements for location, type and amount of bracing as shown in Figure 2308.9.3, specified in Table 2308.9.3(1) and are in line or offset from each other by not more than 4 feet (1219 mm). Braced wall panels shall start not more than 12$^1/_2$ feet (3810 mm) from each end of a braced wall line. Braced wall panels shall be clearly indicated on the plans. Construction of braced wall panels shall be by one of the following methods:

1. Nominal 1-inch by 4-inch (25 mm by 102 mm) continuous diagonal braces let into top and bottom plates and intervening studs, placed at an angle not more than 60 degrees (1.0 rad) or less than 45 degrees (0.79 rad) from the horizontal and attached to the framing in conformance with Table 2304.9.1.
2. Wood boards of $^5/_8$ inch (15.9 mm) net minimum thickness applied diagonally on studs spaced not over 24 inches (610 mm) o.c.
3. Wood structural panel sheathing with a thickness not less than $^3/_8$ inch (9.5 mm) for 16-inch (406 mm) or 24-inch (610 mm) stud spacing in accordance with Tables 2308.9.3(2) and 2308.9.3(3).
4. Fiberboard sheathing panels not less than $^1/_2$ inch (12.7 mm) thick applied vertically or horizontally on studs spaced not over 16 inches (406 mm) o.c. where installed with fasteners in accordance with ~~Section 2306.6 and Table 2306.6~~ <u>Table 2304.9.1</u>.
5. Gypsum board [sheathing $^1/_2$-inch-thick (12.7 mm) by 4-feet-wide (1219 mm) wallboard or veneer base] on studs spaced not over 24 inches (610 mm) o.c. and nailed at 7 inches (178 mm) o.c. ~~with nails as required by Table 2306.7~~ <u>along panel edges (including top and bottom plates) and 7" o.c. in the field with 5d (0.086 inch diameter) cooler nails</u>.
6. Particleboard wall sheathing panels where installed in accordance with Table 2308.9.3(4).
7. Portland cement plaster on studs spaced 16 inches (406 mm) o.c.installed in accordance with Section 2510.
8. Hardboard panel siding where installed in accordance with Section 2303.1.6 and Table 2308.9.3(5).

For cripple wall bracing, see Section 2308.9.4.1. For Methods 2, 3, 4, 6, 7 and 8, each panel must be at least 48 inches (1219 mm) in length, covering three stud spaces where studs are spaced 16 inches (406 mm) apart and covering two stud spaces where studs are spaced 24 inches (610 mm) apart.

For Method 5, each panel must be at least 96 inches (2438 mm) in length where applied to one face of a panel and 48 inches (1219 mm) where applied to both faces. All vertical joints of panel sheathing shall occur over studs and adjacent panel joints shall be nailed to common framing members. Horizontal joints shall occur over blocking or other framing equal in size to the studding except where waived by the installation requirements for the specific sheathing materials. Sole plates shall be nailed to the floor framing and top plates shall be connected to the framing above in accordance with Section 2308.3.2. Where joists are perpendicular to braced wall lines above, blocking shall be provided under and in line with the braced wall panels.

Reason: In the 2012 code, some provisions for fasteners in Chapter 23 were removed and the AF&PA Special Design Provisions for Wind and Seismic was referenced instead. This proposed change cleans up some references to tables that are no longer applicable, while retaining prescriptive guidance in the code for conventional wall bracing methods. For fiberboard sheathing attachment, Section 2306.6 and Table 2306.6 are no longer applicable. In the 2012 IBC, Table 2304.9.1 would be an appropriate reference for fastener size for attachment of fiberboard sheathing. Table 2306.7 is no longer the correct reference in the 2012 IBC for gypsum wallboard attachment. The appropriate fastener, 5d cooler nails, is proposed for consistency with Table 2308.12.4 which addresses nail size for gypsum wallboard bracing used in Seismic Design Category D and E.

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

2308.9.3-S-COATS.doc

Public Hearing Results

Committee Action: Approved as Submitted

Committee Reason: Agreement with the proponent's reason which indicates that the changes clean up references to tables that are no longer appropriate. It also coordinates the bracing requirements with other code sections on gypsum board.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Paul D. Coats, P.E., CBO, American Wood Council, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

2308.9.3 Bracing. Braced wall lines shall consist of braced wall panels that meet the requirements for location, type and amount of bracing as shown in Figure 2308.9.3, specified in Table 2308.9.3(1) and are in line or offset from each other by not more than 4 feet (1219 mm). Braced wall panels shall start not more than 121/2 feet (3810 mm) from each end of a braced wall line. Braced wall panels shall be clearly indicated on the plans. Construction of braced wall panels shall be by one of the following methods:

1. Nominal 1-inch by 4-inch (25 mm by 102 mm) continuous diagonal braces let into top and bottom plates and intervening studs, placed at an angle not more than 60 degrees (1.0 rad) or less than 45 degrees (0.79 rad) from the horizontal and attached to the framing in conformance with Table 2304.9.1.
2. Wood boards of 5/8 inch (15.9 mm) net minimum thickness applied diagonally on studs spaced not over 24 inches (610 mm) o.c.
3. Wood structural panel sheathing with a thickness not less than 3/8 inch (9.5 mm) for 16-inch (406 mm) or 24-inch (610 mm) stud spacing in accordance with Tables 2308.9.3(2) and 2308.9.3(3).
4. Fiberboard sheathing panels not less than 1/2 inch (12.7 mm) thick applied vertically or horizontally on studs spaced not over 16 inches (406 mm) o.c. where installed with fasteners in accordance with Table 2304.9.1.
5. Gypsum board [sheathing 1/2-inch-thick (12.7 mm) or 5/8-inch thick (15.9 mm) by 4-feet-wide (1219 mm) wallboard or veneer base] on studs spaced not over 24 inches (610 mm) o.c. and ~~nailed~~ fastened to studs at 7 inches (178 mm) o.c. ~~along panel edges (including top and bottom plates) and 7" o.c. in the field with 5d (0.086 inch diameter) cooler nails.~~ in the field of the board and at board edges with nails or screws complying with Section 2506.2. Nails shall be 5d annular ringed (1 5/8 inch x 0.086 inch diameter) cooler nails and screws shall be not less than 1 ¼ inches in length.
6. Particleboard wall sheathing panels where installed in accordance with Table 2308.9.3(4).
7. Portland cement plaster on studs spaced 16 inches (406 mm) o.c. installed in accordance with Section 2510.
8. Hardboard panel siding where installed in accordance with Section 2303.1.6 and Table 2308.9.3(5).

For cripple wall bracing, see Section 2308.9.4.1. For Methods 2, 3, 4, 6, 7 and 8, each panel must be at least 48 inches (1219 mm) in length, covering three stud spaces where studs are spaced 16 inches (406 mm) apart and covering two stud spaces where studs are spaced 24 inches (610 mm) apart.

For Method 5, each panel must be at least 96 inches (2438 mm) in length where applied to one face of a panel and 48 inches (1219 mm) where applied to both faces. All vertical joints of panel sheathing shall occur over studs and adjacent panel joints shall be nailed to common framing members. Horizontal joints shall occur over blocking or other framing equal in size to the studding except where waived by the installation requirements for the specific sheathing materials. Sole plates shall be nailed to the floor framing and top plates shall be connected to the framing above in accordance with Section 2308.3.2. Where joists are perpendicular to braced wall lines above, blocking shall be provided under and in line with the braced wall panels.

Commenter's Reason: This comment combines the original changes to item 5 made by S288 and S289 which were both approved by the Structural Committee, and is necessary for the clear wording of the section resulting from both approvals. It also adds the correct nail length (1 5/8-inch) for the 5d cooler nail. Screw dimensions other than the minimum length are specified in the standards listed in Section 2506.2 for screws used with gypsum.

S288-12

Final Action: AS AM AMPC____ D

S291-12
2308.9.3.2, Figure 2308.9.3.2

Proposed Change as Submitted

Proponent: Edward L. Keith, P.E., APA – The Engineered Wood Association (ed.keith@apawood.org)

Revise as follows:

2308.9.3.2 Alternate bracing wall panel adjacent to a door or window opening. Any bracing required by Section 2308.9.3 is permitted to be replaced by the following when used adjacent to a door or window opening with a full-length header:

1. In one-story buildings, each panel shall have a length of not less than 16 inches (406 mm) and a height of not more than 10 feet (3048 mm). Each panel shall be sheathed on one face with a single layer of 3/8 inch (9.5 mm) minimum thickness wood structural panel sheathing nailed with 8d common or galvanized box nails in accordance with Figure 2308.9.3.2. The wood structural panel sheathing shall extend up over the solid sawn or glued-laminated header and shall be nailed in accordance with Figure 2308.9.3.2. A built-up header consisting of at least two 2 × 12s and fastened in accordance with Item 24 of Table 2304.9.1 shall be permitted to be used. A spacer, if used, shall be placed on the side of the built-up beam opposite the wood structural panel sheathing. The header shall extend between the inside faces of the first full-length outer studs of each panel. The clear span of the header between the inner studs of each panel shall be not less than 6 feet (1829 mm) and not more than 18 feet (5486 mm) in length. A strap with an uplift capacity of not less than 1,000 pounds (4,400 N) shall fasten the header to the inner studs opposite the sheathing. One anchor bolt not less than 5/8 inch (15.9 mm) diameter and installed in accordance with Section 2308.6 shall be provided in the center of ach sill plate. The studs at each end of the panel shall have a tie-down device fastened to the foundation with an uplift capacity of not less than ~~4,200~~ <u>3,500</u> pounds (~~18 480~~ <u>15 570</u> N).

   Where a panel is located on one side of the opening, the header shall extend between the inside face of the first full-length stud of the panel and the bearing studs at the other end of the opening. A strap with an uplift capacity of not less than 1,000 pounds (4400 N) shall fasten the header to the bearing studs. The bearing studs shall also have a tie-down device fastened to the foundation with an uplift capacity of not less than 1,000 pounds (4400 N).

   The tie-down devices shall be an embedded strap type, installed in accordance with the manufacturer's recommendations. The panels shall be supported directly on a foundation that is continuous across the entire length of the braced wall line. This foundation shall be reinforced with not less than one No. 4 bar top and bottom.

   Where the continuous foundation is required to have a depth greater than 12 inches (305 mm), a minimum 12-inch by 12-inch (305 mm by 305 mm) continuous footing or turned down slab edge is permitted at door openings in the braced wall line. This continuous footing or turned down slab edge shall be reinforced with not less than one No. 4 bar top and bottom. This reinforcement shall be lapped not less than 15 inches (381 mm) with the reinforcement required in the continuous foundation located directly under the braced wall line.

2. In the first *story* of two-story buildings, each wall panel shall be braced in accordance with Item 1 bove, except that each panel shall have a length of not less than 24 inches (610 mm).

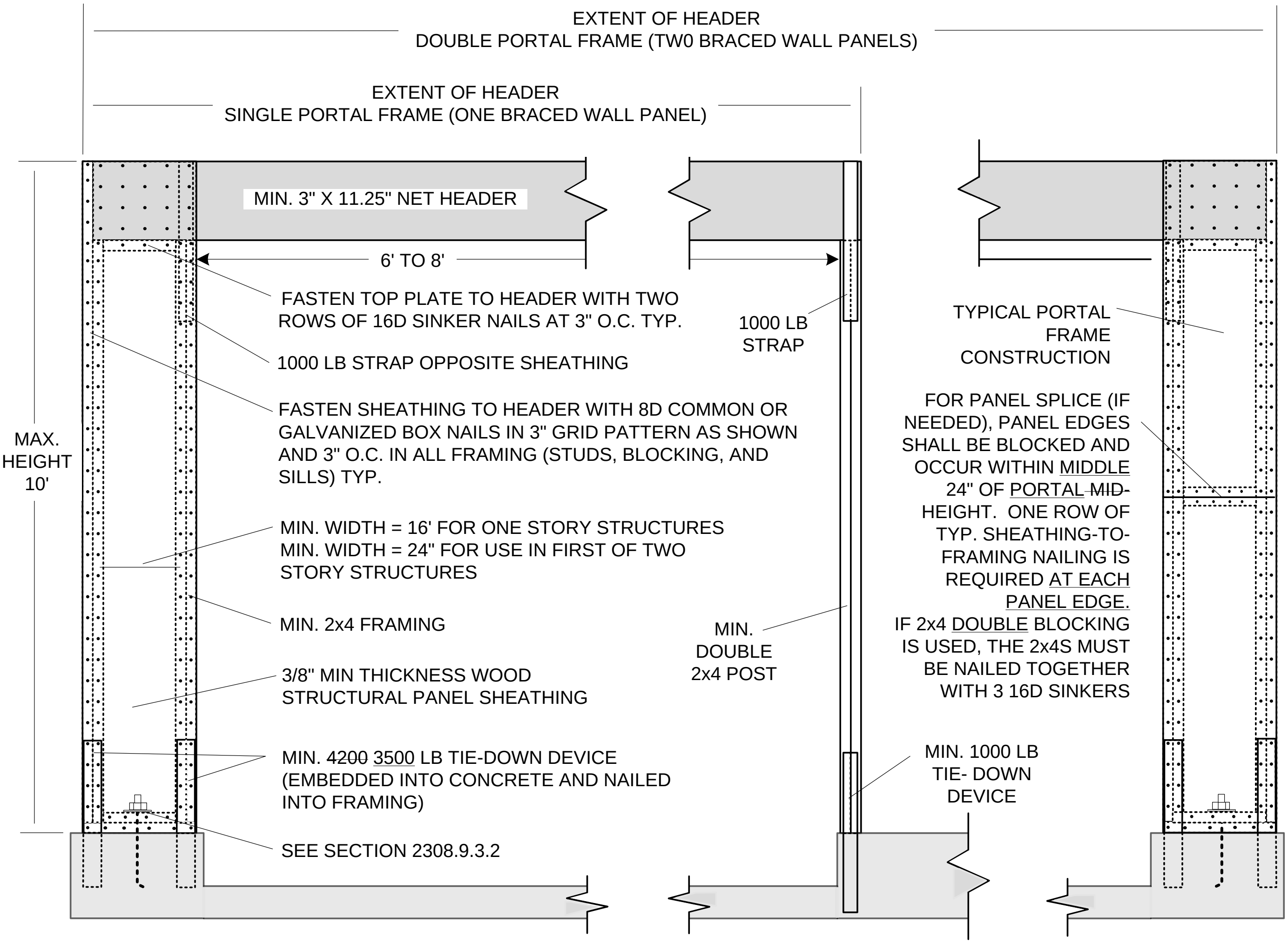


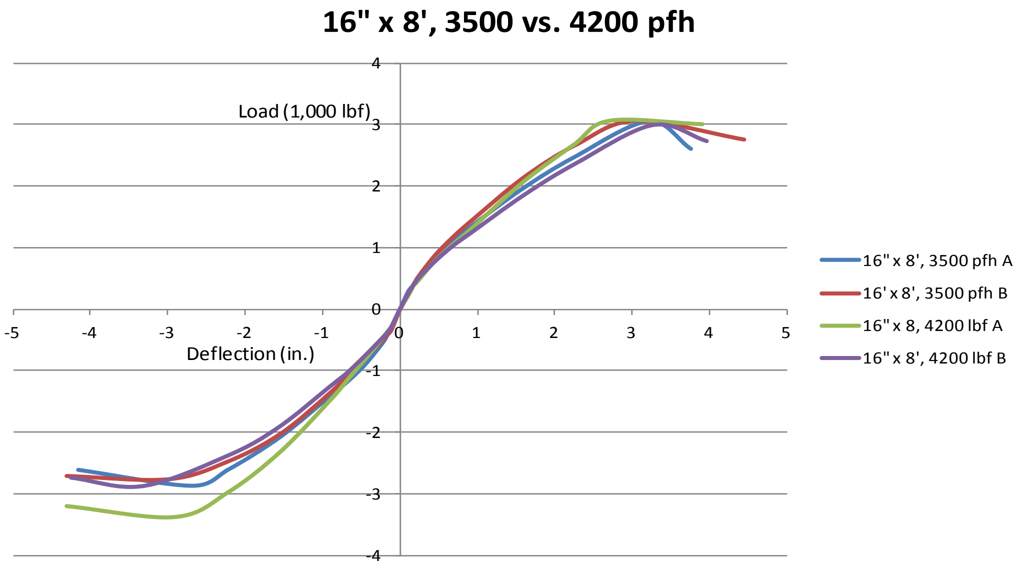
FIGURE 2308.9.3.2
ALTERNATE BRACED WALL PANEL ADJACENT TO A DOOR OR WINDOW OPENING

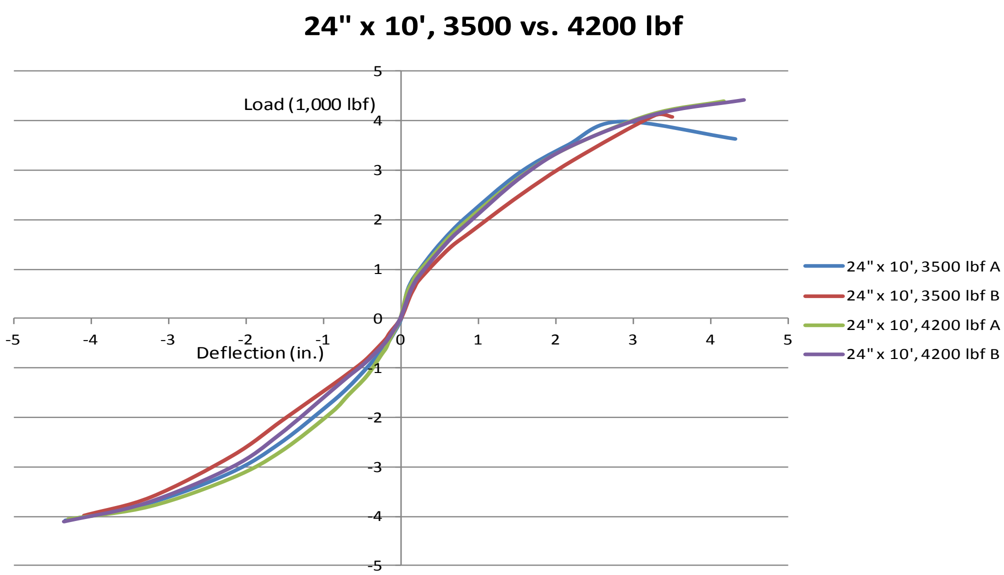
Reason: 1) There are a couple of types of changes to Figure 2308.9.3.2 proposed. There are both technical changes and editorial changes.

Technical changes: The two technical changes made to the figure are the reduction of the capacity of the portal frame leg tie-down devices from 4200 lbf to 3500 lbf and the removal of the third bottom plate at the portal frame leg. (Note that the third bottom plate we propose to delete is NOT shown in the figure above. The normal strikethrough and underline procedures are difficult to apply to figure changes.)

A. The first technical change is the reduction of the tie-down from 4200 lbf to 3500 lbf. The initial testing was conducted on the portal frames utilizing the 4200 lbf hold down because that was what was readily available and in common use by the construction industry. At the time of initial testing, no attempt was made to determine the sensitivity of the system to such a reduction in tie-down capacity. As the initial prescriptive parameters of the portal frame were based on testing, there was no latitude for determining the impact of the industry wide reduction to such tie-downs in response to the cracked-concrete provisions of ACI 318. As such, retesting of the portal frames with both 4200 lbf and 3500 lbf tie-downs was necessary to determine the impact on the performance of the system, if any. Portals with 16” wide legs x 8 ft height as well as 24” wide x 10 ft high were recently retested by APA. Pairs of each size were tested with 4200 lbf tie-downs and then retested with 3500 lbf tie-downs. The results of these tests showed that the system was relatively insensitive to the reduction in tie-down capacity from 4200 lbf to 3500 lbf. No attempt was made to determine how low the tie-down capacity could be reduced before an impact on the performance of the portal frames could be seen.

These tests were conducted using the CUREe method, as described in ASTM E2126, with a frequency of 0.5 Hz. The following charts show the backbone curves for the Method PFH portal frames tested with 3500 lbf and 4200 lbf tie-downs at both the 16” wide leg portals 8’ high as well as the 24” wide portals 10’ high.





Free PDF Copies of the full lab report on this testing program entitled APA Report T2011-15, *Bracing Method PFH (Portal Frame with Hold down) – Alternative Attachment,* can be obtained at http://www.apawood.org.

B. The second technical change is the removal of the third bottom plate. As mentioned above the original testing was conducted with the third plate in place. The third plate causes numerous difficulties in the field, not the least of which is that the normal length threaded anchors are too short to accommodate the third plate and provide the required depth of penetration into the foundation. This results in inadequate anchor depth-of-embedment or the use of threaded sleeves and all-thread to extend the bolt length to accommodate the third plate. When investigating the change to the 3500 lbf hold down, we utilized this opportunity to run the tests with only double bottom plates. All subsequent testing was done without the third bottom plate. The results of this testing indicated that the third bottom plate has negligible impact on the performance of the portal frames.

<u>Non-technical changes:</u>

1. The intent of the note concerning the location of the portal-leg sheathing-splice, when present, is to place the splice butt joint within the middle 24" of the portal frame height. As currently written "within 24" of mid height" means the splice could be placed within 24 inches either above or below of mid height, or within a band 48" wide. This was never the intent. The proposed language is clearer that the joint must "occur within the middle 24" of portal height", where portal height is illustrated in the figure.
2. At the splice plate, the current wording requires a single row of nailing. The proposed change required this at <u>each panel edge</u> at the splice as was the original intent.
3. In the same annotation, a provision is provided that would permit the splice to be made over a pair of 2x4s as long as they are spliced together. The proposal changes "blocking" to "double blocking" to clarify the intent.

2) The revision to Section 2308.9.3.2 is as explained above.

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

F2308.9.3.2-S-KEITH.doc

Public Hearing Results

Modify proposal as follows:

2308.9.3.2 Alternate bracing wall panel adjacent to a door or window opening. Any bracing required by Section 2308.9.3 is permitted to be replaced by the following when used adjacent to a door or window opening with a full-length header:

1. In one-story buildings, each panel shall have a length of not less than 16 inches (406 mm) and a height of not more than 10 feet (3048 mm). Each panel shall be sheathed on one face with a single layer of 3/8 inch (9.5 mm) minimum thickness wood structural panel sheathing nailed with 8d common or galvanized box nails in accordance with Figure 2308.9.3.2. The wood structural panel sheathing shall extend up over the solid sawn or glued-laminated header and shall be nailed in accordance with Figure 2308.9.3.2. A built-up header consisting of at least two 2 × 12s and fastened in accordance with Item 24 of Table 2304.9.1 shall be permitted to be used. A spacer, if used, shall be placed on the side of the built-up beam opposite the wood structural panel sheathing. The header shall extend between the inside faces of the first full-length outer studs of each panel. The clear span of the header between the inner studs of each panel shall be not less than 6 feet (1829 mm) and not more than 18 feet (5486 mm) in length. A strap with an uplift capacity of not less than 1,000 pounds (4,400 N) shall fasten the header to the inner studs opposite the sheathing. One anchor bolt not less than 5/8 inch (15.9 mm) diameter and installed in accordance with Section 2308.6 shall be provided in the center of ach sill plate. The studs at each end of the panel shall have a tie-down device fastened to the foundation with an uplift capacity of not less than <u>4,200</u> ~~3,500~~ pounds (<u>18 480</u> ~~15 570~~ N).

   Where a panel is located on one side of the opening, the header shall extend between the inside face of the first full-length stud of the panel and the bearing studs at the other end of the opening. A strap with an uplift capacity of not less than 1,000 pounds (4400 N) shall fasten the header to the bearing studs. The bearing studs shall also have a tie-down device fastened to the foundation with an uplift capacity of not less than 1,000 pounds (4400 N).

   The tie-down devices shall be an embedded strap type, installed in accordance with the manufacturer's recommendations. The panels shall be supported directly on a foundation that is continuous across the entire length of the braced wall line. This foundation shall be reinforced with not less than one No. 4 bar top and bottom.

   Where the continuous foundation is required to have a depth greater than 12 inches (305 mm), a minimum 12-inch by 12-inch (305 mm by 305 mm) continuous footing or turned down slab edge is permitted at door openings in the braced wall line. This continuous footing or turned down slab edge shall be reinforced with not less than one No. 4 bar top and bottom. This reinforcement shall be lapped not less than 15 inches (381 mm) with the reinforcement required in the continuous foundation located directly under the braced wall line.

2. In the first *story* of two-story buildings, each wall panel shall be braced in accordance with Item 1 above, except that each panel shall have a length of not less than 24 inches (610 mm).

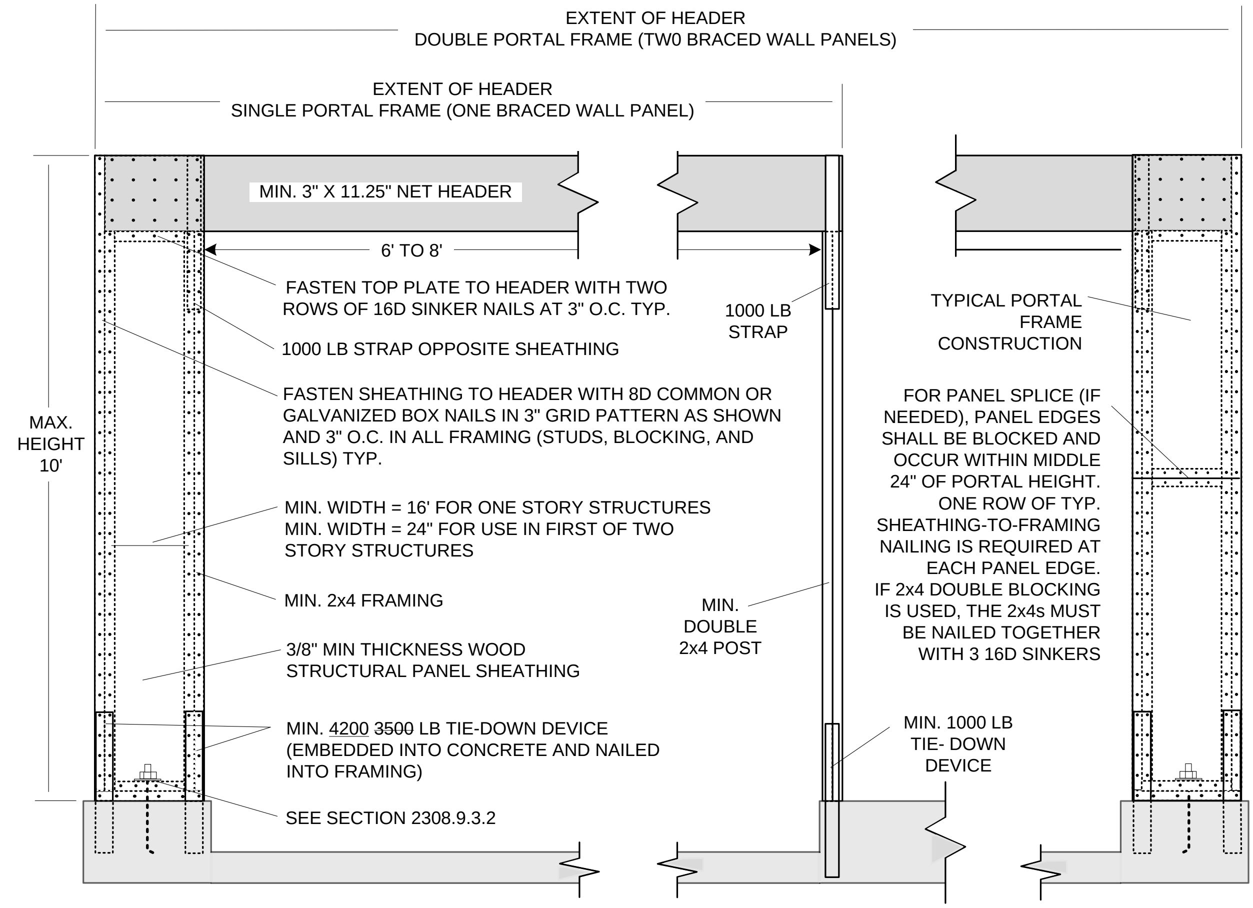


FIGURE 2308.9.3.2
ALTERNATE BRACED WALL PANEL ADJACENT TO A DOOR OR WINDOW OPENING

Committee Reason: This proposal updates the prescriptive portal frame bracing alternative. The modification is acknowledges that the hold-down capacity needs to remain 4200 pounds.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Edward L. Keith, representing APA – The Engineered Wood Association, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

2308.9.3.2 Alternate bracing wall panel adjacent to a door or window opening. Any bracing required by Section 2308.9.3 is permitted to be replaced by the following when used adjacent to a door or window opening with a full-length header:

1. In one-story buildings, each panel shall have a length of not less than 16 inches (406 mm) and a height of not more than 10 feet (3048 mm). Each panel shall be sheathed on one face with a single layer of 3/8 inch (9.5 mm) minimum thickness wood structural panel sheathing nailed with 8d common or galvanized box nails in accordance with Figure 2308.9.3.2. The wood structural panel sheathing shall extend up over the solid sawn or glued-laminated header and shall be nailed in accordance with Figure 2308.9.3.2. A built-up header consisting of at least two 2 × 12s and fastened in accordance with Item 24 of Table 2304.9.1 shall be permitted to be used. A spacer, if used, shall be placed on the side of the built-up beam opposite the wood structural panel sheathing. The header shall extend between the inside faces of the first full-

length outer studs of each panel. The clear span of the header between the inner studs of each panel shall be not less than 6 feet (1829 mm) and not more than 18 feet (5486 mm) in length. A strap with an uplift capacity of not less than 1,000 pounds (4,400 N) shall fasten the header to the inner studs opposite the sheathing. One anchor bolt not less than 5/8 inch (15.9 mm) diameter and installed in accordance with Section 2308.6 shall be provided in the center of ach sill plate. The studs at each end of the panel shall have a tie-down device fastened to the foundation with an uplift capacity of not less than ~~4,200~~ 3,500 pounds (~~18 480~~ 15 570 N).

Where a panel is located on one side of the opening, the header shall extend between the inside face of the first full-length stud of the panel and the bearing studs at the other end of the opening. A strap with an uplift capacity of not less than 1,000 pounds (4400 N) shall fasten the header to the bearing studs. The bearing studs shall also have a tie-down device fastened to the foundation with an uplift capacity of not less than 1,000 pounds (4400 N).

The tie-down devices shall be an embedded strap type, installed in accordance with the manufacturer's recommendations. The panels shall be supported directly on a foundation that is continuous across the entire length of the braced wall line. This foundation shall be reinforced with not less than one No. 4 bar top and bottom.

Where the continuous foundation is required to have a depth greater than 12 inches (305 mm), a minimum 12-inch by 12-inch (305 mm by 305 mm) continuous footing or turned down slab edge is permitted at door openings in the braced wall line. This continuous footing or turned down slab edge shall be reinforced with not less than one No. 4 bar top and bottom. This reinforcement shall be lapped not less than 15 inches (381 mm) with the reinforcement required in the continuous foundation located directly under the braced wall line.

2. In the first *story* of two-story buildings, each wall panel shall be braced in accordance with Item 1 above, except that each panel shall have a length of not less than 24 inches (610 mm).

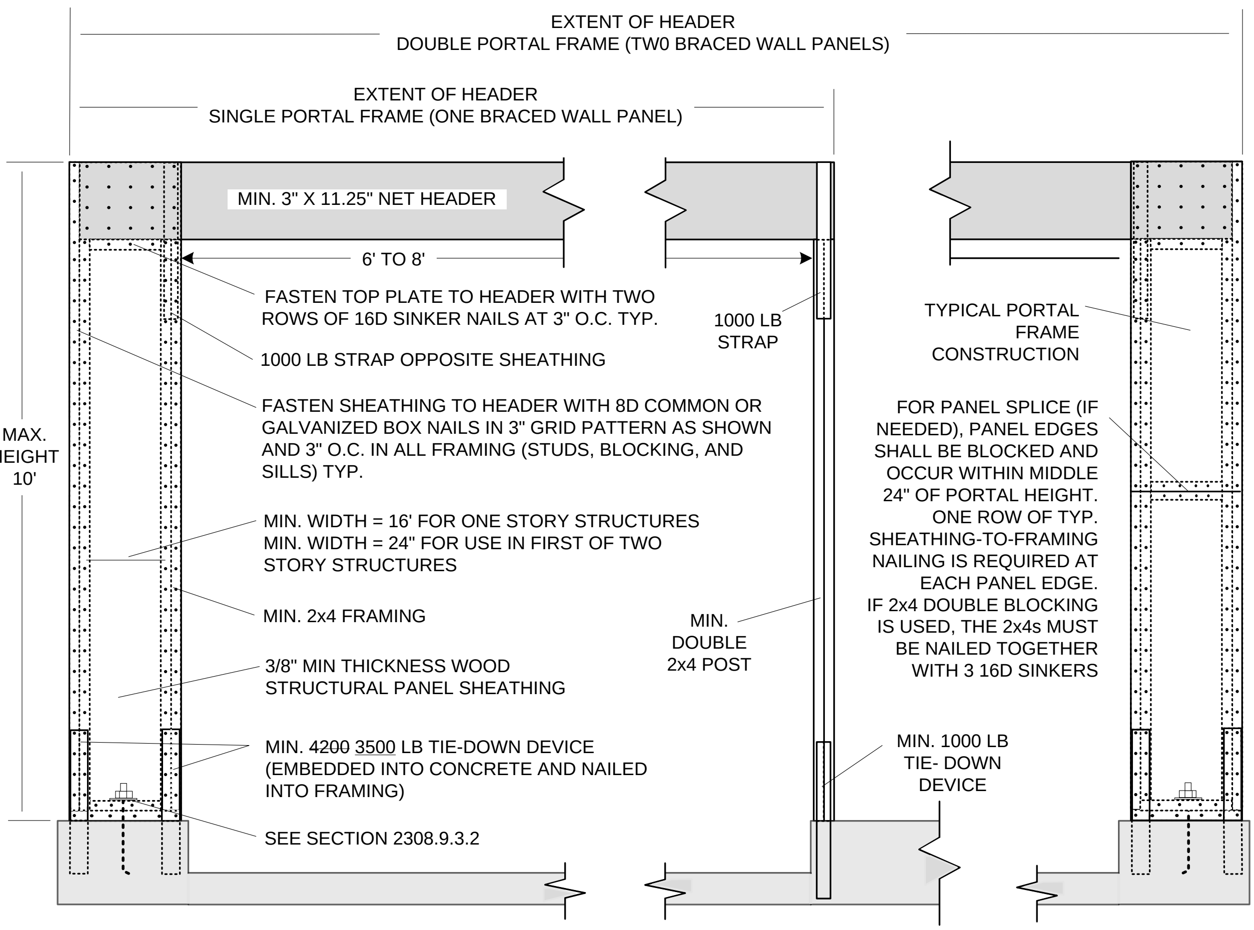
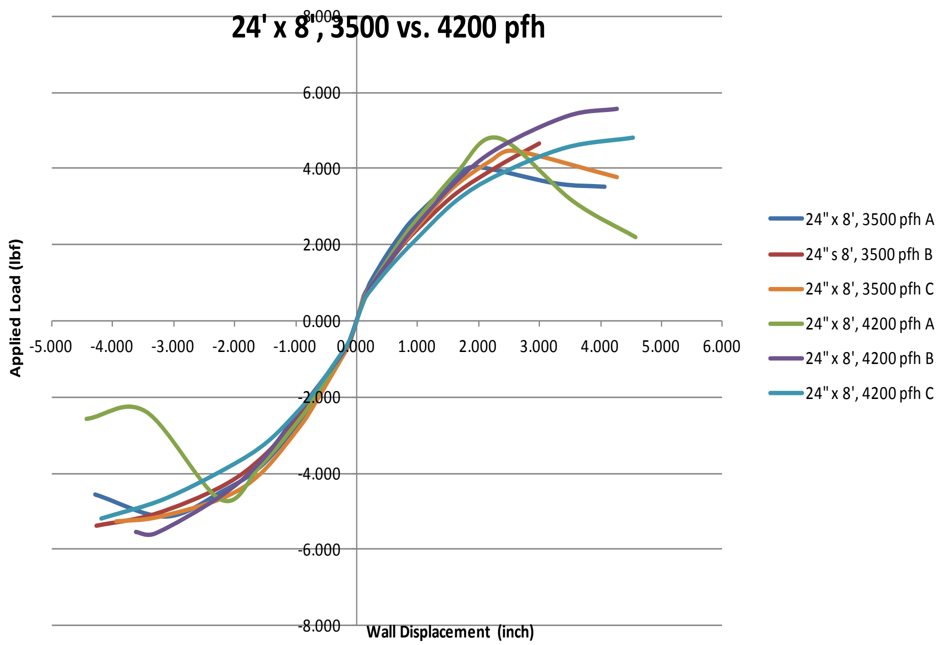


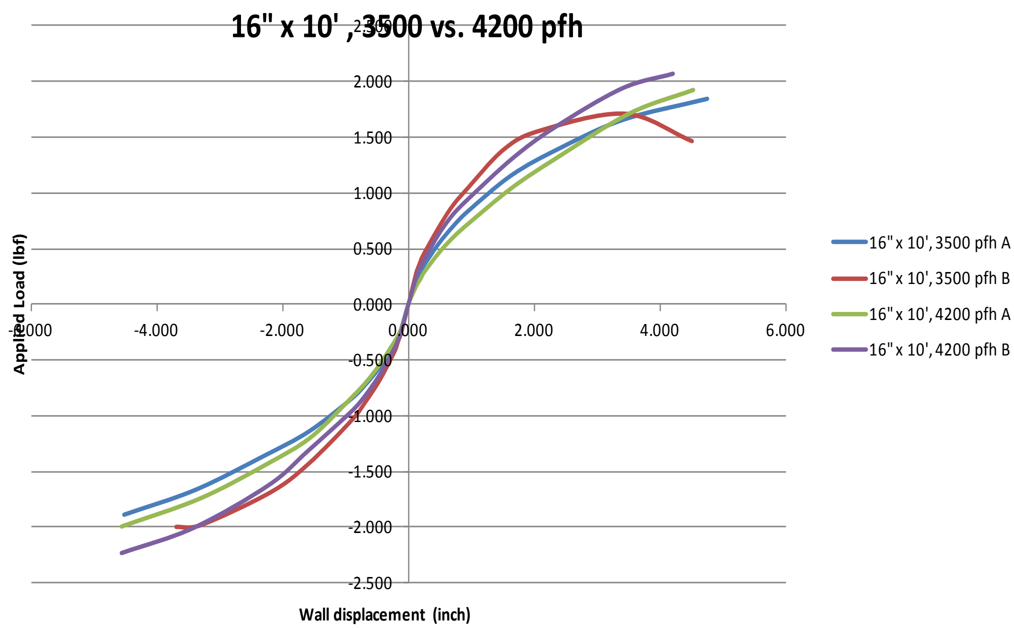
FIGURE 2308.9.3.2
ALTERNATE BRACED WALL PANELADJACENT TO A DOOR OR WINDOW OPENING

Commenter's Reason: The original proposal as submitted included a proposed change for the hold-down capacity in Section 2308.9.3.2 and Figure 2308.9.3.2 from 4,200 lbf to 3,500 lbf along with a number of other minor changes. This change in hold-down capacity was based on some preliminary research done at APA Research Center that was incomplete at the time the proposal was heard by the Committee. At the time, Simpson Strong-Tie was also in the process of developing a 4,200 lbf hold-down solution for such applications. Anticipating the success of Simpson's testing program, APA worked with Simpson to develop a floor modification to return the hold-down capacity to the original 4,200 lbf capacity. This floor modification was accepted and the original proposal was recommended for approval as modified.

Subsequent to the Code Development Hearings, those specific hold-down solutions that Simpson Strong-Tie was testing were unable to develop a 4,200 lbf hold-down capacity at a foundation end (immediately adjacent to a door opening). As there was insufficient time to develop and test additional hold-down solutions, APA, with assistance from Simpson's technical staff, completed a testing program verifying that reducing the recommended hold-down capacity from 4,200 to 3,500 lbf resulted in no appreciable difference in the performance of the hold down.

In addition to the geometries previously tested and reported in the original code change proposal, as reproduced below, two additional geometries were tested by APA. The results of these two geometries can be seen in the load deflection backbone curves shown below. The legends for the plots indicate the leg width of the portal x the height of the portal, the hold-down capacity, and replication letter.





It is clear from the plots below and those provided in the original reason statement, also provided below, that the reduction in the capacity of the hold-down strap from 4,200 to 3,500 lbf has no significant impact on the performance of the portal frame. As such, we request that by this public comment, the reference to the hold-down capacity be changed from 4,200 to 3,500 lbf in both the figure and corresponding text.

A free copy of *APA Report T2012L-24 - Alternative Attachment (IBC), Portal Frame with Hold Downs (Bracing Method PFH) (IRC) – Hold-Down Strap Capacity Variations* is available at http://www.apawood.org/pdfs/TSD/T-Reports/T2012L-24.pdf

S291-12

Final Action: AS AM AMPC____ D

S292-12
2308.11.3.3

Proposed Change as Submitted

Proponent: Robert Rice, C.B.O., Josephine County, OR, representing Oregon Building Officials Association (structdesigner@yahoo.com), R. Terry Malone, P.E., S.E.

Revise as follows:

2308.11.3.3 Openings in horizontal diaphragms. Horizontal diaphragms with openings having dimension perpendicular to the joist that is greater than 4 feet (1219 mm) shall be designed in accordance with accepted engineering practice. Openings in horizontal diaphragms with a dimension perpendicular to the joist that is not greater than 4 feet (1219 mm) shall be constructed ~~in accordance with the following:~~ with metal ties and blocking in accordance with this section and Figure 2308.11.3.3.

1. ~~Blocking shall be provided beyond headers.~~
2. ~~2.~~ Metal ties shall not be less than 0.058 inch thick [1.47 mm (16 galvanized gage)] by $1^1/_2$ inches (38 mm) wide and shall have a minimum yield strength of 33,000 psi (227 MPa). Blocking shall extend 2 feet minimum beyond headers. Ties shall be attached to blocking with eight 16d common nails on each side of the header-joist intersection ~~shall be provided (see Figure 2308.11.3.3). The metal ties shall have a minimum yield of 33,000 psi (227 MPa)~~.

Reason: This proposal re-arranges the existing text to read more clearly, corrects an error in the code and clarifies the requirements and limitations of openings in diaphragms in structures assigned to Seismic Design Category B, C, D and E. The text of the current code is intended to provide a prescriptive solution for diaphragm openings, in high seismic design categories, that are 4 feet or less. The current code is missing the word "not" which would make this section correct. The commentary for this code section correctly states,

Horizontal diaphragms are floor and roof assemblies that are usually clad with structural wood sheathing panels, such as plywood or OSB. Though more complicated and difficult to visualize, lateral forces that are applied to a building from wind or seismic events follow a load path that distributes and transfers shear and overturning forces from the lateral loads. When openings are built into the diaphragm, they disrupt the continuity of load across the diaphragm and they must be reinforced to compensate. Another concern is the stiffness of the diaphragm. These provisions are a prescriptive solution for openings not greater than 4 feet (1219 mm) in dimension and provide a general means for a load path in these specific cases in lieu of an engineered design.- 2009 IBC Commentary, International Code Council

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

2308.11.3.3-S-RICE.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: There is confusion over this proposal to revise the provision on openings in horizontal diaphragms and the source document [NEHRP] for this requirement. The committee would like to see better justification for this change.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment 1:

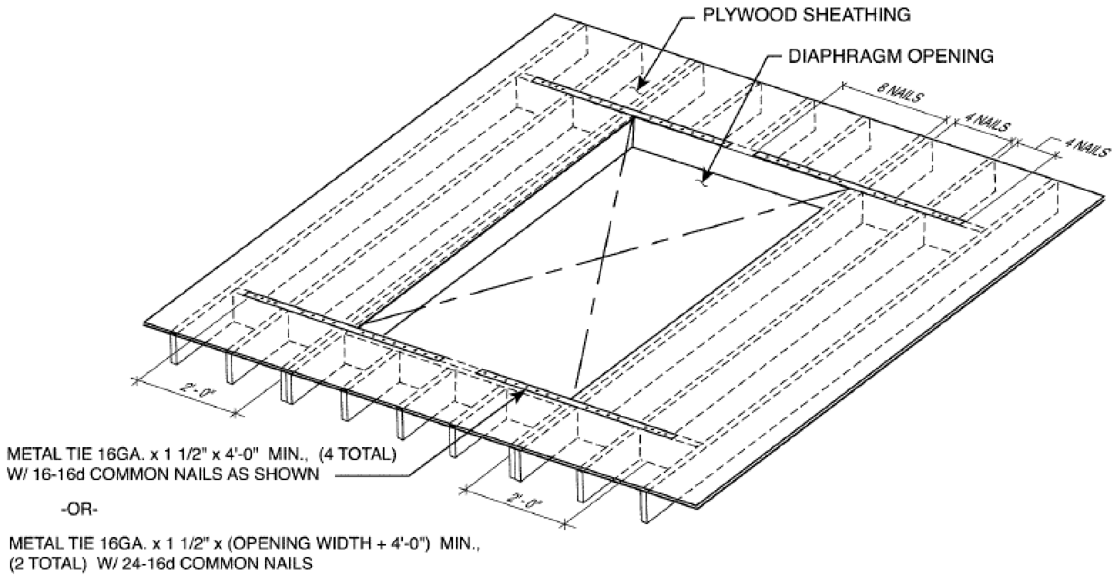
Robert Rice, Josephine County, OR, representing Oregon Building Officials Association and R. Terry Malone, P.E. S.E representing self, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

2308.11.3.3 Openings in horizontal diaphragms. ~~Horizontal diaphragms with openings having a dimension perpendicular to the joist that is greater than 4 feet (1219mm) shall be designed in accordance with accepted engineering practice.~~ Openings in horizontal diaphragms with a dimension perpendicular to the joist that is ~~not~~ greater than 4 feet (1219 mm) shall be constructed with metal ties and blocking in accordance with this section and Figure 2308.11.3.3.

Metal ties shall not be less than 0.058 inch thick [1.47 mm (16 galvanized gage)] by 11/2 inches (38 mm) wide and shall have a minimum yield strength of 33,000 psi (227 MPa). Blocking shall extend 2 feet minimum beyond headers. Ties shall be attached to blocking with eight 16d common nails on each side of the header-joist intersection.

Commenter's Reason: The original proposal attempted to accomplish two goals. First, the existing code language read poorly and necessary information for the treatment of openings was in the referenced figure, Figure 2308.11.3.3 as shown below, but not in the text. So, one goal of the original proposal was to rearrange the text for clarity to add the requirements from the figure into the text.
The second intent of the original proposal was to clarify that this prescriptive solution was originally intended for openings in higher SDC's up to four feet but not over four feet. This Public Comment is a proposed modification that deletes the change to applicability and addresses only the first goal in rearrangement of the text and including the information from the figure. This Public Comment modification does not add anything to the code that is not already there and removes the portion of the original proposal that would have resulted in a change.



For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm.

FIGURE 2308.11.3.3
OPENINGS IN HORIZONTAL DIAPHRAGMS

2012 INTERNATIONAL BUILDING CODE® **513**

Public Comment 2:

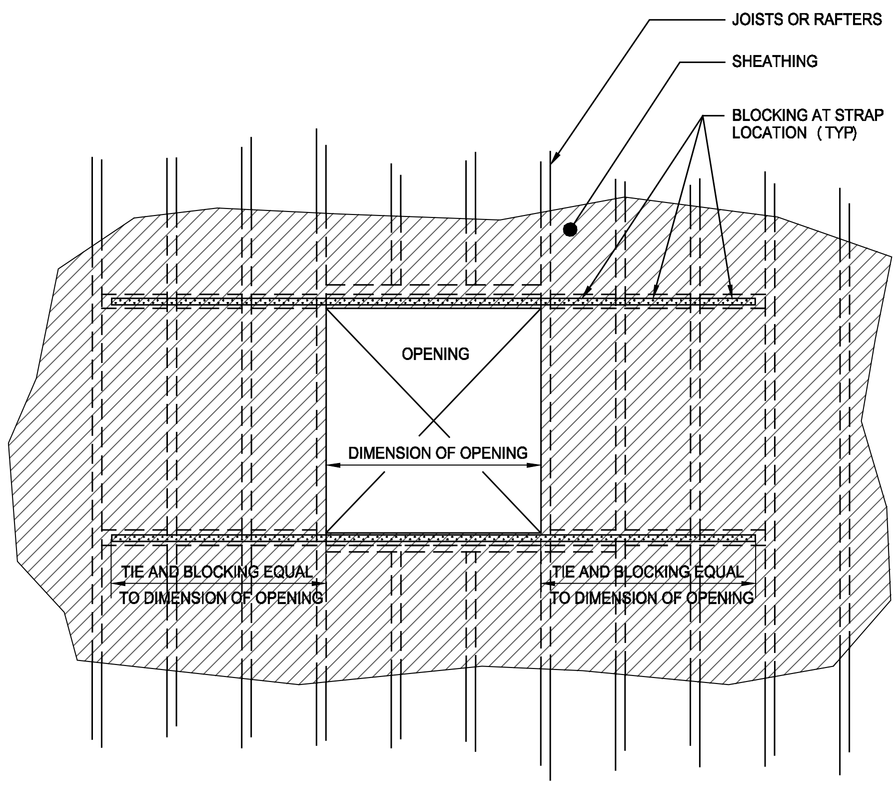
Robert Rice, Josephine County, OR, representing Oregon Building Officials Association and R. Terry Malone, P.E. S.E., representing self and Dr. J. Daniel Dolan, P.E representing self, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

2308.11.3.3 Openings in horizontal diaphragms. ~~Horizontal diaphragms with openings having a dimension perpendicular to the joist that is greater than 4 feet (1219 mm) shall be designed in accordance with accepted engineering practice.~~ Openings in horizontal diaphragms with a dimension ~~perpendicular to the joist~~ that is ~~not~~ greater than 4 feet (1219 mm) shall be constructed with metal ties and blocking in accordance with this section and Figure 2308.11.3.3.

Metal ties shall not be less than 0.058 inch thick [1.47 mm (16 galvanized gage)] by 1-1/2 inches (38 mm) wide and shall have a minimum yield strength of 33,000 psi (227 MPa). Blocking shall extend ~~2 feet minimum beyond headers~~ <u>not less than the dimension of the opening in the direction of the tie and blocking</u>. Ties shall be attached to blocking <u>in accordance with the manufacturer's instructions but with not less than</u> ~~with~~ eight 16d common nails on each side of the header-joist intersection.

Replace figure as shown:



<u>FIGURE 2308.11.3.3</u>
<u>OPENINGS IN HORIZONTAL DIAPHRAGMS</u>

Commenter's Reason: The original proposal attempted to accomplish two goals. First, the existing code language read poorly and the necessary information for the treatment of openings was in the referenced figure (Figure 2308.11.3.3) but not in the text. One goal of the original proposal was to rearrange the text for clarity to add the requirements from the figure into the text. A separate Public Comment has been submitted to address those portions of the original proposal.

The second intent of the original proposal was to clarify that this prescriptive solution was not incorporated into the IBC as originally intended by the supporting background reports and documents and was actually in direct conflict with the description in the IBC commentary. The current IBC text says that this prescriptive detail is to be used for openings **over** 4 feet in SDC B, C, D and E. The commentary says that the detail can be used as a prescriptive solution **up to 4 feet** in those SDC's. The language and implication of the commentary would require an engineered design when the opening is greater than 4 feet.

The commentary reads;

Horizontal diaphragms are floor and roof assemblies that are usually clad with structural wood sheathing panels, such as plywood or OSB. Though more complicated and difficult to visualize, lateral forces that are applied to a building from wind or seismic events follow a load path that distributes and transfers shear and overturning forces from the lateral loads. When openings are built into the diaphragm, they disrupt the continuity of load across the diaphragm and they must be reinforced to compensate. Another concern is the stiffness of the diaphragm. ***These provisions are a prescriptive solution for openings not greater than 4 feet (1219 mm) in dimension and provide a general means for a load path in these specific cases in lieu of an engineered design.****- 2009 IBC Commentary, International Code Council*

Despite efforts to research the source of this code provision, it wasn't until after submission of the proposal for the Code Development Hearings in Dallas that the background for this became more clear. In the 2003 NEHRP; The detail is required for openings greater than 4 feet in all SDC's and for **all openings in SDC D and E** .

The 2003 NEHRP provisions state;

"12.4.3.7 Detailing for openings in diaphragms. For openings with a dimension greater than 4 ft (1.2m), or openings in structures assigned to Seismic Design Category D or E, the following minimum detail shall be provided. Blocking beyond headers and metal ties not less than 0.058 in (16 gauge; 2 mm) thick by 1.5 in. (38 mm) wide by 48 in. (1220 mm) long with eight 16d (0.162 by 3.5 in.; 4 by 89 mm) common nails on each side of the header-joist intersection shall be provided (see Figure 12.4-11). Steel used shall have a minimum yield of 33,000 psi (228 MPa) such as ASTM A 653 SS, Grade 33, ASTM A 792 SS, Grade 33, or ASTM A 875 SS, Grade 33."

Further, APA Research Report 138 states,

"The forces generated by the opening may be calculated by applying the principles of statics. …..However, ***when openings are relatively small****, chord forces do not increase significantly and it is usually sufficient simply to reinforce perimeter framing and assure that it is continuous. Continuous framing should extend from each corner of the opening both directions into the diaphragm,* ***a distance equal to the largest dimension of the opening****."*

As stated in the book, *Analysis of Irregular Shaped Structures, McGraw/Hill (Malone/Rice),* for small openings, *"ATC 7 and Diekmann recommend that at* ***small openings*** *minimal reinforcing at the corners of the opening should extend* ***a minimum distance equal to the depth or width of the opening*** *in the direction under consideration. In other words, the minimum distance left and right would be equal to the width "w" of the opening, and the minimum distance above and below the opening would be equal to the depth "d"."*

An example of what is meant, or not meant, in APA 138 by the term*, "…when openings are* ***relatively small****….",* is given further in the document when in the design considerations of Diaphragm No. 4 in APA report 138 it states, *"The 8-ft x 8-ft openings,* ***very large relative*** *to the size of the diaphragm, removed 50% of the plywood from the high shear areas near each reaction."*

While no definition of a "large opening" is given, it is clear that the force transfer around openings is a critical design feature to ensure a complete load path for wind and seismic loads. As with many requirements in the code, the true effect of inadequate load paths often are not realized until a significant load event occurs. The connection at openings in diaphragms is a critical element in the overall structural integrity of a building and the minimal cost and effort of installing the appropriate strapping and blocking required by this section is justified.

References:

APA The Engineered Wood Association. Rev. 2000 *Research Report 138 – Plywood Diaphragms,* APA, Tacoma, WA.

S292-12

Final Action: AS AM AMPC____ D

S298-12
2406.4.7

Proposed Change as Submitted

Proponent: Tim Pate, City & County of Broomfield Building Division, representing self

Revise as follows:

2406.4.7 Glazing adjacent to the bottom stair landing. Glazing adjacent to the landing at the bottom of a stairway where the glazing is less than 36 inches (914 mm) above the landing and within a 60 inch~~es~~ (1524 mm) horizontal~~ly of~~ arc less than 180 degrees from the bottom tread shall be considered a hazardous location.

> **Exception:** Glazing that is protected by a guard complying with Sections 1013 and 1607.8 where the plane of the glass is greater than 18 inches (457 mm) from the guard.

Reason: Previous editions of the IBC before the 2012 required glazing that is 60" horizontally in any direction to be approved safety glazing. It is not clear why this requirement was changed in the 2012. The previous editions had the additional wording "in any direction" when applying the 60" horizontal rule. This is due to the "splay" factor for when someone gets to the last tread and falls. The tendency is for someone to flail out in any direction. This added wording will make this section apply to any glazing that is in a wall that is less than 180 degrees from the bottom tread. This will make it very clear what the intent was and still is with this section.

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

2406.4.7#2-S-PATE.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: This code change does not clarify the requirements for glazing adjacent to the bottom stair landing. The term arc is not necessary and an illustration in the reason could help clarify the intent of this revision.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Tim Pate, City & County of Broomfield Building Department, representing Colorado Chapter Code Change Committee, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

2406.4.7 Glazing adjacent to the bottom stair landing. Glazing adjacent to the landing at the bottom of a stairway where the glazing is less than 36 inches (914 mm) above the landing and within a 60 inch (1524 mm) horizontal arc less than 180 degrees from the bottom tread nosing shall be considered a hazardous location.

> **Exception:** Glazing that is protected by a guard complying with Sections 1013 and 1607.8 where the plane of the glass is greater than 18 inches (457 mm) from the guard.

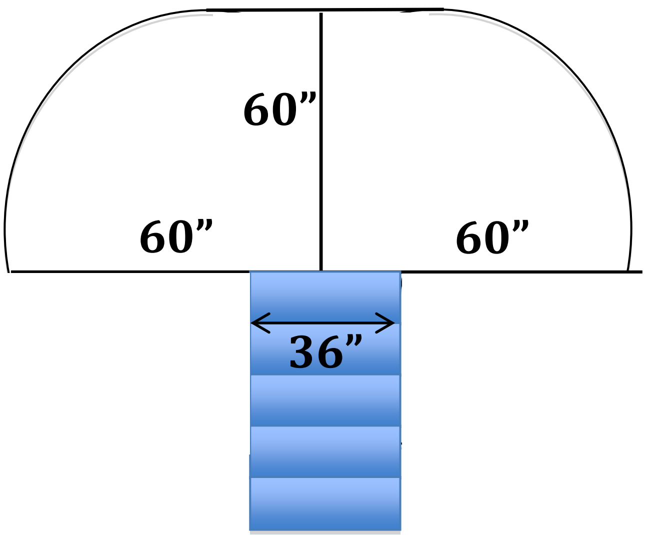
Commenter's Reason: This code change does not clarify the requirements for glazing adjacent to the bottom stair landing. The term arc is not necessary and an illustration in the reason could help clarify the intent of this revision.

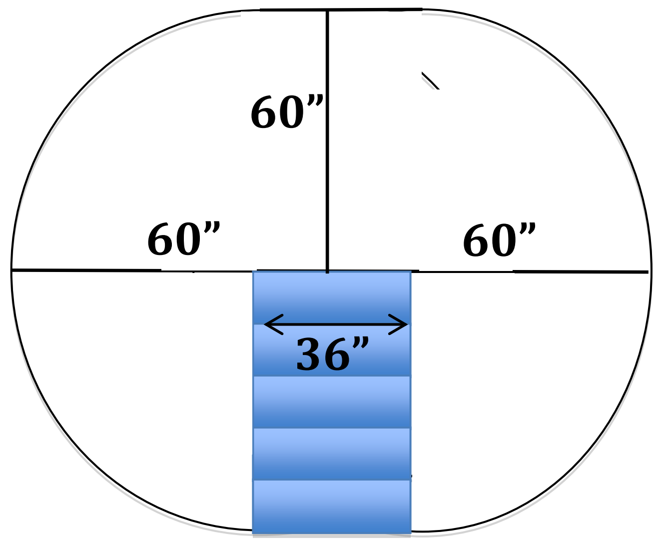
Previous editions of the IBC before the 2012 required glazing that is 60" horizontally in any direction to be approved safety glazing. It is not clear why this requirement was changed in the 2012. The previous editions had the additional wording "in any direction" when applying the 60" horizontal rule. This is due to the "splay" factor for when someone gets to the last tread and falls. The tendency is for someone to flail out in any direction.

This added wording will make this section only apply to any glazing that is in a wall that is less than 180 degrees from the bottom tread nosing. I believe that adding the wording which would limit the area needing safety glazing to any glazing that falls within a 180 degree arc from bottom tread nosing and extending out 60" makes more sense since it is extremely unlikely that someone will fall out and backwards. I have added an illustration which should help everyone see what this changed wording will do.

Please note that there is still a requirement to provide approved safety glazing when located within 36" horizontally of the sides of the stairs.

The new code language will incorporate the areas shown on the left diagram while the current code language covers the areas on the right diagram.





S298-12

Final Action: AS AM AMPC____ D

S299-12
2407.1.2

Proposed Change as Submitted

Proponent: Anthony Leto, The Wagner Companies

Revise as follows:

2407.1.2 Support. Each handrail or *guard* section shall be supported by a minimum of three glass balusters or shall be otherwise supported to remain in place should one baluster panel fail. Glass balusters shall not be installed without an attached ~~handrail or *guard*~~ top rail.

> **Exception:** A top rail shall not be required where the glass balusters are laminated glass with two or more glass plies of equal thickness and the same glass type when *approved* by the *building official*. The panels shall be designed to withstand the loads specified in Section 1607.8.

Reason: While the ICC opinion on top railing requirements for monolithic glass baluster guards has remained consistent, we continue to see installations without the required top rail. Where is the disconnect?

The confusion begins with *IBC Section 2407.1.1.2 Support*. There are three issues:

1. **The term *guard* is used improperly at the end of the second sentence.**
   The ICC defines *guard* as being in place to stop accidental falls and refers to the full assembly not the guard top. The word *guard* should be replaced with the words *top rail* as is noted in the *Exception*.
2. **In a glass baluster handrail, the handrail and top rail are the same component.**
   A glass baluster being used only as a handrail (i.e. a stair where there is less than a 30 inch drop from the top step) will require a handrail which must meet the dimensional and clearance requirements for handrail. It should be noted, that under the strict definition of handrail clearance, a handrail placed directly on top of a glass baluster does not meet code as the glass would be considered a 100% obstruction. The handrail would need to be attached to the glass baluster with brackets to provide code compliant clearance. The handrail would be the top most portion of the assembly, therefore the handrail would also serve as the top rail.
3. **Misinterpretation of the phrase, *Glass balusters shall not be installed without an attached handrail or guard.***
   Handrail is required on stairs and is located 34 to 38 inches above the stair nosing. A guard is required when there is a 30 inch drop. The IBC minimum for a guard is 42 inches above the walking surace. If a stair has a drop of greater than 30 inches, it would be required to have both a handrail and a guard. However, if the stair height does not exceed 30 inches, only a handrail is required.
   There are some who interpret that *Section 2407.1.1.2* allows a glass baluster guard to be installed with *either* a handrail or a guard (top rail).

   However, the section's intention is that a glass baluster *handrail* must have an attached *handrail* and that a glass baluster *guard* must have an attached *guard* (top rail). **The presence of a *handrail* on a *guard* does not eliminate the need for a top railing.** This interpretation is supported by:

   A. **The ICC**
   In 2008, Todd Daniel of the National Ornamental and Miscellaneous Metals Association (NOMMA) asked the following question of the International Code Council (ICC):

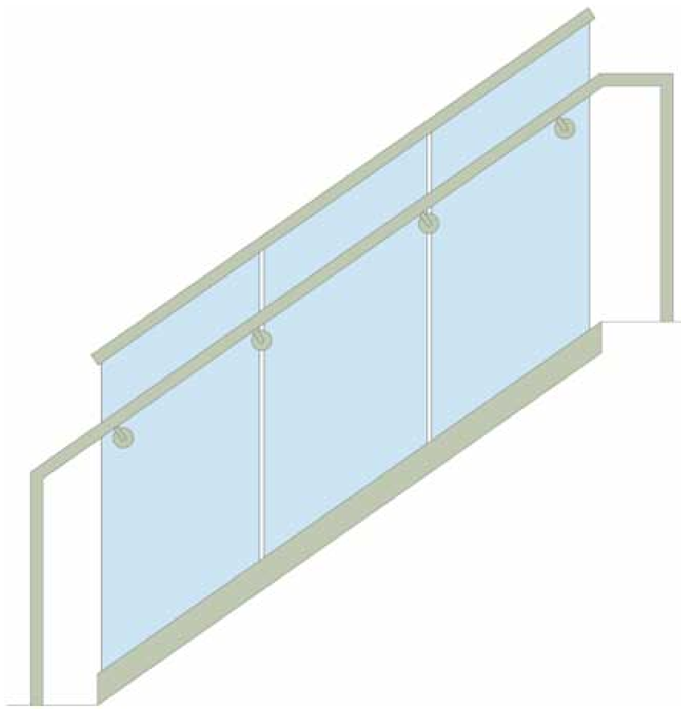
   Can a glass rail system be installed without a guard on top of the glass IF there is a handrail attached to the glass. In other words...no cap, exposed top edge of glass at 42 inch height with a handrail mounted on the side of the glass at handrail heights.
   ICC Staff Opinion: No
   Reason: The application you describe can only be allowed if the glass can withstand the loads for guards and handrails in Section 1607.7

   B. **The 2009 IBC Exception**
   The ICC approved an exception in 2009 that a top railing was not required if laminated glass is used that meets the load requirements and is approved by the building official. If this is the exception to the rule, then it should be understood that a top railing is required in all other situations.

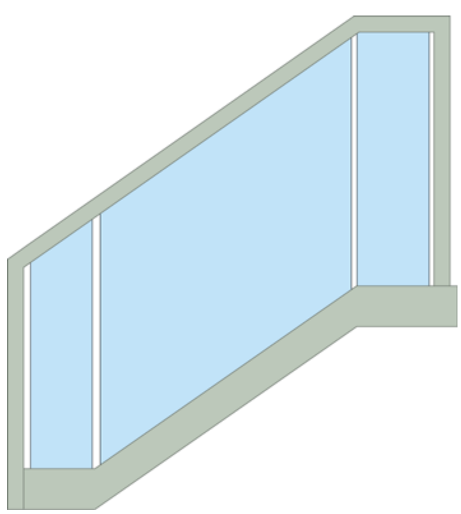
   C. **The Load Requirements**
   Section 2407.1.1 requires that glass baluster handrails and guards must meet the load requirements of 1607.7 with a safety factor of four.
   In a required guard, the loads must be applied to the *top* of the guard -- not the top of the handrail. Having a 42 inch guard with an attached handrail at between 34 and 38 inches will not meet the load requirement unless it is laminated tempered glass or the monolithic, tempered glass is of significant thickness.

Standard 1/2 inch monolithic, tempered glass edges are highly susceptable to rupture under load. Directing an 800 pound concentrated load (200 lbs multiplied by a safety factor of four) to that bare edge will most likely result in failure.

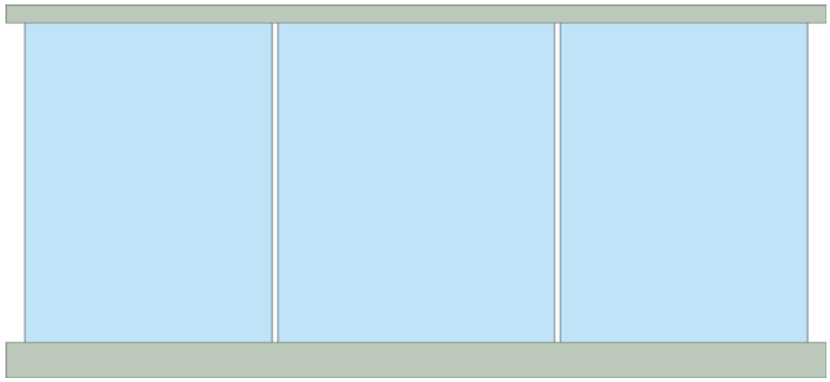
In 2011, there were numerous cases of glass railing failures across the US and Canada. An article relating these failuures was published this past October by US Glass Magazine (http://www.usglassmag.com/digital/2011/Oct2011.pdf). While most cases were likely the result of nickel sulfide inclusions in the glass, the consulting engineering firm brought in to determine the reasons for failure of glass railings at the W Hotel in Austin, TX noted that in one event, the failure was related to debris from above striking a bare edge of a glass panel.



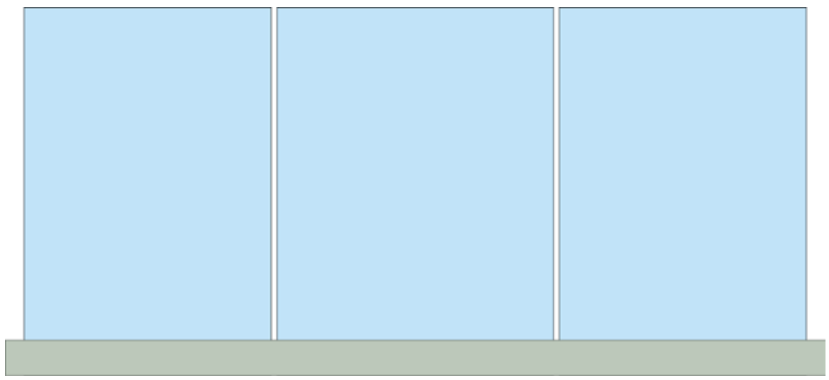
Stair with required guard and attached handrail.



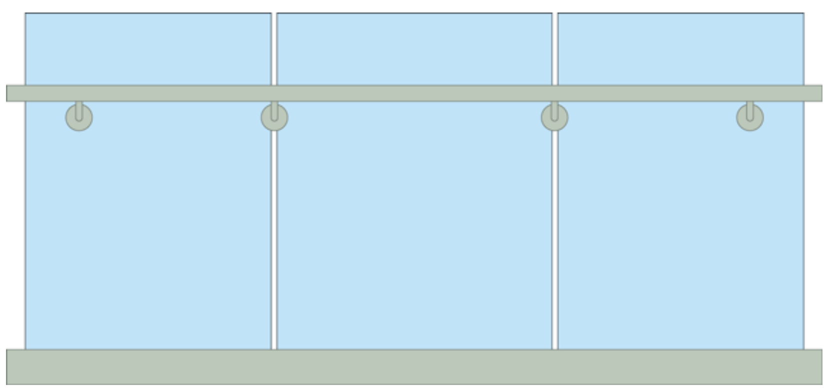
Required handrail for stair when a guard is not required.



Guard with top railing



Guard without top railing. Per ICC staff opinion, permitted only when used with laminated, tempered glass or if the glass meets the structural requirements of 1607.7



Guard with non-required handrail -- handrail is in place in an attempt to meet the requirements of an attached handrail or guard. However, the requirement is that the guard be able to withstand the load at the top of the guard. The handrail is not the top of the guard therefore the load must be met by the top edge of glass -- by a safety factor of four.

Cost Impact: There should be no cost impact since this change is to clarify and eliminate misinterpretation whereby glass railings are being installed without a top rail. In reality there will be long term savings as there are now situations where, as part of due diligence during a building purchase, consulting engineers are pointing out that glass rails without a top rail are not code compliant. Building owners in turn are requiring engineers/architects of record to have the railing redesigned to be code compliant.

2407.1.2-S-LETO.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The proponent requested disapproval, indicating the need to work on the wording and submit a public comment.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Todd Daniel, National Ornamental & Miscellaneous Metals Association, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

2407.1.2 Support. Each handrail or guard section shall be supported by a minimum of three glass balusters or shall be otherwise supported to remain in place should one baluster panel fail. Glass balusters shall not be installed without an attached handrail or top rail.

> **Exception:** A top rail shall not be required where the glass balusters are laminated glass with two or more glass plies of equal thickness and the same glass type when approved by the building official. The panels shall be designed to withstand the loads specified in Section 1607.8

Commenter's Reason: The term "guard" is incorrectly used in this passage. Substituting "top rail" provides greater clarity and accuracy.

The 2012 IBC defines a guard as "a building component or a system of building components located near the open sides of elevated walking surfaces that minimizes the possibility of a fall from the walking surface to the lower level." Following this definition, the word "guard" has been improperly used in the code.

S299-12

Final Action: AS AM AMPC____ D

S300-12
2407.1, 2407.1.1

Proposed Change as Submitted

Proponent: Thomas S. Zaremba, Roetzel & Andress, representing Glazing Industry Code Committee (tzaremba@ralaw.com)

Revise as follows:

2407.1 Materials. Glass used ~~as~~ <u>in</u> a handrail ~~assembly~~ <u>, guardrail</u> or a *guard* section shall be <u>laminated glass</u> constructed of ~~either single fully tempered glass, laminated~~ fully tempered glass or ~~laminated~~ heat-strengthened glass <u>and shall comply with Category II of CPSC 16 CFR Part 1201 or Class A of ANSI Z97.1</u>. Glazing in railing in-fill panels shall be of an *approved* safety glazing material that conforms to the provisions of Section 2406.1.1. For all glazing types, the minimum nominal thickness shall be $1/4$ inch (6.4 mm). ~~Fully tempered glass and laminated glass shall comply with Category II of CPSC 16 CFR Part 1201 or Class A of ANSI Z97.1.~~

> <u>**Exception:** Single fully tempered glass complying with Category II of CPSC 16 CFR Part 1201 or Class A of ANSI Z97.1 may be used in handrails and guardrails if there is no walking surface beneath them or the walking surface is permanently protected from the risk of falling glass.</u>

2407.1.1 Loads. The panels and their support system shall be designed to withstand the loads specified in Section 1607.8. A ~~safety~~ <u>design</u> factor of four shall be used <u>for safety</u>.

Reason: Several recent incidents involving spontaneous breakage of fully tempered glass in handrail or guardrail systems on high rise balconies has prompted the Glazing Industry Code Committee to seek this change which, if adopted, will make mandatory the use of the retentive characteristics of laminated glass in these applications unless there is no walking surface below or it is permanently protected from falling glass, in which case, fully tempered glass meeting the safety criteria of Cat. II of CPSC 16 CFR 1201 or Class A of ANSI Z97.1 would be permitted. Additionally, the proposal adds the term “guardrail” to section 2407.1 since that term is also used in various locations throughout the I-codes in connection with these types of systems.

Finally, proposal changes Section 2407.1.1 are intended to make it clear that a “design” factor of four is required “for safety.” The intent of this section is to use a “design” factor of four when determining the loads of these panels and their support systems. Using the word “safety” in the way it is currently found in this section is ambiguous and may or may not achieve the section’s intended purpose.

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

2407.1-S-ZAREMBA.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: Disapproval is due to confusing testimony on this code change. A cited incident was in an exterior guard yet the proposal would also affect interior installations. No documentation of failures was provided for committee review.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Thomas S. Zaremba, Roetzel & Andress, representing Glazing Industry Code Committee, requests Approval as Submitted.

Commenter's Reason: Starting last summer, panes of tempered glass in guard and handrail assemblies on balconies in 13 different buildings in Toronto and Montreal, Canada spontaneously broke dropping broken tempered glass onto the ground below. See photo below showing a broken tempered pane in a guardrail.

In June of this year, several tempered glass guard and handrail assemblies on balconies on a high rise hotel and residence in Austin, Texas spontaneously broke and fell to the ground.

In September of last year, several tempered glass guard and handrail assemblies on balconies in a Four Season's Hotel in Seattle, Washington spontaneously failed.

Fortunately, no one was injured in any of these events. However, they have resulted in the replacement of thousands of glass guard and handrail assemblies. The fully tempered glass assemblies that were originally installed are being replaced either with fully tempered laminated glass assemblies or heat strengthened laminated glass assemblies.

July 2012 - Broken tempered glass assembly 15th floor condo at 33 Mill Street, in Toronto.

S300-12 was submitted by the Glazing Industry Code Committee in direct response to these glass guard and handrail failures. While fully tempered glass is a safety glazing appropriate for use in many hazardous locations, it is not appropriate for use in guard and handrail assemblies unless it is laminated or walking surfaces below them are permanently protected from the risk of falling glass. If adopted, that is exactly what S300-12 would require.

At the hearings in Dallas, opponents testified that the cost of laminating glass guard and handrail assemblies would increase their cost and that, while the proposed change would apply to glass guard and handrail assemblies in both exterior and interior locations, Proponent failed to come forward with any specific evidence of an interior glass guard or handrail failure.

It is true that laminating glass used in guards and handrails will increase their cost. However, the increased cost is justified by the life safety issue addressed by S300-12, namely, the risk of unexpectedly being struck by falling glass from a fully tempered glass guard or handrail assembly located above a walking surface.

It is also true that Proponent did not come forward with any specific instances of interior failures. However, the cause of the failures at issue has nothing to do with whether the glass guards or handrails are located on the inside or on the outside of buildings.

Fully tempered glass on rare occasion may suffer spontaneous breakage due to nickel sulfide inclusions or other impurities in the glass which are undetectable when they are present. However, if they exist, they may cause fully tempered glass to spontaneously break at one point during its useful life. That breakage may occur sooner, later or not at all. In short, it has nothing to do with whether the glass is used inside or outside of the building.

If a pane of tempered glass in a guard or handrail does break spontaneously, the entire pane of tempered glass will fracture into many, small particles (typically less than the size of a dime), many of which may fall from the frame. The photo above shows a fractured pane of tempered glass where some of the particles had fallen from the opening. These particles can fall as individual particles or as clusters of loosely joined particles. If people happen to be standing or walking below a tempered glass guard or handrail when it spontaneously breaks, it is possible they may be hit by individual particles or clusters of broken glass.

If, on the other hand, a laminated pane of tempered or heat strengthened glass breaks, the plastic used to laminate the glass will tend to hold the broken pieces together so that they won't fall out of the frame. Other than lamination, the only other way to protect against the risk of falling glass from such installations is to require walking surfaces below the glass guards or handrails to be permanently protected from falling glass.

S300-12 presents a life safety issue. The glass industry constantly strives through the ICC code development process and otherwise to ensure that the "right glass is used in the right application." Fully tempered glass that is not laminated is not the right glass for use in guard and handrail locations unless any walking surface below it is permanently protected from the risk of falling glass.

The Glazing Industry Code Committee urges you to support the adoption of S300-12 as submitted. This will require you to vote against the standing motion to disapprove S300-12 and to vote in favor of a subsequent motion to approve S300-12 as submitted.

S300-12

Final Action: AS AM AMPC____ D

S302-12
202 (New), 1710.6, 2410 (New), 2410.1 (New), 2410.2 (New)

Proposed Change as Submitted

Proponent: Timothy Burgos, InterCode Incorporated, representing 3M Company

Add new text as follows:

SECTION 202
DEFINITIONS

SUNLIGHT DELIVERY SYSTEM (SDS). A unit primarily designed to transmit daylight from an exterior surface to an interior space via a reflective duct or conduit. The basic unit consists of an exterior solar collecting device, a daylight-transmitting duct or conduit with a reflective interior surface, and an interior-ceiling device such as a translucent ceiling panel. The unit can be factory assembled, or field-assembled from a manufactured kit.

Revise as follows:

1710.6 Skylights ~~and~~ sloped glazing, and sunlight delivery systems. Unit skylights ~~and tubular daylighting devices (TDDs)~~ shall comply with the requirements of Section 2405. Sunlight delivery systems (SDS's) and tubular daylighting devices (TDDs) shall comply with the requirements of Section 2410. All other skylights and sloped glazing shall comply with the requirements of Chapter 24.

Add new text as follows:

SECTION 2410
SUNLIGHT DELIVERY SYSTEMS AND TUBULAR DAYLIGHTING DEVICES

2410.1 General. Sunlight delivery systems and tubular daylighting devices shall comply with the requirements of this code and be installed per the manufacturer's specifications.

2410.2 Definition. The following terms are defined in Chapter 2:

SUNLIGHT DELIVERY SYSTEM.

TUBULAR DAYLIGHTING DEVICE.

Reason: The purpose of this proposed edit is to create a more expansive definition of the tubular daylighting device. While tubular daylighting devices are a common implementation of the principles of reflective daylighting, new advancements in the field are available worldwide and should be included in the next edition of the International Building Code. Having a more expansive definition in the International Building Code for sunlight delivery systems will open up new technologies that can introduce natural sunlight into the interior areas that do not have windows or natural light entering that room. A sunlight delivery system provides designers with a new method of daylighting that offers significantly greater capabilities than existing alternatives. Traditional daylighting methods, such as skylights or tubular daylighting devices, are limited. These systems can require multiple entry points and are often limited to top floor applications. An example of a sunlight delivery system can be found in the pictures at the end of this reason statement.

The widespread use of electrical lighting in the 20th century changed the design of buildings but often made it impossible to illuminate internal rooms with daylight, thus requiring the use of artificial light in internal spaces. The use of artificial light currently makes up as much as 45% of the energy use in commercial and industrial buildings and up to 35% in residential buildings.

Sunlight delivery systems can significantly reduce energy costs for illumination. In a paper presented to LuxEuropa in 2009 entitled *Hybrid Lighting systems: a feasibility study for Europe* by Mohammed S. Mayhoub and David Carter, energy savings ranging from 28% to 85% (in latitudes ranging from 60^o North (Oslo, Norway) to 36^o North (Khania, Greece) were reported when a variety of sunlight delivery systems were tested. These locations correlate to locations in the United States as follows: Oslo, Norway is similar in latitude to Juneau and Anchorage, AK and Khania, Greece is similar to Virginia Beach, VA; Las Vegas, NV; and Nashville, TN.

The study showed that the greatest savings were realized in the Southern most latitudes (in the Northern Hemisphere) but still showed the possibility that 50% savings could be realized at 60^0 North with the most advanced systems. Because the study was limited to European Union countries, no analysis was conducted in more southern latitudes similar to the southernmost portion of the United States where cities such as Tampa, San Antonio, and New Orleans are located. In fact, most of the land mass of the contiguous 48 United States lies well below 50^0 North indicating that greater savings could be realized in the United States than those projected in Europe.

An abundance of research and knowledge shows not only that the preferred light source in buildings is natural daylight but also that lack of exposure to daylight can lead to biological issues, lack of productivity, higher levels of stress, sleep difficulties and a variety of other human response issues. Studies suggest that creating healthy indoor lighting by providing day-lighting and natural lighting cycles can be a simple form of preventative medicine and can lead to higher production and overall better mental and physical health for the inhabitants. The health benefits that a sunlight delivery device provides is one of the reasons for this code change to be approved.

Roof top solar collecting devices used in a sunlight delivery system.

Sunlight being delivered to the interior space in an open ceiling (on the left) and in a dropped ceiling (on the right) by way of a sunlight duct system.

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

202-SDS-S-BURGOS

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: This code change proposes a definition of sunlight delivery systems and requires them to installed per the manufacturers specifications which does not truly add anything to the code. They still have to be treated as alternative methods and this is available currently without being added to the code.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Vickie Lovell, InterCode Incorporated, representing 3M Company, requests Approval as Modified by this Public Comment.

Replace the proposal as follows:

Sunlight Delivery System. A unit designed to transmit daylight from an exterior surface to an interior space by means of a reflective duct or conduit. The unit consists of an exterior solar collecting device, a reflective duct or conduit, and an interior ceiling device such as a translucent ceiling panel.

SECTION 1210
SUNLIGHT DELIVERY SYSTEMS

1210.1 General. Sunlight delivery systems shall comply with the requirements of this code for duct construction and shall be installed per the manufacturer's specifications.

1210.2 Definition. The following term~~s~~ are defined in Chapter 2:

SUNLIGHT DELIVERY SYSTEM.

Commenter's Reason: Sunlight delivery systems are an existing worldwide technology, but are new to this code. The committee disapproved the original code change proposal stating that SDSs do not belong in Chapter 24. We agree, however, there is no section or chapter that addresses this technology. It was suggested to us by a committee member that language for SDSs should be included in Chapter 12 of the International Building Code and that is what this public comment does.

S302-12
Final Action: AS AM AMPC____ D

S305-12
202, 2102.1 (New), 2502.1 (New)

Proposed Change as Submitted

Proponent: John Mulder, Intertek Testing Services NA, Inc., representing International Standards Organization Technical Committee 77, *Products in Fibre-reinforced Cement* and self

Revise as follows:

SECTION 202
DEFINITIONS

FIBER-CEMENT ~~SIDING~~ PRODUCTS. ~~A~~ Manufactured, ~~fiber-reinforcing product made with an inorganic hydraulic or calcium silicate binder formed by chemical reaction and reinforced with discrete organic or inorganic nonasbestos fibers, or both. Additives that enhance manufacturing or product performance are permitted.~~ thin section composites of hydraulic cementitious matrices and discrete non-asbestos fibers. Fiber-cement backer board products have either a smooth or textured face and are normally installed to wall or ceiling framing over which paint, wallpaper, resilient flooring, tile, natural stone or dimensioned stone *veneer* are applied. Fiber-cement underlayment products have either a smooth or textured face and are installed on a wood subfloor over which resilient flooring, tile, natural stone or dimensioned stone *veneer* are applied. Fiber-cement lap or panel siding, soffit, and trim products have either smooth or textured faces and are intended for *exterior wall* and related applications.

Add new text as follows:

2102.1 General. For the purposes of this chapter and as used elsewhere in this code, the following terms are defined in Chapter 2:

FIBER-CEMENT PRODUCTS

Add new text as follows:

2502.1 Definitions. The following terms are defined in Chapter 2:

FIBER-CEMENT PRODUCTS

Reason: The current definition is limited to fiber-cement siding products. The proposal corrects the definition to that published in ASTM C1154-06, *Standard Terminology for Non-Asbestos Fiber-reinforced Cement Products* (see attached copy of ASTM C1154-06), for "fiber-cement products". Additional text describes types of fiber-cement products to include also fiber-cement backer board, underlayment, soffit and trim products currently recognized in the Code (IBC Sections 1404.10, 1405.16, and 2509.2). The proposed code change eliminates a barrier to trade by including other fiber-cement products currently permitted by the Code.

A revision to Section 2103 (new Section 2103.15) is proposed to include "fiber-cement backer board and underlayment". The term "fiber-cement products" is proposed to be included in the definitions here consistent with the definition published in the Terminology Standard ASTM C1154-06, *Standard Terminology for Non-Asbestos Fiber-Reinforced Cement Products* (see attached Standard).

"Fiber-cement backer board is currently permitted for use in Section 2509.2. A new term is added to reference the permitted backer board material now defined in proposed new TABLE 2509.2, where all 3 permitted products are now listed and the proposed revision to Section 202 to include "fiber-cement products".

Cost Impact: The code change proposal will not increase the cost of construction because the change simply corrects the current definition to be consistent with the National Standard and provides examples of the types of products covered by the definition.

202-FIBER-CEMENT SIDING-S-MULDER.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee felt the language proposed for the definition was confusing and the committee was not convinced on the need to change "fiber-cement siding" to "fiber-cement products".

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

John Mulder representing Intertek Testing Services NA, Inc. and self, requests Approval as Modified by this Public Comment.

Replace the proposal as follows:

FIBER-CEMENT (BACKER BOARD, SIDING, SOFFIT, TRIM, AND UNDERLAYMENT) PRODUCTS. ~~A~~ M~~m~~anufactured thin section composites of hydraulic cementitious matrices and discrete non-asbestos fibers.~~, fiber-reinforced products made with an inorganic hydraulic or calcium silicate binder formed by chemical reaction and reinforced with discrete organic or inorganic nonasbestos fibers, or both. Additives that enhance manufacturing or product performance are permitted.~~

Commenter's Reason: The current definition is limited to fiber-cement siding products. The proposal corrects the definition to that published in ASTM C1154-06, *Standard Terminology for Non-Asbestos Fiber-reinforced Cement Products* (see attached copy of ASTM C1154-06), for "fiber-cement products". Additional text describes types of fiber-cement products to include also fiber-cement backer board, soffit, trim and underlayment products currently recognized in the Code (IBC Sections 1404.10, 1405.16, Table 2304.7(4) a., and 2509.2). The proposed code change eliminates a barrier to trade by including other fiber-cement products currently permitted by the Code.

Cost Impact: The code change proposal will not increase the cost of construction because the change simply corrects the current definition to be consistent with the National Standard and provides examples of the types of products covered by the definition.

S305-12
Final Action: AS AM AMPC____ D

S308-12
2509.3

Proposed Change as Submitted

Proponent: Michael Gardner, Gypsum Association (mgardner@gypsum.org)

Revise as follows:

2509.3 Limitations. Water-resistant gypsum backing board shall not be used in the following locations:

1. Over a vapor retarder in shower or bathtub compartments.
2. Where there will be direct exposure to water or in areas subject to continuous high humidity.
3. ~~On ceilings where frame spacing exceeds 12 inches (305 mm) o.c. for 1/2-inch thick (12.7 mm) water-resistant gypsum backing board and more than 16 inches (406 mm) o.c. for 5/8-inch thick (15.9 mm) water-resistant gypsum backing board.~~

Reason: Concurrent language necessitating the addition of supplemental framing members when water-resistant ceiling board is installed on a ceiling has been or is being removed from the code-referenced gypsum board and panel application standards, GA-216 and ASTM C 840.

Testing has shown that water-resistant gypsum board, as presently manufactured, has better sag resistance than regular core board of the same thickness. As a consequence, the supplemental framing limitation is no longer necessary.

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

2509.3-S-GARDNER

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee believes that there was no justification given for removing this provision for supplemental framing when installing water-resistant ceiling board.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Michael Gardner, representing Gypsum Association, requests Approval as Submitted

Commenter's Reason: The requirement to install supplemental framing when water-resistant gypsum board is applied to a ceiling was introduced into the Uniform Building Code many decades ago when the emulsions added during manufacturing to waterproof the core of the board were heavier in weight than those used today. The added weight of the emulsions led to concerns about board sag in an installed environment.

The water-resistive additives now used to manufacture water-resistant gypsum board are significantly lighter in weight. They also produce board with a stiffer core. As a consequence, contemporary water-resistant gypsum board is less susceptible to sag than its predecessor.

Both of the gypsum board application standards, ASTM C840 and GA-216, referenced by the IBC have been modified to eliminate any prescriptive requirements mandating the installation of supplemental framing support members when water-resistant gypsum board is applied to a ceiling. The ASTM C 840 standard is a consensus standard and reflects the input of manufacturers, contractors, and other interested parties. The intent of the original proposal is to make the IBC consistent with the referenced standards.

Both standard wallboard and water-resistant gypsum board are manufactured to the same standard, ASTM C1396. The humidified deflection and flexural strength tolerances for both products are identical. On the basis of the manufacturing standard, water-resistant gypsum board is no more susceptible to sag than is standard wallboard.

The supplemental framing requirement has historically been an often-overlooked catch-point for contractors and inspectors. It has become irrelevant and should be deleted from the code.

S308-12

Final Action: AS AM AMPC____ D

S310-12
2510.6, Chapter 35 (New)

Proposed Change as Submitted

Proponent: Theresa Weston, DuPont Building Innovation (theresa.a.weston@usa.dupont.com)

Revise as follows:

2510.6 Water-resistive barriers. *Water-resistive barrier*s shall be installed as required in Section 1404.2 and, where applied over wood-based sheathing, shall include a water-resistive vapor-permeable barrier with a performance at least equivalent to two layers of ~~Grade D paper~~ water-resistive barrier complying with ASTM E 2556 Type 1. The individual layers shall be installed independently such that each layer provides a separate continuous plane and any flashing (installed in accordance with Section 1405.4) intended to drain to the water-resistive barrier is directed between the layers.

> **Exception:** Where the *water-resistive barrier* that is applied over wood-based sheathing has a water resistance equal to or greater than that of ~~60-minute Grade D paper~~ a water-resistive barrier complying with ASTM E 2556 Type II and is separated from the stucco by an intervening, substantially nonwater-absorbing layer or drainage space.

Add new standard to Chapter 35 as follows:

ASTM

E 2556 - Standard Specification for Vapor Permeable Flexible Sheet Water-Resistive Barriers Intended for Mechanical Attachment

Reason: The proposal updates the water-resistive barrier reference to the most recent consensus standard. ASTM E2556 includes house wrap materials, building papers and felt, instead of just building paper and therefore is more representative of the state of the industry. Within ASTM E2556 Grade D paper is a Type I WRB and 60 minute Grade D paper is a Type II WRB. ASTM E2556 is consistent with the current ICC-ES acceptance criteria for water-resistive barriers (AC-38) and therefore should not limit the use of current WRBs.

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

Analysis: A review of the standard proposed for inclusion in the code, [IBC] 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, 2012.

2510.6-S-WESTON.doc

Public Hearing Results

Note: For staff analysis of the content of ASTM E 2556 relative to CP#28, Section 3.6, please visit:
http://www.iccsafe.org:8888/cs/codes/Documents/2012-13cycle/Proposed-A/00a_updates.pdf

Committee Action: **Approved as Submitted**

Committee Reason: This proposal replaces Grade D paper which is not defined with a new material referenced standard which will clarify Section 2510.6. The committee concluded that this is strictly a material issue and that the reference to chapter 14 takes care of installation and performance required for a weather-resistive barrier.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Jay H. Crandell, ARES Consulting, representing American Chemistry Council – Foam Sheathing Committee, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

2510.6 Water-resistive barriers. A w~~W~~*ater-resistive barrier*~~s~~ complying with ~~shall be installed as required in~~ Section 1404.2 shall be provided and, where applied over wood-based sheathing, shall ~~include a water-resistive vapor-permeable barrier with a performance at least equivalent to~~ comply with one of the following methods or an approved design:

1. T~~t~~wo layers of paper-based ~~water-resistive~~ barrier (Grade D paper) complying with ASTM E 2556 Type I~~. The individual layers~~ shall be installed independently such that each layer provides a separate continuous plane and any flashing (installed in accordance with Section 1405.4) intended to drain to the water-resistive barrier is directed between the layers.
2. Two layers of felt-based barrier or polymer-based barrier material complying with ASTM E 2556 Type I installed as required in method #1.
3. ~~**Exception:**~~ A single layer ~~Where the~~ *water-resistive barrier* that is vapor permeable or not vapor permeable and that ~~applied over wood-based sheathing~~ has a water resistance equal to or greater than that of 60 minute Grade D paper ~~a water-resistive barrier~~ complying with ASTM E 2556 Type II. The single layer water-resistive barrier shall be ~~and is~~ separated from the stucco by an intervening, substantially nonwater-absorbing layer or drainage space.

2510.6.1 Water vapor control. Where the vapor permeability of a vapor-permeable water-resistive barrier material is greater than that of 60 minute Grade D paper complying with ASTM E2556 Type II, the water-resistive barrier manufacturer's installation instruction or an approved design shall identify appropriate use limitations, if any, with respect to climate conditions that cause inward vapor transmission through the vapor-permeable water-resistive barrier material. Where the water-resistive barrier material is not vapor-permeable, the water-resistive barrier manufacturer's installation instruction or an approved design shall identify appropriate use limitations, if any, with respect to climate conditions and vapor retarder applications in accordance with Section 1405.3.

(Portions of proposal not shown remain unchanged)

Commenter's Reason: This public comment addresses the code development committee's desire to reference material performance requirements included in ASTM E 2556, but makes some necessary improvements and clarifications to the original S310 proposal as approved at the first hearing. This PC makes the requirements more transparent with respect to the vapor-permeable material scope of ASTM E2556 and more clearly distinguishes conditions applicable to water-resistive barriers that are not considered to be vapor permeable. The public comment also explicitly retains the familiar Grade D paper practice (Method 1), transparently describes other materials included in ASTM E2556 that can be used (Method 2), and clarifies that the exception (now Method 3) applies to water-resistive barrier materials that are vapor permeable and not vapor permeable (i.e., are not within the scope of ASTM E2556). The limited scope of ASTM 2556 has necessitated these changes to avoid confusion and misinterpretation of acceptable water-resistive barrier properties, including vapor-permable and non-vapor-permeable types.

The second part of this proposal, proposed new Section 2510.6, addresses a problem inherent to the ASTM E2556 standard in that it applies only to vapor-permeable water-resistive barrier materials and establishes vapor permeability as a minimum requirement with no maximum value. However, products which have a significantly greater permeability than the minimum requirement (or Grade D paper) can and do allow excessive transmission of water vapor from stucco finishes into wall cavities and wood-based sheathing under certain climate conditions. This concern is addressed by an added requirement for an approved design or manufacturer's installation instructions to identify appropriate limitations on use depending on climate. Similarly, when water-resistive barrier is not vapor permeable, it may work very well in warm humid climates and in cold climates when coupled with exterior insulation, but this must be coordinated with vapor retarder requirements in Section 1405.3. This too is addressed by way of adding a requirement for an approved design or manufacturer's installation instructions which identify appropriate limitations or use requirements for a given application.

S310-12

Final Action: AS AM AMPC____ D

S311-12
2510.6

Proposed Change as Submitted

Proponent: John Woestman, Kellen Company, representing Builders Masonry Veneer Manufacturers Association (MVMA) (jwoestman@kellencompany.com)

Revise as follows:

2510.6 Water-resistive barriers. *Water-resistive barrier*s shall be installed as required in Section 1404.2 and, where applied over wood-based sheathing, shall include a ~~water-resistive vapor-permeable barrier with a performance at least equivalent to~~ water-resistive barrier with a moisture vapor permeance equal to or greater than that of two layers of Grade D paper. The individual layers shall be installed independently such that each layer provides a separate continuous plane and any flashing (installed in accordance with Section 1405.4) intended to drain to the water-resistive barrier is directed between the layers.

> **Exception:** Where the *water-resistive barrier* that is applied over wood-based sheathing has a water resistance and a moisture vapor permeance equal to or greater than that of 60-minute Grade D paper and is separated from the stucco by an intervening, substantially nonwater-absorbing layer or drainage space.

Reason: Existing language may be considered ambiguous as to what performance attribute is desired to be at least equivalent to two layers of Grade D paper. Water resistance? Moisture vapor permeance? This proposal clarifies moisture vapor permeability is the performance attribute desired to be at least equivalent to Grade D paper. And in the Exception, states moisture vapor permeance equal to or greater than that of 60-minute Grade D paper.

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

2510.6-S-WOESTMAN.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee's action on S310-12 is preferred. The proponent is encouraged to work on a public comment to iron out differences based on changes approved in S310-12.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment 1:

Jay H. Crandell, ARES Consulting, representing American Chemistry Council – Foam Sheathing Committee, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

2510.6 Water-resistive barriers. A w~~W~~*ater-resistive barrier*~~s~~ complying with ~~shall be installed as required in~~ Section 1404.2 shall be provided and, where applied over wood-based sheathing, shall ~~include a water-resistive barrier with a moisture vapor permeance equal to or greater than that of~~ comply with one of the following methods or an approved design:

1. T~~t~~wo layers of Grade D paper~~. The individual layers~~ shall be installed independently such that each layer provides a separate continuous plane and any flashing (installed in accordance with Section 1405.4) intended to drain to the water-resistive barrier is directed between the layers.

2. Two layers of a vapor-permeable water-resistive barrier shall be installed as required in method #1 with each layer having a water resistance at least equivalent to that of Grade D paper.
3. ~~**Exception:**~~ A single layer ~~Where the~~ *water-resistive barrier* that is vapor permeable or not vapor permeable and that ~~applied over wood-based sheathing~~ has a water resistance ~~and a moisture vapor permeance~~ equal to or greater than that of 60 minute Grade D paper shall be ~~and is~~ separated from the stucco by an intervening, substantially nonwater-absorbing layer or drainage space.

2510.6.1 Water vapor control. Where the vapor permeability of a vapor-permeable water-resistive barrier material is greater than that of 60 minute Grade D paper, the water-resistive barrier manufacturer's installation instruction or an approved design shall identify appropriate use limitations, if any, with respect to climate conditions that cause inward vapor transmission through the vapor-permeable water-resistive barrier material. Where the water-resistive barrier material is not vapor-permeable, the water-resistive barrier manufacturer's installation instruction or an approved design shall identify appropriate use limitations, if any, with respect to climate conditions and vapor retarder applications in accordance with Section 1405.3.

Commenter's Reason: The proponent of the S311 proposal was seeking to bring clarity to the performance requirements needed for WRBs when used together with PC stucco and for the special case where it is used over wood sheathing with PC stucco. The CDC preferred S310, but S310 did not address the proponent's concern and the CDC recommended that the proponent work out differences based on changes approved in S310-12. A separate PC on S310-12 also has been provided to address the concern. But, a PC is also provided on this proposal in the event that S310-12 may be disapproved at the final action hearing.

This PC clarifies the performance requirements for and unique applications of water-resistive barriers applied behind stucco and particularly over wood-based sheathing. The requirements related to different performance attributes of vapor permeable and non-vapor permeable water-resistive barriers are clarified for three distinct methods of application, the first method retaining the traditional and familiar application of Grade D paper. Section 2510.6.1 is added to address consideration of moisture vapor control in the wall assembly when water-resistant barriers of a vapor-permeable type and non-vapor permeable type are used in accordance with the provisions of Section 2510.6.

This PC addresses the answer to the concern expressed in the reason statement for the original S311 proposal: "Existing language may be considered ambiguous as to what performance attribute is desired to be at least equivalent to two layers of Grade D paper." The answer is: IT DEPENDS. If a vapor-permeable water resistive barrier is used, it is vapor permeability and water resistance that are to be equivalent. If a non-vapor-permeable water resistive barrier is used, it is only water resistance. In both cases, there are considerations related to moisture vapor control that must be considered and each approach can have pros and cons depending on the climate conditions and the overall wall assembly design for moisture vapor control.

Public Comment 2:

Theresa Weston, PhD. DuPont Building Innovations, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

2510.6 Water-resistive barriers. *Water-resistive barrier*s shall be installed as required in Section 1404.2 and, where applied over wood-based sheathing, shall include a vapor-permeable water-resistive barrier consisting of ~~with a moisture vapor permeance equal to or greater than that~~ two layers of Grade D paper or other approved material. The individual layers shall be installed independently such that each layer provides a separate continuous plane and any flashing (installed in accordance with Section 1405.4) intended to drain to the water-resistive barrier is directed between the layers.

> **Exception:** Where the *water-resistive barrier* that is applied over wood-based sheathing has a water resistance ~~and a moisture vapor permeance~~ equal to or greater than that of 60 minute Grade D paper and is separated from the stucco by an intervening, substantially nonwater-absorbing layer or drainage space.

Commenter's Reason: The modification makes changes to the original proposal without changing the intent of the original proposal to clarify the section of the code. The modification is provided in response to the committees request to coordinate the proposed changes with S310 which was approved as submitted in preference to this proposal.

S311-12
Final Action: AS AM AMPC____ D

S315-12, Part I
Appendix N (New)

Proposed Change as Submitted

Proponent: Martin Hammer, Architect, representing California Straw Building Association, Colorado Straw Bale Association, Straw Bale Construction Association – New Mexico, Ontario Bale Building Coalition, Development Center for Appropriate Technology, Environmental Building Network (mfhammer@pacbell.net)

THIS IS A TWO PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IBC FIRE SAFETY COMMITTEE. SEE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES

PART I – IBC STRUCTURAL

Add new text as follows:

APPENDIX N
STRAWBALE CONSTRUCTION

The provisions contained in this appendix are not mandatory unless specifically referenced in the adopting ordinance.

SECTION N101
GENERAL

N101.1 Scope. This appendix shall govern the use of baled straw as a building material.

SECTION N102
DEFINITIONS

N102.1 Definitions. The following words and terms shall, for the purposes of this appendix, have the meanings shown herein. Refer to Chapter 2 of the *International Building Code* for general definitions.

BALE. Equivalent to straw bale.

CLAY. Inorganic soil with particle sizes less than 0.00008 in. (0.002 mm) having the characteristics of high to very high dry strength and medium to high plasticity.

CLAY SLIP. A suspension of clay particles in water.

FLAKE. An intact section of compressed straw removed from an untied bale.

LAID FLAT. The orientation of a bale with its largest faces horizontal, its longest dimension parallel with the wall plane, its ties concealed in the unfinished wall and its straw lengths oriented across the thickness of the wall.

LOAD-BEARING WALL. For the purposes of this appendix, any strawbale wall that supports more than 100 lb/linear ft (1,459 N/m) of vertical load in addition to its own weight.

MESH. An openwork fabric of linked strands of metal, plastic, or natural or synthetic fiber, embedded in plaster to provide tensile reinforcement or bonding.

NONLOAD-BEARING WALL. For the purpose of this appendix, any wall that is not a load-bearing wall.

NONSTRUCTURAL WALL. All walls other than load-bearing walls or shear walls.
ON-EDGE. The orientation of a bale with its largest faces vertical, its longest dimension parallel with the wall plane, its ties on the face of the wall, and its straw lengths oriented vertically.

PIN. Metal rod, wood dowel, or bamboo, driven into, or through-tied on the surface of stacked bales for the purpose of connection or stability.

PLASTER. Gypsum, lime, cement-lime, or cement plasters, as defined in Chapter 25 and in Section N106, or clay plaster as defined in Section N106.9, or soil-cement plaster as defined in Section N106.10.

PRE-COMPRESSION. Vertical compression of stacked bales before the application of finish.

REINFORCED PLASTER. A plaster containing mesh reinforcement.

RUNNING BOND. For the purposes of this appendix, the placement of straw bales such that the head joints in successive courses are offset at least one quarter the bale length.

SHEAR WALL. A strawbale wall designed to resist lateral forces parallel to the plane of the wall in accordance with Section N105.15.

SKIN. The compilation of plaster and reinforcing, if any, applied to the surface of stacked bales.

STRUCTURAL WALL. A wall that meets the definition for a load-bearing wall or shear wall.

STACK BOND. For the purposes of this appendix, the placement of straw bales such that head joints in successive courses are vertically aligned.

STRAW. The dry stems of cereal grains after the seed heads have been removed.

STRAW BALE. A rectangular compressed block of straw, bound by ties.

STRAWBALE. The adjective form of straw bale.

STRAW-CLAY. Loose straw mixed and coated with clay slip.

TIE. A synthetic fiber, natural fiber, or metal wire used to confine a straw bale.

TRUTH WINDOW. An area of a strawbale wall left without its finish, to allow view of the straw otherwise concealed by its finish.

SECTION N103
BALES

N103.1 Types of straw. Bales shall be composed of straw from wheat, rice, rye, barley, or oat.

N103.2 Shape. Bales shall be rectangular in shape.

N103.3 Size. Bales shall have a minimum height and thickness of 12 inches (305 mm), except as otherwise permitted or required in this appendix. Bales used within a continuous wall shall be of consistent height and thickness to ensure even distribution of loads within the wall system.

N103.4 Ties. Bales shall be confined with synthetic fiber, natural fiber, or metal ties sufficient to maintain required bale density. Ties shall be at least 3 inches (76 mm) and not more than 6 inches (152 mm) from bale faces and shall be spaced not more than 12 (305 mm) inches apart. Bales with broken ties shall be retied with sufficient tension to maintain required bale density.

N103.5 Moisture content. The moisture content of bales at the time of application of the first coat of plaster or the installation of another finish shall not exceed 20 percent of the weight of the bale. The moisture content of bales shall be determined by use of a moisture meter designed for use with baled straw or hay, equipped with a probe of sufficient length to reach the center of the bale. At least 5 percent and not less than ten bales used shall be randomly selected and tested.

N103.6 Density. Bales shall have a minimum dry density of 6.5 pounds per cubic foot (92 kg/cubic meter). The dry density shall be calculated by subtracting the weight of the moisture in pounds (kg) from the actual bale weight and dividing by the volume of the bale in cubic feet (cubic meters). At least 2 percent and not less than five bales to be used shall be randomly selected and tested on site.

N103.7 Partial bales. Partial bales made after original fabrication shall be retied with ties complying with N103.4.

SECTION N104
MOISTURE CONTROL

N104.1 General. All weather-exposed bale walls and bale walls enclosing showers or steam rooms, shall be protected from water damage and moisture intrusion in accordance with this section.

N104.2 Water-resistive barriers and vapor permeance ratings. Plastered bale walls shall be constructed without any membrane barrier between straw and plaster to facilitate transpiration of moisture from the bales, and to secure a structural bond between straw and plaster, except as permitted or required elsewhere in this appendix. Where a water-resistive barrier is placed behind the exterior finish, it shall have a vapor permeance rating of at least 5 perms, except as permitted or required elsewhere in this appendix. Wall finishes shall be vapor permeable or shall have an equivalent vapor permeance rating of a Class III vapor retarder.

N104.3 Horizontal surfaces. Bale walls and other bale elements shall be provided with a moisture barrier at all weather-exposed horizontal surfaces. The moisture barrier shall be of a material and installation that will prevent water from entering the wall system. Horizontal surfaces shall include, but shall not be limited to, exterior window sills, sills at exterior niches, and buttresses. The finish material at such surfaces shall be sloped not less than 1 unit vertical in 12 units horizontal (8-percent slope) and shall drain away from all bale walls and elements. Where the moisture barrier is below the finish material, it shall be sloped not less than 1 unit vertical in 12 units horizontal (8-percent slope) and shall drain to the outside surface of the bale’s vertical finish.

N104.4 Bale and concrete separation. A sheet or liquid applied Class II vapor retarder shall be installed between bales and supporting concrete or masonry. The bales shall be separated from the vapor retarder by not less than 3/4-inch (19 mm), and that space shall be filled with an insulating material such as wood or rigid insulation, or a material that allows vapor dispersion such as gravel, or other *approved* insulating or vapor dispersion material. Sill plates in structural walls shall comply with Table N105.14 and Table N105.15. Where bales abut a concrete or masonry wall that retains earth, a Class II vapor retarder shall be provided between such wall and the bales.

N104.5 Separation of bales and earth. Bales shall be separated from earth a minimum of 8” (203 mm).

N104.6 Separation of exterior plaster and earth. Exterior plaster applied to straw bales shall be located not less than 4 inches (102 mm) above the earth or 2 inches (51 mm) above paved areas.

N104.7 Showers walls and steam rooms. Bale walls enclosing showers or steam rooms shall be protected by a water-resistive barrier or by a Class I or Class II vapor retarder on the interior face between the finish and the bales.

SECTION N105
STRUCTURAL USE

N105.1 Scope. This section shall apply to structural strawbale walls. Sections N105.11, N105.12, and N105.16 shall also apply to nonstructural strawbale walls.

N105.2 General. An approved engineered design in accordance with Section N105 and the *International Building Code* shall be provided for buildings or portions thereof using structural strawbale walls.

N105.3 Foundations. Foundations for strawbale walls shall be of any type permitted by, and shall be designed in accordance with, the *International Building Code*.

N105.4 Building height and stories. Building height shall not exceed 35 feet and the limits contained in Table N105.13. Structural use of strawbale walls shall be permitted in multi-story buildings where:

1. Complete vertical and lateral load paths are demonstrated by an *approved* engineered design.
2. Strawbale walls interrupted by floor assemblies are designed and detailed by a *registered design professional*.

N105.5 Configuration of bales. Bales in structural walls shall be laid flat or on-edge and in a running bond or stack bond, except that bales in structural walls with unreinforced plasters shall be laid in a running bond only.

N105.6 Pre-compression of load-bearing strawbale walls. Prior to application of plaster, walls designed to be load-bearing shall be pre-compressed by a uniform load of not less than 100 pounds per linear foot.

N105.7 Voids and stuffing. Voids between bales in structural strawbale walls shall not exceed 4 inches (102 mm) in width, and such voids shall be stuffed with flakes of straw or straw-clay, before application of finish.

N105.8 Plaster skins. Plaster skins on structural walls shall be of any type permitted by Section N106, except gypsum plaster, and shall be in accordance with Tables N105.14 and N105.15.

N105.8.1 Straightness. Plaster skins on structural strawbale walls shall be straight, as a function of the bale wall surfaces they are applied to, as follows:

1. As measured across the face of a bale, straw bulges shall not protrude more than 3/4 inch (19 mm) across 2 feet (610 mm) of its height or length.

2. As measured across the face of a bale wall, straw bulges shall not protrude from the vertical plane of a bale wall more than 2 inches (51 mm) over 8 feet (2438 mm).

3. The vertical face of adjacent bales shall not be offset more than 1/2 inch (13 mm)

N105.8.2 Plaster and membranes. Structural strawbale walls shall not have a membrane between straw and plaster, or shall have attachment through the bale wall from one plaster skin to the other in accordance with an *approved* engineered design.

N105.9 Transfer of loads to and from plaster skins. Where plastered strawbale walls are used to support superimposed vertical loads, such loads shall be transferred to the plaster skins by continuous direct bearing or by an *approved* engineered design. Where plastered strawbale walls are used to resist in-plane lateral loads, such loads shall be transferred via the reinforcing mesh from the structural member or assembly above and to the sill plate in accordance with Table N105.15, or by an *approved* engineered design.

N105.10 Support of plaster skins. Plaster skins for structural strawbale walls shall be continuously supported along their bottom edge to facilitate the transfer of loads to the foundation system. Acceptable supports include, but are not limited to: a concrete or masonry stem wall, a concrete slab on grade, a wood-framed floor adequately blocked, with an *approved* engineered design, or a steel angle adequately anchored, with an *approved* engineered design. A conventional metal or plastic weep screed is not an acceptable support.

N105.11 Unrestrained wall height. Strawbale walls shall not exceed the ratios of stacked bale height to bale thickness between restraints, as stated in Section 2505.12, except where an *approved* engineered design demonstrates the wall will resist buckling from superimposed vertical loads and out-of-plane design loads.

N105.12 Resistance to out-of-plane lateral loads. Structural and non-structural strawbale walls shall be considered capable of resisting out-of-plane loads prescribed in the *International Building Code* with the following limitations and requirements, except where an *approved* engineered design is provided:

1. Walls with unreinforced plasters or a non-plaster finish, and without pins in accordance with N105.12.4, or other *approved* means of out-of-plane bracing, shall not exceed a 5:1 ratio of stacked bale height to bale thickness.
2. Clay plaster walls with reinforced plasters, or pins in accordance with N105.12 Item 4, or other *approved* means of out-of-plane bracing, shall not exceed the ratio indicated in Equation 24-1. Plaster reinforcement shall be any type described in Table N105.15 with staples spaced not more than 6 inches (152 mm) on center.

$H^2/T = 65$ **(Equation N-1)**

Where:

H = stacked bale height
T = bale thickness
H and T are measured in feet. (H^2/T = 19,800 when H and T are measured in mm)

3. Cement, cement-lime, lime, or soil cement plaster walls with reinforced plasters, or pins in accordance with N105.12 Item 4, or other *approved* means of out-of-plane bracing, shall not exceed the ratio indicated in Equation 24-2. Plaster reinforcement shall be any type described in Table N105.15 with staples spaced not more than 6 inches (152 mm) on center.

$H^2/T = 80$ **(Equation N-2)**

Where:

H = stacked bale height
T = bale thickness
H and T are measured in feet. (H^2/T = 24,400 when H and T are measured in mm)

4. Pins shall be in accordance with an *approved* engineered design or shall comply with the following:
   - 4.1 Pins shall be 3/8 inch (10 mm) diameter steel, 3/4 inch diameter (19 mm) wood, or 1/2 inch diameter (13 mm) bamboo. Pins shall be external or internal.
   - 4.2 External pins shall be installed on both sides of the wall spaced not more than 24 inches (610 mm) on center.
   - 4.3 External pins shall have full lateral bearing on the sill plate and the roof- or floor-bearing member, and shall be tightly tied through the wall to an opposing pin with ties spaced not more than 30 inches (762 mm) apart and not more than 15 inches (381 mm) from each end.
   - 4.4 Internal pins shall be installed vertically not more than 24 inches (610 mm) on center in the center third of the bales, and shall extend from top course to bottom course.

4.5 The bottom course shall be similarly connected to its support and the top course shall be similarly connected to the roof- or floor-bearing member above with pins or other *approved* means.

4.6 Internal pins shall be continuous or shall overlap through not less than one bale course.

N105.13 Design coefficients and factors for seismic design. The values given in Table N105.13 shall apply to seismic design using strawbale shear walls detailed in accordance with Table N105.15.

N105.14 Load-bearing strawbale walls. Load-bearing strawbale walls shall be in accordance with Table N105.14 as part of an *approved* engineered design to support superimposed vertical loads.

N105.15 Strawbale shear walls. Strawbale shear walls shall be in accordance with Table N105.13 as part of an *approved* engineered design to resist in-plane lateral loads. Other *approved* in-plane lateral load resisting systems shall be permitted to be used in combination with strawbale shear walls with apportionment of design loads as prescribed in the *International Building Code*.

N105.16 Connection of light-frame walls to strawbale walls. Light-frame walls perpendicular to, or at an angle to a straw bale wall assembly, shall be fastened to the bottom and top wood members of the strawbale wall in accordance with requirements for wood or cold-formed steel light-frame walls in the *International Building Code*, or the abutting stud shall be connected to alternating straw bale courses with a 1/2 inch (13mm) diameter steel, 3/4" diameter (19 mm) wood, or 5/8" diameter (16 mm) bamboo dowel, with minimum 8 inch (203 mm) penetration.

TABLE N105.13
DESIGN COEFFICIENTS AND FACTORS FOR SEISMIC-FORCE-RESISTING SYSTEMS

Seismic-Force-Resisting System		**Response Modification Coefficient, *R*[1]**	**System Overstrength Factor, Omega[2]**	**Deflection Amplification Factor, *C***	**Structural System Limitations and Building Height (ft) Limits**				
					Seismic Design Category				
					B	C	D	E	F
A. Bearing Wall Systems									
Strawbale shear walls		3.5	3	3	25	25	15	15	15
B. Building Frame Systems									
Strawbale shear walls		4	3	3.5	35	35	25	25	25

a. R reduces forces to a strength level, not an allowable stress level

b. The tabulated value of the overstrength factor is permitted to be reduced by subtracting 0.5 for structures with flexible diaphragms, but shall not be taken as less than 2.0 for any structure.

TABLE N105.14
ALLOWABLE GRAVITY LOADS (LBS./FOOT) FOR PLASTERED STRAWBALE WALLS

WALL DESIGNATION	**PLASTER (both sides) Thickness each side**	**SILL PLATES[b,c]**	**ANCHOR[c] BOLTS (or other sill fastening)**	**MESH[d]**	**STAPLES[e,f,g]**	**ALLOWABLE BEARING CAPACITY[h] (plf)**
A	Clay[l] 1-1/2"	c	c	None required[i]	None required[i]	400
B	Soil-cement[k] 1"	c	c	d	e,f,g	800
C	Lime[l] 7/8"	c	c	d	e,f,g	500

WALL DESIGNATION	PLASTER (both sides) Thickness each side	SILL PLATES[b,c]	ANCHOR[c] BOLTS (or other sill fastening)	MESH[d]	STAPLES[e,f,g]	ALLOWABLE BEARING CAPACITY[h] (plf)
D	Cement-lime[k] 7/8"	c	c	d	e,f,g	800
E	Cement 7/8"	c	c	d	e,f,g	800

For SI: 1 inch=25.4mm, 1 pound per foot = 14.5939 N/m.

a. Plasters shall conform with Sections N106.9 through N106.12 for makeup and thickness, with Section N105.8.1 for straightness, and with Section N105.10 for support of plaster skins.

b. Sill plates shall support and be flush with each face of the bale wall and shall be preservative-treated where required by the *International Building Code*..

c. For walls supporting gravity loads only or for non-structural walls, sill plates and fastening shall be in accordance with the requirements for wood framed walls in the *International Building Code*. See Table N105.15 for requirements for shear walls.

d. Any metal mesh allowed by this section shall be installed throughout the plaster with minimum 4-inch laps and fastened in accordance with footnote e.

e. Staples shall be at maximum spacing of 2-inches on center, to roof or floor bearing assembly, or as shown in an approved design in accordance with Section N105.9, and at a maximum spacing of 4-inches on center to sill plates.

f. Staples shall be gun staples, stainless steel or electro-galvanized, 16 gauge with 1 ¼-inch legs, 7/16-inch crown; or manually driven staples, galvanized 15 gauge with 7/8-inch legs, 3/16-inch inner spread and rounded shoulder. Other staples shall be permitted to be used as designed by a *registered design professional*. Staples into preservative-treated wood shall be stainless steel.

g. Staples shall be firmly driven diagonally across mesh intersections at the spacing indicated.

h. For walls with a different plaster on each side, the lower value shall be used.

i. Except as necessary to transfer roof or floor loads to the plaster skins in accordance with Section N105.9.

j. The building official is authorized to require a cube compression test to demonstrate a minimum 100 psi compressive strength.

k. The building official is authorized to require a compression test to demonstrate a minimum 1000 psi compressive strength.

l. Lime plaster shall use hydraulic or natural hydraulic lime. The building official is authorized to require a cube compression test to demonstrate a minimum 600 psi compressive strength.

m. The building official is authorized to require a cube compression test to demonstrate a minimum 1400 psi compressive strength.

TABLE N105.15
ALLOWABLE SHEAR (POUNDS PER FOOT) FOR PLASTERED STRAWBALE WALLS[a]

DESIGNATION	PLASTER[b]		SILL PLATES[d]	ANCHOR[d] BOLTS (on center)	MESH[e]	STAPLES[f, g, h] (on center)	ALLOWABLE SHEAR[i, j, k] (plf)
	TYPE	THICK-NESS (each side)					
A1	Clay[m]	1.5-in.	2 x 4	2 ft. 8 in.	None	None	60
A2	Clay[m]	1.5-in.	2 x 4	2 ft. 8 in.	2 in. by 2 in. high-density polypropylene	2-inches	140
A3	Clay[m]	1.5-in,	2 x 4	2 ft. 8 in.	2"x2"x14ga[l]	4-inches	180
B	Soil-cement[o]	1-in.	4 x 4	2 ft. 0 in	2 in. by 2 in. by 14ga[l]	2-inches	520
C1	Lime[n]	7/8-in.	2 x 4	2 ft. 8 in.	17 ga. woven wire	3-inches	330
C2	Lime[n]	7/8-in.	4 x 4	2 ft. 0 in.	2 in. by 2 in. by 14ga[l]	2-inches	450
D1	Cement-lime[o]	7/8-in.	4 x 4	2 ft. 8 in	17 ga. woven wire	2-inches	380
D2	Cement-lime[o]	7/8-in.	4 x 4	2 ft. 0 in.	2 in. by 2 in. by 14ga[l]	2-inches	520
E1	Cement[p]	7/8-in.	4 x 4	2 ft. 8 in.	2 in. by 2 in. by 14 ga[l]	2-inches	540
E2	Cement[p]	1.5-in.	4 x 4	2 ft. 0 in.	2 in. by 2 in. by 14ga[l]	2-inches	680

SI: 1 inch=25.4 mm, 1 pound per foot = 14.5939 N/m

a. Bales shall be not less than 15 inches thick.
b. Plasters shall comply with Sections N106.7 through N106.12 for makeup and thickness, with Section N105.8.1 for straightness, and with Section N105.10 for support.
c. Sill plates shall be Douglas fir-larch or southern pine and shall be preservative-treated where required by the *International Building Code*. Multiply allowable shear value by .82 for other species with specific gravity of .42 or greater, or by .65 for all other species.
d. Anchor bolts shall be 5/8-inch diameter with 2-inch by 2-inch by 3/16-inch washers, with not less than 7-inch embedment in concrete or masonry foundation. Anchor bolts or other fasteners into framed floors shall be engineered.
e. Mesh shall run continuous vertically from sill plate to top plate, roof or floor beam, or roof or floor bearing assembly, or shall lap not less than 12-inches. Horizontal laps shall be a not less than 4-inches. Steel mesh shall be galvanized. Galvanized steel mesh shall be separated from preservative-treated wood by grade D paper, 15# roofing felt, or other *approved* barrier.
f. Staples shall be gun staples, stainless steel or electro-galvanized, 16 gauge with 1 ¼-inch legs, 7/16-inch crown; or manually driven staples, galvanized 15 gauge with 7/8-inch legs, 3/16-inch inner spread and rounded shoulder. Other staples shall be permitted to be used as designed by a registered design professional. Staples into preservative-treated wood shall be stainless steel.
g. Staples at spacing indicated are to boundary conditions, including sill plates, and top plate, roof or floor beam, or roof or floor bearing assembly,
h. Staples shall be firmly driven diagonally across mesh intersections at spacing indicated.
i. Values shown are for aspect ratios of 1:1 or less. Reduce values shown to 50 percent for the limit of a 2:1 aspect ratio. Linear interpolation shall be permitted for ratios between 1:1 and 2:1. The full value shown shall be used for aspect ratios greater than 1:1, where an additional layer of mesh is installed at the base of the wall to a height where the remainder of the wall has an aspect ratio of 1:1 or less, and the second layer of mesh is fastened to the sill plate with the required stapling, and the sill bolt spacing is decreased with linear interpolation between1:1 and 2:1.
j. For walls with a plaster Type A on one side and any other plaster type on the other side, a registered design professional shall show transfer of the design lateral load into the stiffer Type B, C, D, or E plaster only, and 50% of the allowable shear value shown for that wall type shall be used.
k. These values are permitted to be increased 40 percent for wind design.
l. 16 gauge mesh shall be permitted to be used with a reduction to 0.60 of the allowable shear values shown.
m. The building official is authorized to require a cube compression test demonstrating not less than 600 psi compressive strength.
n. Lime plaster shall use hydraulic or natural hydraulic lime. The building official is authorized to require a cube compression test demonstrating not less than 600 psi compressive strength.
o. The building official is authorized to require a cube compression test demonstrating not less than 1000 psi compressive strength.
p. The building official is authorized to require a cube compression test demonstrating not less than 1400 psi compressive strength.

SECTION N106
FINISHES

N106.1 General. Finishes applied to strawbale walls shall be any type permitted by the *International Building Code*, and shall comply with this section and with Chapters 14 and 25 unless stated otherwise in this section.

N106.2 Purpose, and where required. Strawbale walls shall be finished so as to provide mechanical protection, fire resistance, restrict the passage of air through the bales, and protect them from weather in accordance with this appendix and the *International Building Code*.

> **Exception:** Truth windows shall be permitted where a fire-resistive rating is not required. Weather-exposed truth windows shall be fitted with a weather-tight cover.

N106.3 Vapor retarders. Class I and Class II vapor retarders shall not be used on a strawbale walls, nor shall any other material be used that has a vapor permeance rating of less than 5 perms, except as permitted or required elsewhere in this appendix, or as *approved* and demonstrated to be necessary by a *registered design professional*.

N106.4 Plaster. Plaster applied to bales shall be of any type described in Section N106, and as required or limited in this appendix.

N106.5 Plaster and membranes. Plaster shall be applied directly to strawbale walls to facilitate transpiration of moisture from the bales, and to secure a mechanical bond between the skin and the bales, except where a membrane is allowed or required elsewhere in this appendix. Structural bale walls shall have no membrane between straw and plaster, or shall have attachment through the bale wall from one plaster skin to the other in accordance with an approved engineered design.

N106.6 Lath and mesh for plaster. The surface of the straw bales functions as lath, and no other lath or mesh shall be required, except as required for tensile or shear strength in structural applications as required in Table N105.14, Table N105.15, or by an *approved* engineered design.

N106.7 Plaster on non-structural walls. Plaster on non-structural walls shall be in accordance with Section N106.9, N106.10, N106.11, N106.12, N106.13 or N106.14.

N106.8 Plaster on structural walls. Plaster on structural walls shall comply with Section N106.9, N106.10, N106.11, N106.12, N106.13 or N106.14. Plaster on load-bearing walls shall also comply with Table N105.14. Plaster on shear walls shall also comply with Table N105.15.

N106.9 Clay plaster. Clay plaster shall comply with Sections N106.9.1 through N106.9.6.

N106.9.1 General. Clay plaster shall be any plaster having a clay or clay soil binder. Such plaster shall contain sufficient clay to fully bind the plaster, sand or other inert granular material, and shall be permitted to contain reinforcing fibers. Reinforcing fibers shall include, but shall not be limited to, chopped straw, sisal, and animal hair.

N106.9.2 Mesh. Clay plaster shall not be required to contain reinforcing mesh except as required in Table N105.15. Where provided, mesh shall be natural fiber, corrosion-resistant metal, nylon mesh, or high-density polypropylene.

N106.9.3 Thickness and coats. Clay plaster shall be a minimum 1 inch (25 mm) thick, unless required to be thicker for structure or fire-resistance, as described elsewhere in this appendix, and shall be applied with in not less than two coats.

N106.9.4 Rain-exposed. Clay plaster, where exposed to rain, shall be finished with lime wash, linseed oil, or other *approved* erosion resistant finish.

N106.9.5 Prohibited finish coat. Cement plaster shall not be permitted as a finish coat over clay plasters.

N106.9.6 Additives. Additives shall be permitted to increase the plaster's workability, durability, strength, or water resistance.

N106.10 Soil-cement plaster. Soil-cement plaster shall comply with Sections N106.10.1 through N106.10.3.

N106.10.1 General. Soil-cement plaster shall be comprised of soil (free of organic matter), sand, and not less than10 percent Portland cement by volume, and shall be permitted to contain reinforcing fibers.

N106.10.2 Mesh. Soil-cement plaster shall use any corrosion-resistant metal mesh permitted by the *International Building Code*, or as required in Section N105 where used on a structural wall.

N106.10.3 Thickness. Soil-cement plaster shall be not less than 1 inch (25 mm) thick.

N106.11 Gypsum plaster. Gypsum plaster shall comply with Section 2511 of the *International Building Code*. Gypsum plaster shall be limited to use on interior surfaces, and on non-structural walls, except as an interior finish coat over a structural plaster that complies with this appendix.

N106.12 Lime plaster. Lime plaster shall comply with Sections N106.12.1 and N106.12.2.

N106.12.1 General. Lime plaster is any plaster whose binder is comprised of calcium hydroxide (CaOH) including Type N or Type S hydrated lime, hydraulic lime, natural hydraulic lime, or quicklime. Hydrated lime plasters shall comply with ASTM C 206. Quicklime plasters shall comply with ASTM C 5. Lime plaster shall be permitted to be applied in 2 coats, provided that the combined thickness is at least 7/8 inch (22 mm), and each coat is not greater than 1/2 inch (13 mm) thick.

N106.12.2 On structural walls. Lime plaster on structural strawbale walls in accordance with Table N105.14 or Table N105.15 shall use hydraulic or natural hydraulic lime.

N106.13 Cement-lime plaster. Cement-lime plaster shall be plaster mixes CL or FL as described in ASTM C 926. Cement-lime plaster shall be permitted to be applied in 2 coats, provided the combined thickness is at least 7/8 inch (22 mm) thick, and each coat is not greater than 1/2 inch (13 mm) thick.

N106.14 Cement plaster. Cement plaster shall comply with Section 2512 of the *International Building Code*, except that the amount of lime in all plaster coats shall be not less than 1 part lime to 6 parts cement to allow a minimum acceptable vapor permeability. The plaster shall be permitted to be applied in 2 coats, provided the combined thickness is at least 7/8 inch (22 mm), and each coat is not greater than 1/2 inch (13 mm) thick. The combined thickness of all plaster coats shall be not more than 1 1/2 inch (38 mm) thick.

N106.15 Finishes over plaster. Other finishes, as permitted elsewhere in this section and the *International Building Code*, shall be permitted to be applied over the plaster, except as prohibited in Section N106.16.

N106.16 Prohibited plasters and finishes. Any plaster or finish with a singular or cumulative perm rating less than 5 perms shall be prohibited on straw bale walls, except where approved and demonstrated to be necessary by a *registered design professional*, or as required elsewhere in this appendix.

N106.17 Separation of wood and plaster. Where wood framing or wood sheathing occurs in strawbale walls, such wood surfaces shall be separated from exterior plaster with No. 15 asphalt felt, grade D paper, or other *approved* material in accordance with Section 1404.2 of the *International Building Code*, except where the wood is preservative-treated or naturally durable.

Exception: Exterior clay plasters shall not be required to be separated from wood.

SECTION N108
THERMAL INSULATION

N108.1 R-value. The unit R-value of a strawbale wall with bales laid flat is R-1.3 per inch, and with bales on-edge is R-2 per inch.

Reason: Strawbale construction has proven to be a safe, durable, resource efficient, and fully viable method of construction. However, the International Building Code does not contain a section on strawbale construction, which has been an impediment to this construction system's proper and broader use.

First practiced in Nebraska in the late 1800's, with buildings over 100 years old still in service, strawbale construction was rediscovered in the 1980's in the American southwest. Since then it has been further developed and explored, including considerable testing and research regarding structural performance (under vertical and lateral loads), moisture, fire, and its thermal and acoustic properties.

Currently only Oregon and New Mexico have adopted statewide strawbale building codes. California has legislated strawbale construction guidelines that are voluntarily adopted at the local level. In addition, nine U.S. cities or counties have adopted strawbale building codes. Three countries outside the United States – Germany, France, and Belarus - have limited strawbale building codes.

Most of the strawbale building codes that do exist are derived from the first such code, created for and adopted by Tucson / Pima County, Arizona in 1996. Much experience, testing, and research since then have proven these codes to be deficient. They are often either too restrictive, or not restrictive enough, and in some cases don't address important issues at all.

Although strawbale codes are both few and flawed, strawbale buildings are now found in 49 of the 50 United States, and strawbale construction is practiced in over 45 countries throughout the world and in every climate. There are an estimated 600-1000 strawbale buildings in California alone. The strawbale buildings in the U.S. include residences, schools, office buildings, wineries, multi-story buildings, buildings over 10,000 sq.ft in floor area, load-bearing strawbale structures, and structures in areas of high seismic risk (plastered strawbale walls are particularly resistant to earthquakes). The practice of, and the desire to utilize strawbale construction, continues to increase and promises to accelerate as we face increased pressure on our environment and natural resources.

There is great need for a comprehensive strawbale code, with full benefit of the experience and knowledge that has been gained to date about this method of construction. The following proposed Strawbale Construction appendix for the IBC was created to fulfill this need. It is based on the collective experience of the design, construction, and testing of strawbale buildings over 20 years by architects, engineers, builders, and academics throughout the U.S., Canada, and other countries throughout the world. The testing, research, and comprehensive understanding of the performance of strawbale buildings are summarized in the book *Design of Straw Bale Buildings* (B.King, et al, 2006, Green Building Press). Testing, research reports, and other supporting documentation are available for viewing and download at: http://www.ecobuildnetwork.org/strawbale-construction-code-supporting-documentation

As lead author of the proposed appendix, and as a licensed architect for 25 years, I have been involved in the design, construction, testing, and research of strawbale buildings since 1995. In 2001 I spearheaded legislation and revisions to the current California Guidelines for Straw-Bale Structures. The proposed Strawbale Construction appendix for the IBC has benefited from numerous peer reviews by experienced, licensed design and building professionals over the course of more than five years. It would serve designers, builders, owners, inhabitants, and building officials alike in the construction and utilization of strawbale buildings.

Supporting Documentation: List of selected documents available via the above link

Load-Bearing Straw Bale Construction – A summary of worldwide testing and experience, B.King, PE
Testing of Straw Bale Walls with Out-of-Plane Loads – K.Donahue, SE
In-Plane Cyclic Tests of Plastered Straw Bale Wall Assemblies – C.Ash, M.Aschheim, PE, D.Mar, SE
Structural Testing of Plastered Straw Bale Wall Assemblies – K.Lerner, Architect, K.Donahue, SE
Seismic Design Factors and Allowable Shears for Strawbale Wall Assemblies – S. Jalali, M. Aschheim, PE
Shake Table Test Video of Full Scale Straw Bale Building Specimen – D.Donovan, PE
Moisture Properties of Plaster and Stucco for Strawbale Buildings – J.Straube, PE
Monitoring of Hygrothermal Performance of Strawbale Walls – J.Sraube, PE, C.Schumacher
ASTM E119 1-Hour Fire Resistance Test of a Non-Loadbearing Straw Bale Wall with Clay Plaster
ASTM E119 2-Hour Fire Resistance Test of a Non-Loadbearing Straw Bale Wall with Cement Plaster
ASTM E119 Fire Tests - Video
Thermal Performance of Straw Bale Wall Systems (incl. Oak Ridge Lab test results) – N.Stone
Support Letters from Licensed Practitioners: Letters from 2 Structural Engineers, 4 Civil Engineers, 1 Professor of Civil Engineering, 7 Architects

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

APPENDIX N (NEW)-S-HAMMER-AB2-15-12.doc

Public Hearing Results

PART I - IBC STRUCTURAL
Committee Action: **Disapproved**

Committee Reason: There is the same concern for this proposal as an Appendix as there is for S316-12. Even though it is optional as an appendix, when the appendix is adopted it would become mandatory.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment 1:

Martin Hammer, representing California Straw Building Association, Colorado Straw Bale Association, Straw Bale Construction Association – New Mexico, Ontario Straw Bale Building Coalition, Development Center for Appropriate Technology, Ecological Building Network, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

APPENDIX N
STRAWBALE CONSTRUCTION

The provisions contained in this appendix are not mandatory unless specifically referenced in the adopting ordinance.

SECTION N101
GENERAL

N101.1 Scope. ~~This appendix shall govern the use of baled straw as a building material.~~ This appendix provides prescriptive and performance-based requirements for the use of baled straw as a building material. Other methods of strawbale construction shall be subject to approval in accordance with Section 104.11

SECTION N102
DEFINITIONS

N102.1 Definitions. The following words and terms shall, for the purposes of this appendix, have the meanings shown herein. Refer to Chapter 2 of the *International Building Code* for general definitions.

BALE. Equivalent to straw bale.

CLAY. Inorganic soil with particle sizes less than 0.00008 in. (0.002 mm) having the characteristics of high to very high dry strength and medium to high plasticity.

CLAY SLIP. A suspension of clay particles in water.

FINISH. Completed assembly of materials on the interior or exterior faces of stacked bales.

FLAKE. An intact section of compressed straw removed from an untied bale.

LAID FLAT. The orientation of a bale with its largest faces horizontal, its longest dimension parallel with the wall plane, its ties concealed in the unfinished wall and its straw lengths oriented across the thickness of the wall.

LOAD-BEARING WALL. For the purposes of this appendix, any strawbale wall that supports more than 100 lb/linear ft (1,459 N/m) of vertical load in addition to its own weight.

MESH. An openwork fabric of linked strands of metal, plastic, or natural or synthetic fiber, embedded in plaster ~~to provide tensile reinforcement or bonding~~.

NONLOAD-BEARING WALL. For the purpose of this appendix, any wall that is not a load-bearing wall.

NONSTRUCTURAL WALL. All walls other than load-bearing walls or shear walls.

ON-EDGE. The orientation of a bale with its largest faces vertical, its longest dimension parallel with the wall plane, its ties on the face of the wall, and its straw lengths oriented vertically.

PIN. A vertical metal rod, wood dowel, or bamboo, driven into the center of stacked bales, or placed ~~through-tied~~ on ~~the~~opposite surfaces of stacked bales and through-tied~~for the purpose of connection or stability~~.

PLASTER. Gypsum, lime, cement-lime, or cement plasters, as defined in Chapter 25 and in Section N106, or clay plaster as defined in Section ~~N106.9~~ N104.7, or soil-cement plaster as defined in Section ~~N106.10~~ N104.8.

PRE-COMPRESSION. Vertical compression of stacked bales before the application of finish.

REINFORCED PLASTER. A plaster containing mesh reinforcement.

RUNNING BOND. For the purposes of this appendix, the placement of straw bales such that the head joints in successive courses are offset at least one quarter the bale length.

SHEAR WALL. A strawbale wall designed to resist lateral forces parallel to the plane of the wall in accordance with Section ~~N105.15~~ N106.15.

SKIN. The compilation of plaster and reinforcing, if any, applied to the surface of stacked bales.

STRUCTURAL WALL. A wall that meets the definition for a load-bearing wall or shear wall.

STACK BOND. For the purposes of this appendix, the placement of straw bales such that head joints in successive courses are vertically aligned.

STRAW. The dry stems of cereal grains after the seed heads have been removed.

STRAW BALE. A rectangular compressed block of straw, bound by ties.

STRAWBALE. The adjective form of straw bale.

STRAW-CLAY. Loose straw mixed and coated with clay slip.

TIE. A synthetic fiber, natural fiber, or metal wire used to confine a straw bale.

TRUTH WINDOW. An area of a strawbale wall left without its finish, to allow view of the straw otherwise concealed by its finish.

SECTION N103
BALES

~~**N103.1 Types of straw.** Bales shall be composed of straw from wheat, rice, rye, barley, or oat.~~

~~**N103.2**~~ <u>**N103.1**</u> **Shape.** Bales shall be rectangular in shape.

~~**N103.3**~~ <u>**N103.2**</u> **Size**. Bales shall have a minimum height and thickness of 12 inches (305 mm), except as otherwise permitted or required in this appendix. Bales used within a continuous wall shall be of consistent height and thickness to ensure even distribution of loads within the wall system.

~~**N103.4**~~ <u>**N103.3**</u> **Ties.** Bales shall be confined ~~with~~ <u>by</u> synthetic fiber, natural fiber, or metal ties sufficient to maintain required bale density. Ties shall be at least 3 inches (76 mm) and not more than 6 inches (152 mm) from bale faces and shall be spaced not more than 12 (305 mm) inches apart. Bales with broken ties shall be retied with sufficient tension to maintain required bale density.

~~**N103.5**~~ <u>**N103.4**</u> **Moisture content.** The moisture content of bales at the time of application of the first coat of plaster or the installation of another finish shall not exceed 20 percent of the weight of the bale. The moisture content of bales shall be determined by use of a moisture meter designed for use with baled straw or hay, equipped with a probe of sufficient length to reach the center of the bale. At least 5 percent and not less than ten bales used shall be randomly selected and tested.

~~**N103.6**~~ <u>**N103.5**</u> **Density.** Bales shall have a minimum dry density of 6.5 pounds per cubic foot (92 kg/cubic meter). The dry density shall be calculated by subtracting the weight of the moisture in pounds (kg) from the actual bale weight and dividing by the volume of the bale in cubic feet (cubic meters). At least 2 percent and not less than five bales to be used shall be randomly selected and tested on site.

~~**N103.7**~~ <u>**N103.6**</u> **Partial bales.** Partial bales made after original fabrication shall be retied with ties complying with ~~N103.4~~ <u>N103.3</u>.

<u>**N103.7 Types of straw.** Bales shall be composed of straw from wheat, rice, rye, barley, or oat.</u>

<u>**N103.8 Other baled material.** The dry stems of other cereal grains or grasses shall be acceptable when *approved* by the *building official*.</u>

SECTION ~~N106~~ <u>N104</u>
FINISHES

~~**N106.1**~~ <u>**N104.1**</u> **General.** Finishes applied to strawbale walls shall be any type permitted by the *International Building Code*, and shall comply with this section and with Chapters 14 and 25 unless stated otherwise in this section.

~~**N106.2**~~ <u>**N104.2**</u> **Purpose, and where required.** Strawbale walls shall be finished so as to provide mechanical protection, fire resistance, ~~restrict the passage of air through the bales, and protect them~~ <u>protection</u> from weather<u>, and to restrict the passage of air through the bales,</u> in accordance with this appendix and the *International Building Code*.

> **Exception:** Truth windows shall be permitted where a fire-resistive rating is not required. Weather-exposed truth windows shall be fitted with a weather-tight cover.

~~N106.3~~ N104.3 Vapor retarders. Class I and Class II vapor retarders shall not be used on a strawbale wall~~s~~, nor shall any other material be used that has a vapor permeance rating of less than 5 perms, except as permitted or required elsewhere in this appendix~~, or as *approved* and demonstrated to be necessary by a *registered design professional*~~.

~~N106.4~~ N104.4 Plaster. Plaster applied to bales shall be ~~of~~ any type described in this section ~~N106~~, and as required or limited in this appendix.

~~N106.5~~ N104.5 Plaster and membranes. Plaster shall be applied directly to strawbale walls to facilitate transpiration of moisture from the bales, and to secure a mechanical bond between the skin and the bales, except where a membrane is allowed or required elsewhere in this appendix. ~~Structural bale walls shall have no membrane between straw and plaster, or shall have attachment through the bale wall from one plaster skin to the other in accordance with an *approved* engineered design.~~

~~N106.6~~ N104.6 Lath and mesh for plaster. The surface of the straw bales functions as lath, and no other lath or mesh shall be required, except as required for structural walls.~~for tensile or shear strength in structural applications as required in Table N105.14, Table N105.15, or by an *approved* engineered design.~~

~~**N106.7 Plaster on non-structural walls.** Plaster on non-structural walls shall be in accordance with Section N106.9, N106.10, N106.11, N106.12, N106.13 or N106.14.~~

~~**N106.8 Plaster on structural walls.** Plaster on structural walls shall comply with Section N106.9, N106.10, N106.11, N106.12, N106.13 or N106.14. Plaster on load-bearing walls shall also comply with Table N105.14. Plaster on shear walls shall also comply with Table N105.15.~~

~~N106.9~~ N104.7 Clay plaster. Clay plaster shall comply with ~~Sections~~ N10~~6~~4.~~ 9~~7.1 through N10~~6~~4.~~ 9~~7.6.

~~N106. 9.1~~ N104.7.1 General. Clay plaster shall be any plaster having a clay or clay soil binder. Such plaster shall contain sufficient clay to fully bind the plaster, sand or other inert granular material, and shall be permitted to contain reinforcing fibers. Acceptable reinforcing fibers ~~shall~~ include, ~~but shall not be limited to,~~ chopped straw, sisal, and animal hair.

~~N106. 9.2~~ N104.7.2 Mesh. Clay plaster shall not be required to contain reinforcing mesh except as required in Table N105.15. Where provided, mesh shall be natural fiber, corrosion-resistant metal, nylon ~~mesh~~, or high-density polypropylene.

~~N106. 9.3~~ N104.7.3 Thickness and coats. Clay plaster shall be a minimum 1 inch (25 mm) thick, unless required to be thicker for structure or fire-resistance, as described elsewhere in this appendix, and shall be applied with in not less than two coats.

~~N106. 9.4~~ N104.7.4 Rain-exposed. Clay plaster, where exposed to rain, shall be finished with lime wash, linseed oil, or other *approved* erosion-resistant finish.

~~N106. 9.5~~ N104.7.5 Prohibited finish coat. Cement plaster shall not be permitted as a finish coat over clay plasters.

~~N106. 9.6~~ N104.7.6 Additives. Additives shall be permitted to increase ~~the~~ plaster~~'s~~ workability, durability, strength, or water resistance.

~~N106.10~~ N104.8 Soil-cement plaster. Soil-cement plaster shall comply with ~~Sections~~ ~~N106. 10.1~~ N104.8.1, N104.8.2 and ~~through~~ ~~N106.10.3~~ N104.8.3.

~~N106. 10.1~~ N104.8.1 General. Soil-cement plaster shall be comprised of soil (free of organic matter), sand, and not less than 10% Portland cement by volume, and shall be permitted to contain reinforcing fibers.

~~N106. 10.2~~ N104.8.2 Mesh. Soil-cement plaster shall use any corrosion-resistant metal mesh permitted by the *International Building Code*, or as required in Section N105 where used on a structural wall.

~~N106. 10.3~~ N104.8.3 Thickness. Soil-cement plaster shall be not less than 1 inch (25 mm) thick.

~~N106.11~~ N104.9 Gypsum plaster. Gypsum plaster shall comply with Section 2511 of the *International Building Code*. Gypsum plaster shall be limited to use on interior surface~~s, and on~~ of non-structural walls, ~~except~~ and as an interior finish coat over a structural plaster that complies with this appendix.

~~N106.12~~ N104.10 Lime plaster. Lime plaster shall comply with ~~Sections~~ ~~N106. 12.1~~ N104.10.1 and ~~N106. 12.2~~ N104.10.2.

~~N106. 12.1~~ N104.10.1 General. Lime plaster is any plaster whose binder is comprised of calcium hydroxide (CaOH) including Type N or Type S hydrated lime, hydraulic lime, natural hydraulic lime, or quicklime. Hydrated lime plasters shall comply with ASTM C 206. Quicklime plasters shall comply with ASTM C 5. Lime plaster shall be permitted to be applied in 2 coats, provided that the combined thickness is at least 7/8 inch (22 mm), and each coat is not greater than 1/2 inch (13 mm) thick.

~~N106. 12.2~~ N104.10.2 On structural walls. Lime plaster on structural strawbale walls in accordance with Table N105.14 or Table N105.15 shall use a binder comprised of hydraulic or natural hydraulic lime.

~~N106.13~~ N104.11 Cement-lime plaster. Cement-lime plaster shall be plaster mixes CL or FL as described in ASTM C 926. Cement-lime plaster shall be permitted to be applied in 2 coats, provided the combined thickness is at least 7/8 inch (22 mm) thick, and each coat is no~~t~~ greater than 1/2 inch (13 mm) thick.

~~N106.14~~ N104.12 Cement plaster. Cement plaster shall comply with Section 2512 of the *International Building Code*, except that the amount of lime in all plaster coats shall be not less than 1 part lime to 6 parts cement to allow a minimum acceptable vapor permeability. The plaster shall be permitted to be applied in 2 coats, provided the combined thickness is at least 7/8 inch (22 mm), and each coat is no~~t~~ greater than 1/2 inch (13 mm) thick. The combined thickness of all plaster coats shall be not more than 1 1/2 inch (38 mm) thick.

~~N106.15~~ N104.13 Finishes over plaster. Other finishes, as permitted elsewhere in this section and the *International Building Code*, shall be permitted to be applied over the plaster, except as prohibited in ~~Section N1064.16~~ N104.14.

~~N106.16~~ N104.14 Prohibited plasters and finishes. Any plaster or finish with a singular or cumulative perm rating less than 5 perms shall be prohibited on straw bale walls, ~~except where approved and demonstrated to be necessary by a *registered design professional*,~~ or as required elsewhere in this appendix.

~~N106.17~~ N104.15 Separation of wood and plaster. Where wood framing or wood sheathing occurs in strawbale walls, such wood surfaces shall be separated from exterior plaster with No. 15 asphalt felt, grade D paper, or other *approved* material in accordance with Section 1404.2 of the *International Building Code*, except where the wood is preservative-treated or naturally durable.

Exception: Exterior clay plasters shall not be required to be separated from wood.

SECTION N105
STRAWBALE WALLS – GENERAL

N105.1 General. Strawbale walls shall be designed and constructed in accordance with this section.

N105.2 Finishes. Finishes shall be in accordance with N104.

N105.3 Sill plate attachment to concrete. Sill plate attachment to concrete shall comply with Section 2308.6 except as required in N106.15.

N105.4 Out-of-plane resistance and unrestrained wall height. Strawbale walls shall not exceed the limits of stacked bale height between restraints of Table N105.4, except where an *approved* engineered design demonstrates the wall will resist buckling from superimposed vertical loads and out-of-plane design loads. Lateral resistance perpendicular to the face of the wall shall be provided according to the prescriptive requirements of Table N105.4 in accordance with *allowable stress design,* except where an *approved* engineering design is provided.

TABLE N105.4: OUT-OF-PLANE RESISTANCE AND UNRESTRAINED WALL HEIGHT

Type of Restraint[a]	Maximum allowable lateral loading (pounds per square foot)	Unrestrained Wall Height, H,		Mesh Staple Spacing at Boundary Restraints
		Absolute limit in feet	Slenderness limit[b] in feet (mm)	
Non-plaster finish or unreinforced plaster,	25	H ≤ 10	H ≤ 5T	none
Pins per N105.4.1	25	H ≤ 12	H ≤ 7T	none
Reinforced clay plaster[c]	30	H ≤ 10	$H \le 8T^{0.5}$ ($H \le 140T^{0.5}$)	≤ 6 in (152 mm)
Reinforced clay plaster[c]	30	10 < H ≤ 12	$H \le 8T^{0.5}$ ($H \le 140T^{0.5}$)	≤ 4 in (102 mm)
Reinforced cement, cement-lime, lime, or soil-cement plaster[c]	30	H ≤ 10	$H \le 9T^{0.5}$ ($H \le 157T^{0.5}$)	≤ 6 in (152 mm)
Reinforced cement, cement-lime, lime, or soil-cement plaster[c]	40	10 < H ≤ 13	$H \le 9T^{0.5}$ ($H \le 157T^{0.5}$)	≤ 4 in (102 mm)

For SI: 1 foot = 304.8 mm, 1 pound per square foot = 47.8803 N/m^2

[a] Finishes applied to both sides of stacked bales. Where different finishes are used on opposite sides of a wall, the more restrictive requirements shall apply.

[b] H = stacked bale height in feet (mm), or the horizontal distance in feet (mm) between vertical restraints. T= bale thickness in feet (mm)

[c] Plaster reinforcement must conform to Table N106.15

N105.4.1 Pins. Pins used for out-of-plane resistance shall comply with items a. and b. or items a. and c., or shall be in accordance with an *approved* engineered design:

a. External and internal pins shall be 3/8 inch (10 mm) diameter steel, 3/4 inch (19 mm) diameter wood, or 1/2 inch (13 mm) diameter bamboo.

b. External pins shall be installed vertically on both sides of the wall spaced not more than 24 inches (610 mm) on center. External pins shall have full lateral bearing on the sill plate and the roof- or floor-bearing member, and shall be tightly tied through the wall to an opposing pin with ties spaced not more than 32 inches (762 mm) apart and not more than 6 inches (381 mm) from each end of the pin.

c. Internal pins shall be installed vertically within the center third of the bales, at spacing not exceeding 24 inches (610 mm) and shall extend from top course to bottom course. The bottom course shall be similarly connected to its support and the top course shall be similarly connected to the roof- or floor-bearing member above with pins or other *approved* means. Internal pins shall be continuous or shall overlap through not less than one bale course.

~~N105.16~~ N105.5 Connection of light-frame walls to strawbale walls. Light-frame walls perpendicular to, or at an angle to a straw bale wall assembly, shall be fastened to the bottom and top wood members of the strawbale wall in accordance with requirements for wood or cold-formed steel light-frame walls in the *International Building Code*, or the abutting stud shall be connected to alternating straw bale courses with a 1/2 inch (13mm) diameter steel, 3/4" diameter (19 mm) wood, or 5/8" diameter (16 mm) bamboo dowel, with minimum 8 inch (203 mm) penetration.

~~SECTION N104~~
~~MOISTURE CONTROL~~

~~N104.1~~ N105.6 ~~General~~Moisture control. All exterior surfaces of weather-exposed bale walls and interior surfaces of bale walls enclosing showers or steam rooms, shall be protected from ~~water~~ moisture damage and ~~moisture~~ intrusion in accordance with this sub-section.

~~N104.2~~ N105.6.1 Water-resistive barriers and vapor permeance ratings. Plastered bale walls shall be constructed without any membrane barrier between straw and plaster to facilitate transpiration of moisture from the bales, and to secure a structural bond between straw and plaster, except as permitted or required elsewhere in this appendix. Where a water-resistive barrier is placed behind ~~the~~ an exterior finish, it shall have a vapor permeance rating of at least 5 perms, except as permitted or required elsewhere in this appendix. Wall finishes ~~shall be vapor permeable or~~ shall have an equivalent vapor permeance rating of a Class III vapor retarder.

N105.6.2 Vapor retarders. Wall finishes shall have an equivalent vapor permeance rating of a Class III vapor retarder, except that a Class I or Class II vapor retarder shall be provided on the interior of side of exterior strawbale walls in Climate Zones 5, 6, 7, 8 and Marine 4 as defined in Chapter 3 of the *International Energy Conservation Code*. Bales in walls enclosing showers or steam rooms shall be protected on the interior side by a Class I or Class II vapor retarder.

N105.6.3 Penetrations in exterior strawbale walls. Penetrations in exterior strawbale walls shall be sealed with an *approved* sealant or gasket on the exterior side of the wall, and on the interior sided of the wall in Climate Zones 5, 6, 7, 8 and Marine 4 as defined in Chapter 3 of the *International Energy Conservation Code*.

~~N104.3~~ N105.6.4 Horizontal surfaces. Bale walls and other bale elements shall be provided with a moisture barrier at all weather-exposed horizontal surfaces. The moisture barrier shall be of a material and installation that will prevent water from entering the wall system. Horizontal surfaces shall include, but shall not be limited to, exterior window sills, sills at exterior niches, and buttresses. The finish material at such surfaces shall be sloped not less than 1 unit vertical in 12 units horizontal (8-percent slope) and shall drain away from all bale walls and elements. Where the moisture barrier is below the finish material, it shall be sloped not less than 1 unit vertical in 12 units horizontal (8-percent slope) and shall drain to the outside surface of the bale's vertical finish.

~~N104.4~~ N105.6.5 Bale and concrete separation. A sheet or liquid applied Class II vapor retarder shall be installed between bales and supporting concrete or masonry. The bales shall be separated from the vapor retarder by not less than 3/4 inch (19 mm), and that space shall be filled with an insulating material such as wood or rigid insulation, ~~or~~ a material that allows vapor dispersion such as gravel, or other *approved* insulating or vapor dispersion material. Sill plates in structural walls shall comply with Table ~~N105.14~~N106.2 and Table ~~N105.15~~N106.3. Where bales abut a concrete or masonry wall that retains earth, a Class II vapor retarder shall be provided between such wall and the bales.

~~N104.5~~ N105.6.6 Separation of bales and earth. Bales shall be separated from earth a minimum of 8" (203 mm).

~~N104.6~~ N105.6.7 Separation of exterior plaster and earth. Exterior plaster applied to straw bales shall be located not less than 4 inches (102 mm) above the earth or 2 inches (51 mm) above paved areas.

~~**N104.7 Showers walls and steam rooms.** Bale walls enclosing showers or steam rooms shall be protected by a water-resistive barrier or by a Class I or Class II vapor retarder on the interior face between the finish and the bales.~~

SECTION ~~N105~~ N106
STRAWBALE WALLS - STRUCTURAL ~~USE~~

~~**N105.1 Scope.** This section shall apply to structural strawbale walls. Sections N105.11, N105.12, and N105.16 shall also apply to nonstructural strawbale walls.~~

~~N105.2~~ N106.1 General. An *approved* engineered design demonstrating complete vertical and lateral load paths in accordance with this section ~~N105~~ and the *International Building Code* shall be provided for buildings or portions thereof ~~using~~ that use structural strawbale walls.

~~N105.3~~ N106.2 Foundations. Foundations ~~for strawbale walls~~ shall be ~~of any type permitted by, and shall be designed~~ in accordance with Chapter 18 of the *International Building Code*.

~~N105.4~~ N106.3 Building height and stories. ~~Building height shall not exceed 35 feet and the limits contained in Table N105.13. Structural use of strawbale walls shall be permitted in multi-story buildings where:~~ Buildings or portions of buildings constructed with structural strawbale walls shall be subject to the following limitations:

1. ~~Complete vertical and lateral load paths are demonstrated by an *approved* engineered design.~~ Building height shall not exceed 35 feet and the limits contained in Table N106.14.

2. ~~Strawbale walls interrupted by floor assemblies are designed and detailed by a *registered design professional*.~~ The number of stories above grade plane shall not exceed two.

3. Structural strawbale walls interrupted by floor assemblies shall be designed and detailed by a *registered design professional*.

~~N105.5~~ N106.4 Configuration of bales. Bales in structural walls shall be laid flat or on-edge and in a running bond or stack bond, except that bales in structural walls with unreinforced plasters shall be laid in a running bond only.

~~N105.6~~ N106.5 Pre-compression of load-bearing strawbale walls. Prior to application of plaster, walls designed to be load-bearing shall be pre-compressed by a uniform load of not less than 100 pounds per linear foot.

~~N105.7~~ N106.6 Voids and stuffing. Voids between bales in structural strawbale walls shall not exceed 4 inches (102 mm) in width, and such voids shall be stuffed with flakes of straw or straw-clay, before application of finish.

~~N105.8~~ N106.7 Plaster on structural walls~~skins~~. Plaster ~~skins~~ on ~~structural~~ loadbearing walls shall be ~~of any type permitted by Section N106, except gypsum plaster, and shall be~~ in accordance with Table~~s~~ ~~N105.14~~N106.13. ~~and N105.15~~ Plaster on shear walls shall be in accordance with Table N106.15.

~~N105.8.1~~ N106.8 Straightness of plaster. Plaster ~~skins~~ on structural strawbale walls shall be straight, as a function of the bale wall surfaces they are applied to, as follows:

1. As measured across the face of a bale, straw bulges shall not protrude more than 3/4 inch (19 mm) across 2 feet (610 mm) of its height or length.

2. As measured across the face of a bale wall, straw bulges shall not protrude from the vertical plane of a bale wall more than 2 inches (51 mm) over 8 feet (2438 mm).

3. The vertical faces of adjacent bales shall not be offset more than 1/2 inch (13 mm)

~~N105.8.2~~ N106.9 Plaster and membranes. Structural strawbale walls shall not have a membrane between straw and plaster, or shall have attachment through the bale wall from one plaster skin to the other in accordance with an *approved* engineered design.

~~N105.9~~ N106.10 Transfer of loads to and from plaster skins. Where plastered strawbale walls are used to support superimposed vertical loads, such loads shall be transferred to the plaster skins by continuous direct bearing or by an *approved* engineered design. Where plastered strawbale walls are used to resist in-plane lateral loads, such loads shall be transferred ~~via~~ to the reinforcing mesh from the structural member or assembly above and to the sill plate in accordance with Table ~~N105.15~~ N106.15, or by an *approved* engineered design.

~~N105.10~~ N106.11 Support of plaster skins. Plaster skins for structural strawbale walls shall be continuously supported along their bottom edge ~~to facilitate the transfer of loads to the foundation system~~. Acceptable supports include~~, but are not limited to~~: a concrete or masonry stem wall, a concrete slab on grade, a wood-framed floor adequately blocked~~,~~ with an *approved* engineered design, or a steel angle adequately anchored~~,~~ with an *approved* engineered design. An ~~conventional metal or plastic~~ unsupported weep screed is not an acceptable support.

N106.12 Resistance to uplift loads. Where plastered strawbale walls are used to resist vertical uplift loads, such loads shall be transferred to the plaster skins by an *approved* engineered design. In lieu of an *approved* engineered design, plaster mesh in skins complying with Table N106.15, with staples at 2 inches (51 mm) on center, shall be considered capable of resisting uplift loads not associated with in-plane shear resistance, of 200 plf (2.918 kN/m) per plaster skin.

~~**N105.11 Unrestrained wall height.** Strawbale walls shall not exceed the ratios of stacked bale height to bale thickness between restraints, as stated in Section 2505.12, except where an *approved* engineered design demonstrates the wall will resist buckling from superimposed vertical loads and out-of-plane design loads.~~

~~**N105.12 Resistance to out-of-plane lateral loads.** Structural and non-structural strawbale walls shall be considered capable of resisting out-of-plane loads prescribed in the *International Building Code* with the following limitations and requirements, except where an *approved* engineered design is provided:~~

~~1. Walls with unreinforced plasters or a non-plaster finish, and without pins in accordance with N105.12.4, or other *approved* means of out-of-plane bracing, shall not exceed a 5:1 ratio of stacked bale height to bale thickness.~~

~~2. Clay plaster walls with reinforced plasters, or pins in accordance with N105.12 Item 4, or other *approved* means of out-of-plane bracing, shall not exceed the ratio indicated in Equation 24-1. Plaster reinforcement shall be any type described in Table N105.15 with staples spaced not more than 6 inches (152 mm) on center.~~

~~$H^2/T = 65$~~ **~~(Equation N-1)~~**

~~*Where:*~~

~~H = stacked bale height~~
~~T= bale thickness~~
~~H and T are measured in feet. ($H^2/T = 19{,}800$ when H and T are measured in mm)~~

~~3. Cement, cement-lime, lime, or soil cement plaster walls with reinforced plasters, or pins in accordance with N105.12 Item 4, or other *approved* means of out-of-plane bracing, shall not exceed the ratio indicated in Equation 24-2. Plaster reinforcement shall be any type described in Table N105.15 with staples spaced not more than 6 inches (152 mm) on center.~~

~~$H^2/T = 80$~~ **~~(Equation N-2)~~**

~~*Where:*~~

~~H = stacked bale height~~
~~T= bale thickness~~
~~H and T are measured in feet. ($H^2/T = 24{,}400$ when H and T are measured in mm)~~

~~4. Pins shall be in accordance with an *approved* engineered design or shall comply with the following:~~

- ~~4.1 Pins shall be 3/8 inch (10 mm) diameter steel, 3/4 inch diameter (19 mm) wood, or 1/2 inch diameter (13 mm) bamboo. Pins shall be external or internal.~~
- ~~4.2 External pins shall be installed on both sides of the wall spaced not more than 24 inches (610 mm) on center.~~
- ~~4.3 External pins shall have full lateral bearing on the sill plate and the roof- or floor-bearing member, and shall be tightly tied through the wall to an opposing pin with ties spaced not more than 30 inches (762 mm) apart and not more than 15 inches (381 mm) from each end.~~
- ~~4.4 Internal pins shall be installed vertically not more than 24 inches (610 mm) on center in the center third of the bales, and shall extend from top course to bottom course.~~
- ~~4.5 The bottom course shall be similarly connected to its support and the top course shall be similarly connected to the roof- or floor-bearing member above with pins or other *approved* means.~~
- ~~4.6 Internal pins shall be continuous or shall overlap through not less than one bale course.~~

~~N105.14~~ N106.13 Load-bearing strawbale walls. Load-bearing strawbale walls shall be in accordance with Table ~~N105.14~~ N106.13 as part of an *approved* engineered design to support superimposed vertical loads. Concentrated loads shall be distributed by a structural element capable of distributing the loads to the bearing wall within the uniform load limits in N106.13. The allowable bearing capacity values in Table N106.13 are in accordance with *allowable stress design*.

N106.13.1 Sill plates and sill fastening. Sill plates shall support and be flush with each face of the straw bales above and shall be preservative-treated where required by the *International Building Code*. For walls supporting superimposed vertical loads only sill plates and fastening shall be in accordance with Section 2308.3. See Table N106.15 for sill plate requirements for shear walls.

~~N105.13~~ N106.14 Design coefficients and factors for seismic design. The values ~~given~~ in Table ~~N105.13~~ N106.14 shall apply to seismic design using strawbale shear walls detailed in accordance with Table ~~N105.15~~ N106.15.

~~N105.15~~ N106.15 Strawbale shear walls. Strawbale shear walls shall be in accordance with Table ~~N105.13~~ N106.15 as part of an *approved* engineered design to resist in-plane lateral loads. The allowable shear values in Table N106.15 are in accordance with *allowable stress design*. Other *approved* in-plane lateral load resisting systems shall be permitted ~~to be used~~ for use in combination with strawbale shear walls with apportionment of design loads as prescribed in the *International Building Code*.

~~N105.16 Connection of light-frame walls to strawbale walls.~~ ~~Light-frame walls perpendicular to, or at an angle to a straw bale wall assembly, shall be fastened to the bottom and top wood members of the strawbale wall in accordance with requirements for wood or cold-formed steel light-frame walls in the *International Building Code*, or the abutting stud shall be connected to alternating straw bale courses with a 1/2 inch (13mm) diameter steel, 3/4" diameter (19 mm) wood, or 5/8" diameter (16 mm) bamboo dowel, with minimum 8 inch (203 mm) penetration.~~

TABLE ~~N105.14~~ N106.13
ALLOWABLE ~~GRAVITY~~ SUPERIMPOSED VERTICAL LOADS (LBS./FOOT) FOR PLASTERED STRAWBALE WALLS

WALL DESIGNATION	PLASTER[a] (both sides) ~~Thickness each side~~		~~SILL PLATES[b,c]~~	~~ANCHOR[c] BOLTS (or other sill fastening)~~	MESH[~~d~~ b]	STAPLES [~~e,f,g~~ c,d,e]	ALLOWABLE BEARING CAPACITY[~~h~~f] (plf)
	TYPE	THICKNESS (each side)					
A	Clay[~~i~~ g] ~~1-1/2"~~	1-1/2 in.	~~c~~	~~c~~	None required[~~i~~ g]	None required[~~i~~ g]	400
B	Soil-cement[~~k~~i] ~~1"~~	1 in.	~~c~~	~~c~~	~~d~~ b	~~e,f,g~~ c,d,e	800
C	~~Lime[j]~~ ~~7/8"~~	7/8 in.	~~c~~	~~c~~	~~d~~ b	~~e,f,g~~ c,d,e	500
D	Cement-lime[~~k~~ i] ~~7/8"~~	7/8 in.	~~c~~	~~c~~	~~d~~ b	~~e,f,g~~ c,d,e	800
E	Cement ~~7/8"~~	7/8 in.	~~c~~	~~c~~	~~d~~ b	~~e,f,g~~ c,d,e	800

For SI: 1 inch=25.4mm, 1 pound per foot = 14.5939 N/m.

a. Plasters shall conform with ~~Sections N106.9~~ N104.7 through ~~N106.12~~ N104.12 for makeup and thickness, with ~~Section N105.8.1~~N106.8 for straightness, and with ~~Section N105.10~~ N106.11 for support of plaster skins.

~~b. Sill plates shall support and be flush with each face of the bale wall and shall be preservative-treated where required by the *International Building Code*..~~

~~c. For walls supporting gravity loads only or for non-structural walls, sill plates and fastening shall be in accordance with the requirements for wood framed walls in the *International Building Code*. See Table N105.15 for requirements for shear walls.~~

b. Any metal mesh allowed by this section shall be installed throughout the plaster with minimum 4-inch laps and fastened in accordance with footnote e.

c. Staples shall be at maximum spacing of 2-inches on center, to roof or floor bearing assembly, or as shown in an *approved* design in accordance with Section ~~N105.9~~ N106.10, and at a maximum spacing of 4-inches on center to sill plates.

d. Staples shall be ~~gun~~ pneumatically driven staples, stainless steel or electro-galvanized, 16 gauge with 1 ¼-inch legs, 7/16-inch crown; or manually driven staples, galvanized 15 gauge with 7/8-inch legs, 3/16-inch inner spread and rounded shoulder. Other staples shall be permitted to be used as designed by a *registered design professional*. Staples into preservative-treated wood shall be stainless steel.

e. Staples shall be firmly driven diagonally across mesh intersections at the spacing indicated.

f. For walls with a different plaster on each side, the lower value shall be used.

g. Except as necessary to transfer roof or floor loads to the plaster skins in accordance with Section ~~N105.9~~ N106.10.

h. The building official is authorized to require a cube compression test to demonstrate a minimum 100 psi compressive strength.

i. The building official is authorized to require a cube compression test to demonstrate a minimum 1000 psi compressive strength.

j. Lime plaster shall use hydraulic or natural hydraulic lime. The building official is authorized to require a cube compression test to demonstrate a minimum 600 psi compressive strength.

k. The building official is authorized to require a cube compression test to demonstrate a minimum 1400 psi compressive strength.

TABLE ~~N105.13~~ N106.14
DESIGN COEFFICIENTS AND FACTORS FOR SEISMIC-FORCE-RESISTING SYSTEMS

Seismic-Force-Resisting System		Response Modification Coefficient, R[a]	System Overstrength Factor, Omega[b]	Deflection Amplification Factor, C	Structural System Limitations and Building Height (ft) Limits				
					Seismic Design Category				
					B	C	D	E	F
A. Bearing Wall Systems									
Strawbale shear walls		3.5	3	3	25	25	15	15	~~15~~NP
B. Building Frame Systems									
Strawbale shear walls		4	3	3.5	35	35	25	~~25~~ 15	~~25~~NP

a. R reduces forces to a strength level, not an allowable stress level

b. The tabulated value of the overstrength factor is permitted to be reduced by subtracting 0.5 for structures with flexible diaphragms, but shall not be taken as less than 2.0 for any structure.

TABLE ~~N105.15~~ N106.15
ALLOWABLE SHEAR (POUNDS PER FOOT) FOR PLASTERED STRAWBALE WALLS[a]

WALL DESIGNATION	PLASTER[b]		SILL PLATES[d]	ANCHOR[d] BOLTS (on center)	MESH[e]	STAPLE SPACING[f, g, h] (on center)	ALLOWABLE SHEAR[i, j, k] (plf)
	TYPE	THICK-NESS (each side)					
A1	Clay[m]	1.5-in.	2 x 4	~~2 ft. 8~~ 32 in.	None	None	60
A2	Clay[m]	1.5-in.	2 x 4	~~2 ft. 8~~ 32 in.	2 in. by 2 in. high-density polypropylene	2-inches	140
A3	Clay[m]	1.5-in,	2 x 4	~~2 ft. 8~~ 32 in.	2”x2”x14ga[l]	4-inches	180
B	Soil-cement[o]	1-in.	4 x 4	~~2 ft. 0~~ 24 in.	2 in. by 2 in. by 14ga[l]	2-inches	520
C1	Lime[n]	7/8-in.	2 x 4	~~2 ft. 8~~ 32 in.	17 ga. woven wire	3-inches	330
C2	Lime[n]	7/8-in.	4 x 4	~~2 ft. 0~~ 24 in.	2 in. by 2 in. by 14ga[l]	2-inches	450
D1	Cement-lime[o]	7/8-in.	4 x 4	~~2 ft. 8~~ 32 in.	17 ga. woven wire	2-inches	380
D2	Cement-lime[o]	7/8-in.	4 x 4	~~2 ft. 0~~ 24 in.	2 in. by 2 in. by 14ga[l]	2-inches	520
E1	Cement[p]	7/8-in.	4 x 4	~~2 ft. 8~~ 32 in.	2 in. by 2 in. by 14 ga[l]	2-inches	540
E2	Cement[p]	1.5-in.	4 x 4	~~2 ft. 0~~ 24 in.	2 in. by 2 in. by 14ga[l]	2-inches	680

SI: 1 inch=25.4 mm, 1 pound per foot = 14.5939 N/m

a. Bales shall be not less than 15 inches thick.

b. Plasters shall conform with ~~Sections N106.9~~ N104.7 through ~~N106.12~~ N104.12 for makeup and thickness, with ~~Section N105.8.1~~N106.8 for straightness, and with ~~Section N105.10~~ N106.11 for support of plaster skins.

c. Sill plates shall be Douglas fir-larch or southern pine and shall be preservative-treated where required by the *International Building Code*. Multiply allowable shear value by .82 for other species with specific gravity of .42 or greater, or by .65 for all other species.

d. Anchor bolts shall be 5/8-inch diameter with 2-inch by 2-inch by 3/16-inch washers, with not less than 7-inch embedment in concrete or masonry foundation. Anchor bolts or other fasteners into framed floors shall be engineered.

e. Mesh shall run continuous vertically from sill plate to top plate, roof or floor beam, or roof or floor bearing assembly, or shall lap not less than ~~12-inches~~ 8 inches. ~~Horizontal laps shall be a not less than 4-inches.~~ Steel mesh shall be galvanized. Galvanized steel mesh shall be separated from preservative-treated wood by grade D paper, 15# roofing felt, or other *approved* barrier.

f. Staples shall be ~~gun~~ pneumatically driven staples, stainless steel or electro-galvanized, 16 gauge with ~~1 ¼-inch legs,~~ 7/16-inch crown~~;~~. Staple legs shall be 1 ¾ inch long except that 1-1/4 inch legs shall be permitted to be used to fasten mesh in clay plaster walls. ~~or~~ manually driven staples, galvanized 15 gauge with 7/8-inch legs, 3/16-inch inner spread and rounded shoulder shall be permitted to be used in clay plaster walls. Other staples shall be permitted to be used as designed by a *registered design professional*. Staples into preservative-treated wood shall be stainless steel.

g. Staples at spacing indicated are to boundary ~~conditions~~members, including sill plates, and top plate, roof or floor beam, or roof or floor bearing assembly,

h. Staples shall be firmly driven diagonally across mesh intersections at spacing indicated.

i. Values shown are for aspect ratios of 1:1 or less. Reduce values shown to 50% for the limit of a 2:1 aspect ratio. Linear interpolation shall be permitted for aspect ratios between 1:1 and 2:1. The full value shown shall be used for aspect ratios greater than 1:1, where an additional layer of mesh is installed at the base of the wall to a height where the remainder of the wall has an aspect ratio of 1:1 or less, and the second layer of mesh is fastened to the sill plate with the required stapling, and the sill bolt spacing is decreased with linear interpolation between1:1 and 2:1.

j. For walls with a plaster Type A on one side and any other plaster type on the other side, a registered design professional shall show transfer of the design lateral load into the stiffer Type B, C, D, or E plaster only, and 50% of the allowable shear value shown for that wall ~~type~~ designation shall be used.

k. These values are permitted to be increased 40 percent for wind design.

l. 16 gauge mesh shall be permitted to be used with a reduction to 0.60 of the allowable shear values shown.

m. The building official is authorized to require a cube compression test demonstrating not less than ~~600~~100 psi compressive strength.

n. Lime plaster shall use a binder consisting of hydraulic or natural hydraulic lime. The building official is authorized to require a cube compression test demonstrating not less than 600 psi compressive strength.

o. The building official is authorized to require a cube compression test demonstrating not less than 1000 psi compressive strength.

p. The building official is authorized to require a cube compression test demonstrating not less than 1400 psi compressive strength.

SECTION N108
THERMAL INSULATION

N108.1 R-value. The unit R-value of a strawbale wall with bales laid flat is R-1.3 per inch of bale thickness.~~, and~~ The unit R- value of a straw bale wall with bales on-edge is R-2 per inch of bale thickness.

Commenter's Reason: Proposal S315-12 to create an Appendix on Strawbale Construction has been modified to respond to concerns raised at the ICC Code Development hearing in Dallas, and raised in review by members of the National Council of Structural Engineers Associations (NCSEA) Code Committee and members of the Structural Engineers Association of California (SEAOC) Code Committee, as well as review by the primary authors of the original proposal.

Substantial changes include:

- Restricting buildings that utilize strawbale walls as load-bearing or lateral-force-resisting systems to 35 ft and two-stories in height above grade.
- Excluding strawbale walls from use as lateral load-resisting systems in Seismic Design Category F and restricting the height of buildings utilizing strawbale walls as lateral load resisting systems to 15 ft. in Seismic Design Category E.
- Introducing prescriptive criteria for resistance to out-of-plane loads.
- Adding a Table regarding out-of-plane resistance and unrestrained wall height.
- Introducing requirements for concentrated superimposed vertical loads.
- Introducing requirements for the use of strawbale walls to resist uplift loads.

In addition, the organization of the proposed Appendix has changed for clarity. A new section entitled "Strawbale Walls - General" has been formed, largely from language in the original proposal, including the original "Moisture Control" section. The section entitled "Finishes" has been relocated and the section entitled "Structural Use" is now entitled "Strawbale Walls – Structural", with non-structural content moved to the "Strawbale Walls - General" section.

Vague or unenforceable language has been modified, and redundant language has been removed.

Also, the Reason Statement from the original proposal stands, including its link to supporting documentation, as follows:

Strawbale construction has proven to be a safe, durable, resource efficient, and fully viable method of construction. However, the International Building Code does not contain a section on strawbale construction, which has been an impediment to this construction system's proper and broader use.

First practiced in Nebraska in the late 1800's, with buildings over 100 years old still in service, strawbale construction was rediscovered in the 1980's in the American southwest. Since then it has been further developed and explored, including considerable testing and research regarding structural performance (under vertical and lateral loads), moisture, fire, and its thermal and acoustic properties.

Currently only Oregon and New Mexico have adopted statewide strawbale building codes. California has legislated strawbale construction guidelines for voluntary adoption by local jurisdictions. In addition, nine U.S. cities or counties have adopted strawbale building codes. Three countries outside the United States – Germany, France, and Belarus - have limited strawbale building codes.

Most of the strawbale building codes that do exist are derived from the first such code, created for and adopted by Tucson / Pima County, Arizona in 1996. Much experience, testing, and research since then have proven these codes to be deficient. They are often either too restrictive, or not restrictive enough, and in some cases don't address important issues at all.

Although strawbale codes are both few and flawed, strawbale buildings are now found in 49 of the 50 United States, and strawbale construction is practiced in over 45 countries throughout the world and in every climate. There are an estimated 600-1000 strawbale buildings in California alone. The strawbale buildings in the U.S. include residences, schools, office buildings, wineries, multi-story buildings, buildings over 10,000 sq.ft in floor area, load-bearing strawbale structures, and structures in areas of high seismic risk (plastered strawbale walls are particularly resistant to earthquakes). The practice of, and the desire to utilize strawbale construction, continues to increase and promises to accelerate as we face increased pressure on our environment and natural resources.

There is great need for a comprehensive strawbale code, with full benefit of the experience and knowledge that has been gained to date about this method of construction. The proposed Strawbale Construction appendix for the IBC was created to fulfill this need. It is based on the collective experience of the design, construction, and testing of strawbale buildings over 20 years by architects, engineers, builders, and academics throughout the U.S., Canada, and other countries throughout the world. The testing, research, and comprehensive understanding of the performance of strawbale buildings are summarized in the book *Design of Straw Bale Buildings* (B.King, et al, 2006, Green Building Press). Testing, research reports, and other supporting documentation are available for viewing and download at: http://www.ecobuildnetwork.org/strawbale-construction-code-supporting-documentation

As lead author of the proposed appendix, and as a licensed architect for 25 years, I have been involved in the design, construction, testing, and research of strawbale buildings since 1995. In 2001 I spearheaded legislation and revisions to the current California Guidelines for Straw-Bale Structures. The proposed Strawbale Construction appendix for the IBC has benefited from numerous peer reviews by experienced, licensed design and building professionals over the course of more than five years. It would serve designers, builders, owners, inhabitants, and building officials alike in the construction and utilization of strawbale buildings.

Supporting Documentation: Selected documents that are available via the above link

Load-Bearing Straw Bale Construction – A summary of worldwide testing and experience, B.King, PE
Testing of Straw Bale Walls with Out-of-Plane Loads – K.Donahue, SE
In-Plane Cyclic Tests of Plastered Straw Bale Wall Assemblies – C.Ash, M.Aschheim, PE, D.Mar, SE
Structural Testing of Plastered Straw Bale Wall Assemblies – K.Lerner, Architect, K.Donahue, SE
Basis for Allowable Shears for Strawbale Walls – M.Aschheim, PE, M.Hammer, Architect
Proposed Shear Values and Seismic Design Factors for Strawbale Walls – M.Aschheim, PE

Shake Table Test Video of Full Scale Straw Bale Building Specimen – D.Donovan, PE
Moisture Properties of Plaster and Stucco for Strawbale Buildings – J.Straube, PE
Monitoring of Hygrothermal Performance of Strawbale Walls – J.Sraube, PE, C.Schumacher
ASTM E119 1-Hour Fire Resistance Test of a Non-Loadbearing Straw Bale Wall with Clay Plaster
ASTM E119 2-Hour Fire Resistance Test of a Non-Loadbearing Straw Bale Wall with Cement Plaster
ASTM E119 Fire Tests - Video
Thermal Performance of Straw Bale Wall Systems (incl. Oak Ridge Lab test results) – N.Stone
Support Letters from Licensed Practitioners: Letters from 2 Structural Engineers, 4 Civil Engineers, 1 Professor of Civil Engineering, 7 Architects

Public Comment 2:

Hope Medina, Town of Castle Rock, CO, representing self and Kirk Nagle City of Arvada, CO, representing self, requests Approval as Submitted.

Commenter's Reason: The committee's concern regarding adoption into the appendix is not relevant because all jurisdictions can exclude any or all portions of the appendix as necessary. Code adaption by local jurisdictions or by states could exclude straw bale construction if they felt that it was unacceptable. There are existing structures built from 1896 through 1907 in Nebraska, with nine structures still in use. There are more than 1,000 straw bales homes in California which have withstood earthquakes, demonstrating their structural strength to withstand lateral and longitudinal forces. These buildings are highly energy efficient, sustainable, and use local materials for construction. These structures are homes, churches, schools, commercial buildings, gyms, stores, and many other viable sustainable structures used by occupants for their safety and energy efficiency. Overturning the committee is the right thing to do.

Wood buildings if purposed as a new material for construction, based on current engineering standards would not be allowed into the body of the code. Wood buildings have been around for hundreds of years and so has straw bale construction. Straw bale is a viable building material that has proved itself for hundreds of years.

S315, Part I-12
Final Action: AS AM AMPC____ D

S315-12, Part II
Appendix N (New)

Proposed Change as Submitted

Proponent: Martin Hammer, Architect, representing California Straw Building Association, Colorado Straw Bale Association, Straw Bale Construction Association – New Mexico, Ontario Bale Building Coalition, Development Center for Appropriate Technology, Environmental Building Network (mfhammer@pacbell.net)

THIS IS A TWO PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IBC FIRE SAFETY COMMITTEE. SEE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES

PART II – IBC FIRE SAFETY

SECTION N107
FIRE RESISTANCE

N107.1 Fire-resistance rating. Fire-resistance ratings for strawbale walls shall be established in accordance with Section N107.1.1 or N107.1.2, or shall be determined in accordance with Section 703.2 or 703.3 of the *International Building Code*.

N107.1.1 1-hour rated clay plastered wall. 1-hour fire-resistance-rated nonload-bearing clay plastered strawbale walls shall comply with all of the following:

1. Bales shall be laid flat or on-edge in a running bond. Gaps shall be fire-stopped with straw-clay,
2. Bales shall maintain thickness of not less than 18 inches (457 mm),
3. Clay plaster on each side of the wall shall be not less than 1 inch (25 mm) thick and shall be comprised of a mixture of 3 parts clay, 2 parts chopped straw, and 6 parts sand, or an alternative *approved* clay plaster.
4. Plaster application shall be in accordance with Section N106.9 for the number and thickness of coats.

N107.1.2 2-hour rated cement plastered wall. 2-hour fire-resistance-rated nonload-bearing cement plastered strawbale walls shall comply with all of the following:

1. Bales shall be laid flat or on-edge in a running bond. Gaps shall be fire-stopped with straw-clay.
2. Bales shall maintain a thickness of not less than 14 inches (356 mm).
3. 1 1/2 inch (38 mm) by 17 gauge galvanized woven wire mesh shall be attached to wood members with 1 1/2 inch (38 mm) staples at 6 inches (406 mm) on center. 9 gauge U-pins with minimum 8 inch (203 mm) legs shall be installed in the field at 18 inches (457 mm) on center.
4. Cement plaster on each side of the wall shall be not less than 1 inch (25 mm) thick.
5. Plaster application shall be in accordance with Section N106.14 for the number and thickness of coats.

N107.2 Openings in rated walls. Openings and penetrations in bale walls required to have a fire-resistance rating shall satisfy the same requirements for openings and penetrations as prescribed in the *International Building Code*.

N107.3 Clearance to fireplaces and chimneys. Strawbale surfaces adjacent to fireplaces or chimneys shall have a minimum 3/8 inch (10 mm) thick plaster coat of any type permitted by this section, and shall maintain the specified clearances to the plaster finish as required to combustibles in *International Building*

Code Chapter 21, Sections 2111, 2112, and 2113, or as required by manufacturer's installation instructions, whichever is more restrictive.

N107.4 Type of construction. Buildings or portions thereof utilizing strawbale walls in accordance with this appendix shall be classified as Type V-B construction. Strawbale walls constructed in compliance with Section N107.1.1 or N107.1.2 shall be permitted wherever combustible walls of the same fire-resistance are allowed by Chapter 6 of the *International Buildlng Code*. Strawbale walls with any finish allowed by this appendix shall be permitted wherever non-rated combustible walls are allowed by the *International Building Code*.

Reason: Strawbale construction has proven to be a safe, durable, resource efficient, and fully viable method of construction. However, the International Building Code does not contain a section on strawbale construction, which has been an impediment to this construction system's proper and broader use.

First practiced in Nebraska in the late 1800's, with buildings over 100 years old still in service, strawbale construction was rediscovered in the 1980's in the American southwest. Since then it has been further developed and explored, including considerable testing and research regarding structural performance (under vertical and lateral loads), moisture, fire, and its thermal and acoustic properties.

Currently only Oregon and New Mexico have adopted statewide strawbale building codes. California has legislated strawbale construction guidelines that are voluntarily adopted at the local level. In addition, nine U.S. cities or counties have adopted strawbale building codes. Three countries outside the United States – Germany, France, and Belarus - have limited strawbale building codes.

Most of the strawbale building codes that do exist are derived from the first such code, created for and adopted by Tucson / Pima County, Arizona in 1996. Much experience, testing, and research since then have proven these codes to be deficient. They are often either too restrictive, or not restrictive enough, and in some cases don't address important issues at all.

Although strawbale codes are both few and flawed, strawbale buildings are now found in 49 of the 50 United States, and strawbale construction is practiced in over 45 countries throughout the world and in every climate. There are an estimated 600-1000 strawbale buildings in California alone. The strawbale buildings in the U.S. include residences, schools, office buildings, wineries, multi-story buildings, buildings over 10,000 sq.ft in floor area, load-bearing strawbale structures, and structures in areas of high seismic risk (plastered strawbale walls are particularly resistant to earthquakes). The practice of, and the desire to utilize strawbale construction, continues to increase and promises to accelerate as we face increased pressure on our environment and natural resources.

There is great need for a comprehensive strawbale code, with full benefit of the experience and knowledge that has been gained to date about this method of construction. The following proposed Strawbale Construction appendix for the IBC was created to fulfill this need. It is based on the collective experience of the design, construction, and testing of strawbale buildings over 20 years by architects, engineers, builders, and academics throughout the U.S., Canada, and other countries throughout the world. The testing, research, and comprehensive understanding of the performance of strawbale buildings are summarized in the book *Design of Straw Bale Buildings* (B.King, et al, 2006, Green Building Press). Testing, research reports, and other supporting documentation are available for viewing and download at: http://www.ecobuildnetwork.org/strawbale-construction-code-supporting-documentation

As lead author of the proposed appendix, and as a licensed architect for 25 years, I have been involved in the design, construction, testing, and research of strawbale buildings since 1995. In 2001 I spearheaded legislation and revisions to the current California Guidelines for Straw-Bale Structures. The proposed Strawbale Construction appendix for the IBC has benefited from numerous peer reviews by experienced, licensed design and building professionals over the course of more than five years. It would serve designers, builders, owners, inhabitants, and building officials alike in the construction and utilization of strawbale buildings.

Supporting Documentation: List of selected documents available via the above link

Load-Bearing Straw Bale Construction – A summary of worldwide testing and experience, B.King, PE
Testing of Straw Bale Walls with Out-of-Plane Loads – K.Donahue, SE
In-Plane Cyclic Tests of Plastered Straw Bale Wall Assemblies – C.Ash, M.Aschheim, PE, D.Mar, SE
Structural Testing of Plastered Straw Bale Wall Assemblies – K.Lerner, Architect, K.Donahue, SE
Seismic Design Factors and Allowable Shears for Strawbale Wall Assemblies – S. Jalali, M. Aschheim, PE
Shake Table Test Video of Full Scale Straw Bale Building Specimen – D.Donovan, PE
Moisture Properties of Plaster and Stucco for Strawbale Buildings – J.Straube, PE
Monitoring of Hygrothermal Performance of Strawbale Walls – J.Sraube, PE, C.Schumacher
ASTM E119 1-Hour Fire Resistance Test of a Non-Loadbearing Straw Bale Wall with Clay Plaster
ASTM E119 2-Hour Fire Resistance Test of a Non-Loadbearing Straw Bale Wall with Cement Plaster
ASTM E119 Fire Tests - Video
Thermal Performance of Straw Bale Wall Systems (incl. Oak Ridge Lab test results) – N.Stone
Support Letters from Licensed Practitioners: Letters from 2 Structural Engineers, 4 Civil Engineers, 1 Professor of Civil Engineering, 7 Architects

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

APPENDIX N (NEW)-S-HAMMER-AB2-15-12.doc

Public Hearing Results

PART II – IBC FIRE SAFETY
Committee Action: Disapproved

Committee Reason: The committee felt that the proposal was insufficient for the following reasons: The fire resistance rated assemblies are not complete and do not specify the installation of materials; load-bearing assemblies should be included; evidence of tested opening protective assemblies should be provided; and the mixture ratio of clay plaster as it relates to Section 2407.1.1 should be provided.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment 1:

Martin Hammer, representing California Straw Building Association, Colorado Straw Bale Association, Straw Bale Construction Association – New Mexico, Ontario Straw Bale Building Coalition, Development Center for Appropriate Technology, Ecological Building Network, requests Approval as Submitted.

Commenter's Reason: Proposal S315-12 to create an Appendix on Strawbale Construction has been modified to respond to concerns raised at the ICC Code Development hearing in Dallas, and raised in review by members of the National Council of Structural Engineers Associations (NCSEA) Code Committee and members of the Structural Engineers Association of California (SEAOC) Code Committee, as well as review by the primary authors of the original proposal.

Substantial changes include:

- Restricting buildings that utilize strawbale walls as load-bearing or lateral-force-resisting systems to 35 ft and two-stories in height above grade.
- Excluding strawbale walls from use as lateral load-resisting systems in Seismic Design Category F and restricting the height of buildings utilizing strawbale walls as lateral load resisting systems to 15 ft. in Seismic Design Category E.
- Introducing prescriptive criteria for resistance to out-of-plane loads.
- Adding a Table regarding out-of-plane resistance and unrestrained wall height.
- Introducing requirements for concentrated superimposed vertical loads.
- Introducing requirements for the use of strawbale walls to resist uplift loads.

In addition, the organization of the proposed Appendix has changed for clarity. A new section entitled "Strawbale Walls - General" has been formed, largely from language in the original proposal, including the original "Moisture Control" section. The section entitled "Finishes" has been relocated and the section entitled "Structural Use" is now entitled "Strawbale Walls – Structural", with non-structural content moved to the "Strawbale Walls - General" section.

Vague or unenforceable language has been modified, and redundant language has been removed.

Also, the Reason Statement from the original proposal stands, including its link to supporting documentation, as follows:

Strawbale construction has proven to be a safe, durable, resource efficient, and fully viable method of construction. However, the International Building Code does not contain a section on strawbale construction, which has been an impediment to this construction system's proper and broader use.

First practiced in Nebraska in the late 1800's, with buildings over 100 years old still in service, strawbale construction was rediscovered in the 1980's in the American southwest. Since then it has been further developed and explored, including considerable testing and research regarding structural performance (under vertical and lateral loads), moisture, fire, and its thermal and acoustic properties.

Currently only Oregon and New Mexico have adopted statewide strawbale building codes. California has legislated strawbale construction guidelines for voluntary adoption by local jurisdictions. In addition, nine U.S. cities or counties have adopted strawbale building codes. Three countries outside the United States – Germany, France, and Belarus - have limited strawbale building codes.

Most of the strawbale building codes that do exist are derived from the first such code, created for and adopted by Tucson / Pima County, Arizona in 1996. Much experience, testing, and research since then have proven these codes to be deficient. They are often either too restrictive, or not restrictive enough, and in some cases don't address important issues at all.

Although strawbale codes are both few and flawed, strawbale buildings are now found in 49 of the 50 United States, and strawbale construction is practiced in over 45 countries throughout the world and in every climate. There are an estimated 600-1000 strawbale buildings in California alone. The strawbale buildings in the U.S. include residences, schools, office buildings, wineries, multi-story buildings, buildings over 10,000 sq.ft in floor area, load-bearing strawbale structures, and structures in areas of high seismic risk (plastered strawbale walls are particularly resistant to earthquakes). The practice of, and the desire to utilize strawbale construction, continues to increase and promises to accelerate as we face increased pressure on our environment and natural resources.

There is great need for a comprehensive strawbale code, with full benefit of the experience and knowledge that has been gained to date about this method of construction. The proposed Strawbale Construction appendix for the IBC was created to fulfill this need. It is based on the collective experience of the design, construction, and testing of strawbale buildings over 20 years by architects, engineers, builders, and academics throughout the U.S., Canada, and other countries throughout the world. The testing, research, and comprehensive understanding of the performance of strawbale buildings are summarized in the book *Design of Straw Bale Buildings* (B.King, et al, 2006, Green Building Press). Testing, research reports, and other supporting documentation are available for viewing and download at: http://www.ecobuildnetwork.org/strawbale-construction-code-supporting-documentation

As lead author of the proposed appendix, and as a licensed architect for 25 years, I have been involved in the design, construction, testing, and research of strawbale buildings since 1995. In 2001 I spearheaded legislation and revisions to the current California Guidelines for Straw-Bale Structures. The proposed Strawbale Construction appendix for the IBC has benefited from numerous peer reviews by experienced, licensed design and building professionals over the course of more than five years. It would serve designers, builders, owners, inhabitants, and building officials alike in the construction and utilization of strawbale buildings.

Supporting Documentation: Selected documents that are available via the above link

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In-Plane Cyclic Tests of Plastered Straw Bale Wall Assemblies – C.Ash, M.Aschheim, PE, D.Mar, SE
Structural Testing of Plastered Straw Bale Wall Assemblies – K.Lerner, Architect, K.Donahue, SE
Basis for Allowable Shears for Strawbale Walls – M.Aschheim, PE, M.Hammer, Architect
Proposed Shear Values and Seismic Design Factors for Strawbale Walls – M.Aschheim, PE
Shake Table Test Video of Full Scale Straw Bale Building Specimen – D.Donovan, PE
Moisture Properties of Plaster and Stucco for Strawbale Buildings – J.Straube, PE
Monitoring of Hygrothermal Performance of Strawbale Walls – J.Sraube, PE, C.Schumacher
ASTM E119 1-Hour Fire Resistance Test of a Non-Loadbearing Straw Bale Wall with Clay Plaster
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ASTM E119 Fire Tests - Video
Thermal Performance of Straw Bale Wall Systems (incl. Oak Ridge Lab test results) – N.Stone
Support Letters from Licensed Practitioners: Letters from 2 Structural Engineers, 4 Civil Engineers, 1 Professor of Civil Engineering, 7 Architects

Public Comment 2:

Hope Medina, Town of Castle Rock, CO, representing self and Kirk Nagle City of Arvada, CO, representing self, requests Approval as Submitted.

Commenter's Reason: The committee's concern regarding adoption into the appendix is not relevant because all jurisdictions can exclude any or all portions of the appendix as necessary. Code adaption by local jurisdictions or by states could exclude straw bale construction if they felt that it was unacceptable. There are existing structures built from 1896 through 1907 in Nebraska, with nine structures still in use. There are more than 1,000 straw bales homes in California which have withstood earthquakes, demonstrating their structural strength to withstand lateral and longitudinal forces. These buildings are highly energy efficient, sustainable, and use local materials for construction. These structures are homes, churches, schools, commercial buildings, gyms, stores, and many other viable sustainable structures used by occupants for their safety and energy efficiency. Overturning the committee is the right thing to do.

Wood buildings if purposed as a new material for construction, based on current engineering standards would not be allowed into the body of the code. Wood buildings have been around for hundreds of years and so has straw bale construction. Straw bale is a viable building material that has proved itself for hundreds of years.

S315, Part II-12
Final Action: AS AM AMPC____ D

S319-12
G102.1

<u>Proposed Change as Submitted</u>

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

Revise as follows:

G102.1 General. This appendix, in conjunction with the *International Building Code*, provides minimum requirements for development located in flood hazard areas, including the subdivision of land; <u>site improvements and</u> installation of utilities; placement and replacement of manufactured homes; <u>placement of recreational vehicles;</u> new construction and repair, reconstruction, rehabilitation or additions to new construction; substantial improvement of existing buildings and structures, including restoration after damage, <u>installation of tanks;</u> temporary structures, and temporary or permanent storage, utility and miscellaneous Group U buildings and structures, and certain building work exempt from permit under Section 105.2 <u>and other buildings and development activities</u>.

Reason: The purpose of this section is to identify the development activities for which minimum requirements are listed in Appendix G. The proposed changes are consistent with the subsections in Appendix G (including some proposed new subsections).

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

G102.1-S-INGARGIOLA-WILSON-QUINN.doc

<u>Public Hearing Results</u>

Committee Action: **Approved as Submitted**

Committee Reason: This code change aligns the IBC appendix with FEMA requirements and ASE 24. It also clarifies the appendix by coordinating the wording of Section G102.1 with the remainder of the appendix.

Assembly Action: **None**

<u>Individual Consideration Agenda</u>

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

Public Comment:

Jonathan Siu, City of Seattle Department of Planning & Development requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

~~**G102.1 General.** This appendix, in conjunction with the *International Building Code*, provides minimum requirements for development located in flood hazard areas, including the subdivision of land; site improvements and installation of utilities; placement and replacement of manufactured homes; placement of recreational vehicles; new construction and repair, reconstruction, rehabilitation or additions to new construction; substantial improvement of existing buildings and structures, including restoration after damage, installation of tanks; temporary structures, and temporary or permanent storage, utility and miscellaneous Group U buildings and structures, and certain building work exempt from permit under Section 105.2 and other buildings and development activities.~~

G102.1 General. This appendix, in conjunction with the *International Building Code*, provides minimum requirements for development located in flood hazard areas, including:

1. The subdivision of land;
2. Site improvements and installation of utilities;
3. Placement and replacement of manufactured homes;
4. Placement of recreational vehicles;
5. New construction and repair, reconstruction, rehabilitation or additions to new construction;
6. Substantial improvement of existing buildings and structures, including restoration after damage;
7. Installation of tanks;
8. Temporary structures;
9. Temporary or permanent storage, utility and miscellaneous Group U buildings and structures; and
10. Certain building work exempt from permit under Section 105.2 and other buildings and development activities.

Commenter's Reason: The purpose of this public comment is to reformat the list of activities within the scope of this appendix in to a bullet list. This makes the section more readable and easier to understand. No technical changes are made—the text in the bullet list is taken verbatim from the existing text in the code, and includes the additional items approved by the Structural Committee in Dallas.

S319-12

Final Action: AS AM AMPC____ D

S323-12

G103.8 (New), G104.2

Proposed Change as Submitted

Proponent: John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency (john.ingargiola@dhs.gov, gregory.p.wilson@dhs.gov) and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency (rcquinn@earthlink.net)

Revise as follows:

G103.8 Substantial improvement and substantial damage determinations. For permit applications to improve or repair buildings and structures, including additions, repairs, rehabilitations, renovations, alterations, relocations, reconstructions, or other work, the building official, shall:

1. Estimate the market value, or require the applicant to obtain a professional appraisal of the market value, of the building or structure before the proposed work is performed; the market value of the building or structure shall be the market value before the damage occurred or before any improvement is made;
2. Compare the cost to perform the improvement, the cost to repair the damaged building to its pre-damaged condition, or the combined costs of improvements and repairs, if applicable, to the market value of the building or structure;
3. Determine and document whether the proposed work constitutes substantial improvement or repair of substantial damage; and
4. If the determination finds that the proposed work constitutes substantial improvement or repair of substantial damage, notify the applicant of the results of the determination and whether compliance with the requirements of the building code is required.

~~G103.8~~ G103.9 Records. The *building official* shall maintain a permanent record of all *permits* issued in *flood hazard areas*, including copies of inspection reports and certifications required in Section 1612.

G104.2 Application for permit. The applicant shall file an application in writing on a form furnished by the *building official*. Such application shall:

1. Identify and describe the development to be covered by the permit.
2. Describe the land on which the proposed development is to be conducted by legal description, street address or similar description that will readily identify and definitely locate the site.
3. Include a site plan showing the delineation of flood hazard areas, floodway boundaries, flood zones, design flood elevations, ground elevations, proposed fill and excavation and drainage patterns and facilities.
4. Indicate the use and occupancy for which the proposed development is intended.
5. Be accompanied by construction documents, grading and filling plans and other information deemed appropriate by the building official.
6. State the valuation of the proposed work.
7. Include a market value appraisal of the building (excluding land), for applications for work on existing buildings, unless otherwise advised by the building official.

~~7~~8. Be signed by the applicant or the applicant's authorized agent.

Reason: Communities that participate in the NFIP agree to regulate all development in flood hazard areas. FEMA states that the flood provisions in the I-Codes are consistent with the NFIP requirements for the design and construction of buildings. To fully meet the requirements of the NFIP local jurisdictions must adopt a local ordinance or Appendix G in order to have the necessary administrative provisions and requirements for development other than buildings.

Section 105.3 of the code requires the applicant to describe the work to be covered by the permit and to state the valuation of the proposed work. The building code defines and uses the terms "substantial improvement" and "substantial damage." This proposal clarifies how the building official is to use the information to determine whether proposed work meets the definitions.

FEMA recently published FEMA P-758, *Substantial Improvement/Substantial Damage Desk Reference*, that includes guidance for local officials on estimating market value as well as estimating costs. This proposal states that the applicant shall submit a market value appraisal unless otherwise advised; FEMA guidance now states that local officials may use "adjusted assessed value" or "actual cash value" (replacement minus depreciation).

Cost Impact: The code change proposal will not increase the cost of construction. Determining whether work proposed on an existing building is substantial improvement or repair of substantial damage is implicit in the definitions of those terms. This proposal does not change the fact that determining whether proposed work meets those definitions has to be done. It simply clarifies how it is to be done.

G103.8 #2-S-INGARGIOLA-WILSON-QUINN.doc

Public Hearing Results

Committee Action: Disapproved

Committee Reason: Substantial improvement determinations are already required, but the provision proposed for Appendix G is a very prescriptive requirement that seems to place more of a burden on the building official. It is possible that the requirement does not belong in the building code and may be more appropriate for zoning regulation.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

John Ingargiola and Gregory Wilson, representing Department of Homeland Security, Federal Emergency Management Agency and Rebecca Quinn, RCQuinn Consulting, Inc., representing Federal Emergency Management Agency, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

ADJUSTED ASSESSED VALUE. The value of a building determined for property tax or assessment purposes, adjusted by a factor to reasonably reflect current market value by accounting for appraisal cycle, land value and assessment level.

ACTUAL CASH VALUE. The cost to replace a building on the same parcel with a new building of like-kind and quality, minimum depreciation due to age, use, and neglect.

G103.8 Substantial improvement and substantial damage determinations. For permit applications to improve or repair buildings and structures, including additions, repairs, rehabilitations, renovations, alterations, relocations, reconstructions, or other work, the building official, shall:

1. Estimate the market value using adjusted assessed value or actual cash value, or require the applicant to obtain a professional appraisal of the market value, of the building or structure before the proposed work is performed; the market value of the building or structure shall be the market value before the damage occurred or before any improvement is made;
2. Compare the cost to perform the improvement, the cost to repair the damaged building to its pre-damaged condition, or the combined costs of improvements and repairs, if applicable, to the market value of the building or structure;
3. Determine and document whether the proposed work constitutes substantial improvement or repair of substantial damage; and
4. If the determination finds that the proposed work constitutes substantial improvement or repair of substantial damage, notify the applicant of the results of the determination and whether compliance with the requirements of the building code is required.

G103.9 Records. The *building official* shall maintain a permanent record of all *permits* issued in *flood hazard areas*, including copies of inspection reports and certifications required in Section 1612.

G104.2 Application for permit. The applicant shall file an application in writing on a form furnished by the *building official*. Such application shall:

1. Identify and describe the development to be covered by the permit.

2. Describe the land on which the proposed development is to be conducted by legal description, street address or similar description that will readily identify and definitely locate the site.
3. Include a site plan showing the delineation of flood hazard areas, floodway boundaries, flood zones, design flood elevations, ground elevations, proposed fill and excavation and drainage patterns and facilities.
4. Indicate the use and occupancy for which the proposed development is intended.
5. Be accompanied by construction documents, grading and filling plans and other information deemed appropriate by the building official.
6. State the valuation of the proposed work.
7. Include a market value appraisal of the building (excluding land), for applications for work on existing buildings, unless ~~otherwise advised by~~ the building official <u>has estimated market value pursuant to G103.8, :item 1.</u>
8. Be signed by the applicant or the applicant's authorized agent.

Commenter's Reason: The IBC and IEBC define "substantial improvement" and "substantial damage" without indicating the source of the market value. Both definitions require the cost of proposed work to be compared to the market value of the building. Section 105.3 requires the applicant to describe the work to be covered by the permit and to state the valuation of the proposed work. This proposal, modified to reflect committee discussion and comments made during the hearing, clarifies how the building official is to get the market value in order to be able to determine whether proposed work meets the definitions. As modified, this proposal permits use of adjusted assessed value or actual cash value as estimates of market value, while reserving the option to require the applicant to obtain a professional appraisal. Most communities accept professional appraisals if the applicant elects to provide one.

The committee commented on the prescriptive nature of the proposal and expressed concern about placing burden on the building official. This proposal adds no additional responsibility or burden. It is already the building official's responsibility to determine if work on existing buildings in flood hazard areas meets the definition of substantial improvement or substantial damage. This proposal is consistent with FEMA guidance FEMA P-758, *Substantial Improvement/Substantial Damage Desk Reference*, regarding estimating market value.

S323-12

Final Action: AS AM AMPC____ D

S335-12
L101.1

Proposed Change as Submitted

Proponent: James Bela, Oregon Earthquake Awareness, representing self

Revise as follows:

L101.1 General. Every ~~structure~~ building located ~~where the 1-second spectral response acceleration, S1, in accordance with Section 1613.3 is greater than 0.40~~ within 15 miles distance of an active fault with a maximum potential earthquake M 6 or above, or lies within 25 miles distance of an active fault wit a maximum potential earthquake M 7 or avove; that either 1) exceeds six stories in height with an aggregate floor area of 60,000 square feet (5574 m2) or more, or 2) exceeds ten stories in height regardless of floor area, shall be equipped with not less than three approved recording accelerographs. The accelerographs shall be interconnected for common start and common timing.

Reason: The 1-second spectral response acceleration contours are interesting, but their locations are yo-yoing around with each new addition of the maps; such that they are not reliable over time. See discussion per Code Change: **IBC-12.13 FIGURE 1613.3.3.1 (1)(2)(3)(4)(5)(6)_.**

An earthquake will occur on a fault, and it is the proximity of a building to an earthquake source that determines its actual experience to ground shaking in a real earthquake. This additional charging language fills this hole in locations, particularly in the western U.S. where there are active faults; but the sum total (of probabilities of exceedence) of all contributing faults is not enough to give 1-second contours of 0.40g.

The term building is as used in the city of Los Angeles strong motion accelerograph language. We have building officials, building codes, building permits, building maintenance, Building Owners and Managers Associations . . . so everyone is pretty clear what a "building" actually is. Maybe, for example, an airplane hangar is more of a structure, than it is a building?

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

L101.1-S-BELA.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The committee prefers the current trigger in Appendix L which is simple and based on the maps that are in the code.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

James Bela, Oregon Earthquake Awareness, representing self, requests Approval as Submitted.

Commenter's Reason: The Committee Action for Disapproval: (a) incorrectly substitutes the committee's so-called "preference" for retaining "the current trigger in Appendix L" without stating specific and defensible objections against the Proposed Change S335-12; and (b) misunderstands that the current trigger in Appendix L (based on the USGS seismic hazard maps) is, in reality, not simple – because it is non-stable over repeating cycles of USGS generated seismic hazard mapping assessments.

Tying the suggested requirements to installing strong motion instrumentation to a non-stable platform, such as the ground motion parameters depicted in the USGS seismic hazard maps; is neither wise nor practical. Will these installed strong ground motion instruments continue to be maintained throughout the future -- should the "current triggers" dip below the stated thresholds for the requirement for instrumentation?

Furthermore, the current triggers mischaracterize the earthquake risk, because they do not focus on any one particular earthquake source (with its potential maximum magnitude). Focusing on real earthquakes (i.e., Deterministic earthquakes) makes much more sense, since the recording of actual buildings' performances (and deformations) during earthquakes is being sought (and required) fundamentally so as to be able to improve building code requirements -- from lessons learned from actual earthquakes.

Therefore, changing the current triggers from fictitious mathematical models of earthquake hazard [fatally flawed as described in additional Public Comments S107-12 ASCE-7 and S110-12 Figs. 1613.1(1-6) (NEW) Deleting MCE_R] to a robust and stable platform anchored on earthquake source "fault length" and potential maximum "magnitude" -- provides a much simpler and much needed improvement that will not only assuredly improve building code design requirements; but also will prevent the irretrievable loss of valuable earthquake records from affected buildings that were, unfortunately, not instrumented because earthquake shaking was determined (unwisely) by PSHA methodology to be "very unlikely."

So, to protect public safety . . . let's get to the heart of the matter!

"PAY NO ATTENTION TO THAT MAN BEHIND THE CURTAIN!"

http://www.youtube.com/watch?v=XMsO4Mmtp5E&feature=related

S335-12

Final Action: AS AM AMPC____ D

S336-12
M101

Proposed Change as Submitted

Proponent: Michael Mahoney, Federal Emergency Management Agency, representing National Tsunami Hazard Mitigation Program

Revise as follows:

APPENDIX M
TSUNAMI-GENERATED FLOOD HAZARD
The provisions contained in this appendix are not mandatory unless specifically referenced in the adopting ordinance

~~SECTION M101~~
~~TSUNAMI-GENERATED FLOOD HAZARD~~

SECTION M101
GENERAL

M101.1 ~~General~~ Scope. The purpose of this appendix is to provide tsunami regulatory criteria for those communities that have a tsunami hazard and have elected to develop and adopt a map of their tsunami hazard ~~inundation~~ zone. This appendix applies to structures located within an identified Tsunami Hazard Zone, as defined by the Authority Having Jurisdiction.

M101.2 Performance objectives. All structures that are considered either essential to the community and its disaster response or structures that represent a substantial hazard to human life in the event of failure, as defined by Risk Category III and IV as specified under Section 1604.5 of the *International Building Code*, must be protected from tsunamis by either being located outside of the Tsunami Hazard Zone or be designed and constructed to withstand without collapse the specified loads and effects associated with the Maximum Considered Tsunami. For structures in other Risk Categories, life safety protection is to be provided by a community Tsunami Warning and Evacuation Procedure.

M101.3 Tsunami Design Hazard Level. The regulatory criteria contained in this appendix is based on the Maximum Considered Tsunami and its associated flow elevation and velocity, which shall be determined by the Authority Having Jurisdiction. The Maximum Considered Tsunami shall be permitted to be derived either deterministically or probabilistically by the Authority Having Jurisdiction. The Maximum Considered Tsunami shall be represented using a Tsunami Hazard Zone Map adopted by the Authority Having Jurisdiction.

~~M101.2~~ M101.4 Definitions. The following words and terms shall, for the purposes of this appendix, have the meanings shown herein.

MAXIMUM CONSIDERED TSUNAMI. A tsunami that is determined and adopted by the Authority Having Jurisdiction for design purposes and represented using a Tsunami Hazard Zone Map. The Maximum Considered Tsunami shall be taken as having a collapse prevention design equivalent of a 2% probability of being exceeded in a 50-year period or a 2500 year average return period.

TSUNAMI HAZARD ZONE MAP. A map adopted by the ~~community~~ authority having jurisdiction that designates the extent of inundation ~~by a design event~~ the maximum considered tsunami. This map shall ~~be based on the~~ take into consideration any available tsunami inundation map which is developed and provided to a community by either the applicable State agency or the National Oceanic and Atmospheric

~~and Oceanic~~ Administration (NOAA) under the National Tsunami Hazard Mitigation program, but shall be permitted to utilize a different probability or hazard level.

TSUNAMI HAZARD ZONE. The area vulnerable to being flooded or inundated ~~by a design event~~ the maximum considered tsunami as identified on a community's Tsunami Hazard Zone Map.

TSUNAMI VERTICAL EVACUATION REFUGE. A Tsunami Vertical Evacuation Refuge is a structure designated to serve as a point of refuge to which a community's population can evacuate above a tsunami when high ground is not available. It is designed and constructed so as to comply with the applicable provisions of the latest edition of *Guidelines for Design of Structures for Vertical Evacuation from Tsunamis*, published by the Federal Emergency Management Agency (FEMA P-646).

TSUNAMI WARNING AND EVACUATION PROCEDURE. A Tsunami Warning and Evacuation Procedure is a plan and procedure developed and adopted by a community that would receive a tsunami warning from the National Oceanic and Atmospheric Administration (NOAA) at all hours and transmit that warning to its citizens and establishes and designates evacuation routes for its citizens to either high ground or to a designated Tsunami Vertical Evacuation Refuge. Tsunami evacuation procedures may use evacuation maps that are significantly greater in extent than the tsunami hazard zone and are not developed for design purposes. Tsunami evacuation maps are based on the tsunami inundation map which is developed and provided to a community by either the applicable State agency or NOAA under the National Tsunami Hazard Mitigation Program.

SECTION M102
TSUNAMI REGULATORY CRITERIA

~~M101.3~~ M102.1 Establishment of Tsunami Hazard Zone. Where applicable, if a community has adopted a Tsunami Hazard Zone Map, that map shall be used to establish a community's Tsunami Hazard Zone.

~~M101.4~~ M102.2 Construction within the Tsunami Hazard Zone. Construction of structures designated Risk Category III and IV as specified under Section 1604.5 shall be prohibited within a Tsunami Hazard Zone.

Exception~~s~~:

~~1. A vertical evacuation tsunami refuge shall be permitted to be located in a Tsunami Hazard Zone provided it is constructed in accordance with FEMA P646.~~

~~2.~~ Community Risk Category III and IV structures and other critical facilities shall be permitted to be located within the Tsunami Hazard Zone when such a location is necessary to fulfill their function, providing ~~suitable structural and emergency evacuation~~ the following measures have been incorporated.

1. The structure and its foundation shall be designed to resist without collapse all tsunami loads associated with the Maximum Considered Tsunami, including hydrostatic, hydrodynamic, waterborne debris accumulation and impact loads, and scour.
2. A Tsunami Warning and Evacuation Procedure has been incorporated for the facilities.

M102.3 Tsunami Vertical Evacuation Refuge. A structure designated as a Tsunami Vertical Evacuation Refuge shall be permitted to be located in a Tsunami Hazard Zone provided it meets the following criteria:

1. The structure shall be designated as a Tsunami Vertical Evacuation Refuge Structure and shall be capable of being operational within the community's tsunami warning time.
2. The structure shall be designed and constructed so as to comply with the applicable provisions of the latest edition of *Guidelines for Design of Structures for Vertical Evacuation from Tsunamis*, published by the Federal Emergency Management Agency (FEMA P-646).
3. All operational components of the refuge structure necessary for life safety shall be located above the elevation of the Maximum Considered Tsunami. .

The structure and its foundation shall be designed and constructed to resist seismic loads as defined in Chapter 16 of the *International Building Code* for Risk Category IV structures.

M102.4 Tsunami Warning and Evacuation Procedure. The jurisdiction shall have a Tsunami Warning and Evacuation Procedure adopted and enforced by a community that shall be capable of receiving a tsunami warning from the National Oceanic and Atmospheric Administration (NOAA) at all hours and transmit that warning to its citizens and shall establish and designate evacuation routes for its citizens to either high ground or to a designated Tsunami Vertical Evacuation Refuge.

SECTION ~~M102~~ M103
REFERENCED STANDARDS

FEMA P646—08 Guidelines for Design of Structures for Vertical Evacuation from Tsunamis

Reason: On March 11, 2011, a magnitude 9.0 earthquake struck off the coast of Japan. Although Japan is the most advanced country in the world when it comes to tsunami protection measures, 20,000 people perished from the resulting tsunami. While the damage was utterly devastating with over 250,000 structures collapsed, there were many examples of engineered buildings of multi-story construction that survived the earthquake and subsequent tsunami as well as many partially inundated vertical evacuation refuge buildings that successfully saved many lives.

This same type of subduction fault lies off the coastline of Washington, Oregon and northern California, and Alaska and is capable of unleashing a similar magnitude earthquake and resulting tsunami. Furthermore, tsunamis can and have struck the entire Pacific coast, Hawaii, the Caribbean, portions of the Atlantic coast and even within the Gulf of Mexico. While the probability of a damaging tsunami may be low, the consequences would be enormous.

Prior to the 2011 Japan tsunami, the American Society of Civil Engineers/Structural Engineering Institute Standard ASCE/SEI 7 *Minimum Design Loads for Buildings and Other Structures* had formed a new committee to develop a new chapter on tsunami design. While the committee's work is ongoing, we should update Appendix M with some of their work to date relating to the tsunami load criteria and associated design provisions for essential facilities, such as defining a Maximum Considered Tsunami.

The first Appendix M, adopted and published in the 2012 IBC, focused on keeping critical and high risk structures out of the tsunami inundation zone. This revision keeps that same philosophy but expands the description of what is a properly constructed Tsunami Vertical Evacuation Refuge that can withstand without collapse the hydrostatic, hydrodynamic, debris accumulation and impact loads, and scour associated with the Maximum Considered Tsunami.

The National Tsunami Hazard Mitigation Program is proposing this change to keep Appendix M as current as possible with the latest appropriate information to come out of the ongoing ASCE/SEI 7 Tsunami Loads and Effects Committee's development work. This change proposal has been reviewed by the committee.

Cost Impact: Since the primary difference between this proposed change and the current Appendix M is that it would allow for construction within the Tsunami Inundation Zone providing it meets certain criteria, cost impact is not applicable.

M101 (NEW)-S-MAHONEY.doc

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: This proposal to update the tsunami-generated flood hazard appendix is not quite ready as written. Requirements in definitions repeat what is in the code text. Risk Category III structures may pose a problem.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Michael Mahoney, Federal Emergency Management Agency, representing National Tsunami Hazard Mitigation Program, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

APPENDIX M: TSUNAMI GENERATED FLOOD HAZARD

SECTION M101
GENERAL

M101.1 Scope. The purpose of this appendix is to provide tsunami regulatory criteria for those communities that have a recognized tsunami hazard and have developed and adopted a map of their Tsunami Hazard Zone. This appendix applies to structures located within an identified Tsunami Hazard Zone, as defined by the Authority Having Jurisdiction.

M101.2 Performance Objectives. All structures that are considered either essential to the community and its disaster response or structures that represent a substantial hazard to human life in the event of failure, as defined by Risk Category III and IV as specified under Section 1604.5 of the *International Building Code*, must be protected from tsunamis by either being located outside of the Tsunami Hazard Zone or be designed and constructed to withstand without collapse the specified loads and effects associated with the Maximum Considered Tsunami. For structures in other Risk Categories, life safety protection is to be provided by a community Tsunami Warning and Evacuation Procedure.

M101.3 Tsunami Design Hazard Level. The regulatory criteria contained in this appendix is based on the Maximum Considered Tsunami and its associated flow elevation and velocity~~, which shall be determined by the Authority Having Jurisdiction. The Maximum Considered Tsunami shall be permitted to be derived either deterministically or probabilistically by the Authority Having Jurisdiction. The Maximum Considered Tsunami shall be represented using a Tsunami Hazard Zone Map adopted by the Authority Having Jurisdiction~~.

M101.4 Definitions. The following words and terms shall, for the purposes of this appendix, have the meanings shown herein. Refer to Chapter 2 of the *International Building Code* for general definitions.

MAXIMUM CONSIDERED TSUNAMI. A tsunami that is determined and adopted by the Authority Having Jurisdiction for design purposes and represented using a Tsunami Hazard Zone Map. The Maximum Considered Tsunami ~~shall be developed~~ <u>is defined</u> as a collapse prevention design equivalent of a 2500 year probability of recurrence.

TSUNAMI HAZARD ZONE MAP. A map adopted by the Authority Having Jurisdiction that designates the extent and depth of inundation by the Maximum Considered Tsunami. ~~This map should be based on shall take into consideration any available tsunami inundation map that is developed and provided to a community by either the applicable State agency or the National Oceanic and Atmospheric and Oceanic Administration (NOAA) under the National Tsunami Hazard Mitigation Program.~~

TSUNAMI HAZARD ZONE. The area vulnerable to being flooded or inundated by the Maximum Considered Tsunami as identified on a community's Tsunami Hazard Zone Map.

TSUNAMI VERTICAL EVACUATION REFUGE. A ~~Tsunami Vertical Evacuation Refuge is a~~ structure designated to serve as a point of refuge to which a community's population can evacuate above a tsunami ~~if high ground is not available. It is designed and constructed so as to comply with the applicable provisions of the latest edition of *Guidelines for Design of Structures for Vertical Evacuation from Tsunamis*, published by the Federal Emergency Management Agency (FEMA P-646).~~

TSUNAMI WARNING AND EVACUATION PROCEDURE. A ~~Tsunami Warning and Evacuation Procedure is a plan and~~ procedure developed and adopted by a community that would receive ~~a tsunami warning from the National Oceanic and Atmospheric Administration (NOAA) at all hours~~ and transmit ~~that~~ a tsunami warning to its citizens and establishes and designates evacuation routes for its citizens to either high ground or to a designated Tsunami Vertical Evacuation Refuge(s)<u> or both</u>. ~~Tsunami evacuation procedures typically utilize evacuation maps that are significantly greater in extent than the tsunami hazard zone and are not developed for design purposes. Tsunami evacuation maps are based on the tsunami inundation map which is developed and provided to a community by either the applicable State agency or NOAA under the National Tsunami Hazard Mitigation Program.~~

SECTION M102
TSUNAMI REGULATORY CRITERIA

<u>**M102.1 Adoption of Tsunami Hazard Zone Map.** Where applicable, a community shall adopt a Tsunami Hazard Zone Map. The Tsunami Hazard Zone Map shall be based on the Maximum Considered Tsunami. The Maximum Considered Tsunami shall provide the collapse prevention design equivalent of a 2500 year probability of recurrence. The Tsunami Hazard Zone Map shall be</u>

permitted to take into consideration available tsunami inundation mapping developed by either the applicable State agency or the National Oceanic and Atmospheric Administration (NOAA) under the National Tsunami Hazard Mitigation Program.

M102.~~1~~2 Establishment of Tsunami Hazard Zone. Where ~~applicable, if~~ a community has adopted a Tsunami Hazard Zone Map, that map shall be used to establish a community's Tsunami Hazard Zone(s).

M102.~~2~~3 Construction within the Tsunami Hazard Zone. Construction of structures designated Risk Category III and IV as specified under Section 1604.5 shall be prohibited within a Tsunami Hazard Zone.

Exception: ~~Community~~ Risk Category III and IV structures ~~and other critical facilities~~ shall be permitted to be located within the Tsunami Hazard Zone ~~when such a location is necessary to fulfill their function~~, providing the following measures have been incorporated:

1. The structure and its foundation shall be designed to resist without collapse all tsunami loads associated with the Maximum Considered Tsunami, including hydrostatic, hydrodynamic, waterborne debris accumulation and impact loads, and scour, and,
2. A Tsunami Warning and Evacuation Procedure has been incorporated for the facilities such that the building occupants shall be able to reach high ground or a Tsunami Vertical Evacuation Refuge within the allowable warning time.

M102.~~3~~4 Tsunami Vertical Evacuation Refuge. A structure designated as a Tsunami Vertical Evacuation Refuge shall be permitted to be located in a Tsunami Hazard Zone provided it meets the following criteria:

1. The structure shall be designated as a Tsunami Vertical Evacuation Refuge Structure and shall be capable of being operational within the community's tsunami warning time.
2. The structure shall be designed and constructed so as to comply with the applicable provisions of ~~the latest edition of~~ *Guidelines for Design of Structures for Vertical Evacuation from Tsunamis*, published by the Federal Emergency Management Agency (FEMA P-646).
3. All ~~operational~~ components of the refuge structure necessary for ~~life safety~~ its operation shall be either be located above the elevation of the Maximum Considered Tsunami or shall be designed so as to remain functional after tsunami inundation.
4. The structure and its foundation shall be designed and constructed to resist seismic loads as defined in Chapter 16 of the *International Building Code* for Risk Category IV structures.

M102.~~4~~5 Tsunami Warning and Evacuation Procedure. The jurisdiction shall have a Tsunami Warning and Evacuation Procedure adopted and enforced ~~by a community~~ that shall be capable of receiving a tsunami warning from the National Oceanic and Atmospheric Administration (NOAA) at all hours and transmit that warning to its citizens. The Procedure ~~and~~ shall establish and designate evacuation routes for its citizens to either high ground or to ~~a~~ designated Tsunami Vertical Evacuation Refuge(s) utilizing tsunami inundation maps developed and provided to a community by either the applicable State agency or NOAA under the National Tsunami Hazard Mitigation Program.

SECTION 103
REFERENCED STANDARDS

FEMA

P646-~~08~~12 Guidelines for Design of Vertical Evacuation Structures For Vertical Evacuation from Tsunamis

Commenter's Reason: The Appendix M text submitted under S336 has been revised as shown to address comments raised by the Structural Code Change committee, including duplicate requirements in the definitions and how Risk Category III buildings are addressed

On March 11, 2011, a magnitude 9.0 earthquake struck off the coast of Japan. Although Japan is the most advanced country in the world when it comes to tsunami protection measures, 20,000 people perished from the resulting tsunami. While the damage was utterly devastating with over 250,000 structures collapsed, there were many examples of engineered buildings of multi-story construction that survived the earthquake and subsequent tsunami as well as many partially inundated vertical evacuation refuge buildings that successfully saved many lives.

This same type of subduction fault lies off the coastline of Washington, Oregon and northern California, and Alaska and is capable of unleashing a similar magnitude earthquake and resulting tsunami. Furthermore, tsunamis can and have struck the entire Pacific coast, Hawaii, the Caribbean, portions of the Atlantic coast and even within the Gulf of Mexico. While the probability of a damaging tsunami may be low, the consequences would be enormous.

Prior to the 2011 Japan tsunami, the American Society of Civil Engineers/Structural Engineering Institute Standard ASCE/SEI 7 *Minimum Design Loads for Buildings and Other Structures* had formed a new committee to develop a new chapter on tsunami design. While the committee's work is ongoing, we should update Appendix M with some of their work to date relating to the tsunami load criteria and associated design provisions for essential facilities, such as defining a Maximum Considered Tsunami.

The first Appendix M, adopted and published in the 2012 IBC, focused on keeping critical and high risk structures out of the tsunami inundation zone. This revision keeps that same philosophy but expands the description of what is a properly constructed

Tsunami Vertical Evacuation Refuge that can withstand without collapse the hydrostatic, hydrodynamic, debris accumulation and impact loads, and scour associated with the Maximum Considered Tsunami.

The National Tsunami Hazard Mitigation Program is proposing this change to keep Appendix M as current as possible with the latest appropriate information to come out of the ongoing ASCE/SEI 7 Tsunami Loads and Effects Committee's development work. This change proposal has been reviewed by the committee.

S336-12

Final Action: AS AM AMPC____ D

S337-12
M101

<u>Proposed Change as Submitted</u>

Proponent: James Bela, Oregon Earthquake Awareness, representing self

Delete without substitution:

~~**SECTION M101 TSUNAMI-GENERATED FLOOD HAZARD**~~

~~**M101.1 General.** The purpose of this appendix is to provide tsunami regulatory criteria for those communities that have a tsunami hazard and have elected to develop and adopt a map of their tsunami hazard inundation zone.~~

~~**M101.2 Definitions.** The following words and terms shall, for the purposes of this appendix, have the meanings shown herein.~~

~~**TSUNAMI HAZARD ZONE MAP.** A map adopted by the community that designates the extent of inundation by a design event tsunami. This map shall be based on the tsunami inundation map which is developed and provided to a community by either the applicable State agency or the National Atmospheric and Oceanic Administration (NOAA) under the National Tsunami Hazard Mitigation program, but shall be permitted to utilize a different probability or hazard level.~~

~~**TSUNAMI HAZARD ZONE.** The area vulnerable to being flooded or inundated by a design event tsunami as identified on a community's Tsunami Hazard Zone Map.~~

~~**M101.3 Establishment of Tsunami Hazard Zone.** Where applicable, if a community has adopted a Tsunami Hazard Zone Map, that map shall be used to establish a community's Tsunami Hazard Zone.~~

~~**M101.4 Construction within the Tsunami Hazard Zone.** Construction of structures designated Risk Category III and IV as specified under Section 1604.5 shall be prohibited within a Tsunami Hazard Zone.~~

~~**Exceptions:**~~

1. ~~A vertical evacuation tsunami refuge shall be permitted to be located in a Tsunami Hazard Zone provided it is constructed in accordance with FEMA P646.~~
2. ~~Community critical facilities shall be permitted to be located within the Tsunami Hazard Zone when such a location is necessary to fulfill their function, providing suitable structural and emergency evacuation measures have been incorporated.~~

~~**SECTION M102 REFERENCED STANDARDS**~~

~~FEMA P646—08 Guidelines for Design of M101.4 Structures for Vertical Evacuation from Tsunamis~~

Reason: Given the recent M 9.1 Great 11 March 2011 Tohoku Earthquake and Tsunami disaster in Japan, I would view this code section to be extremely dangerous to public safety; and I believe that it should be removed. Vertical evacuation structures were overtopped in the Tohoku earthquake, and people were killed as a result. Even concrete structures (previously assumed to be "invincible" were overturned and destroyed.

This "weak" and very problematical FEMA effort has copied the same "failed approach" for U.S. Building design practice – it presupposes a "design tsunami event" – and somehow probabilistically determined. No one is accountable for its failures and tragic loss-of-life that could result if such a standard were "followed." They are too uncertain for "local tsunami" generated waves and coastal innundation.

There needs to be a more "stringent" for accepting something into the building code as a "standard". The fact that it is located in the appendix speaks for itself.

For further background information:

Union Frontiers of Geophysics Lecture: Tohoku to Tsunami: Personal Account From Science to Experience by Hiroo Kanamori
http://sites.agu.org/fallmeeting/scientific-program/lectures/

Insights from the great 2011 Japan earthquake:
The diverse set of waves generated in Earth's interior, oceans, and atmosphere during the devastating Tohoku-oki earthquake reveal some extraordinary geophysics -- Thorne Lay and Hiroo Kanamori
http://www.physicstoday.org/resource/1/phtoad/v64/i12/p33_s1?bypassSSO=1

S23C Gutenberg Lecture*
Great Earthquake Ruptures in the Age of Seismo-Geodesy
Presented by Thorne Lay, University of California, Santa Cruz, USA
http://sites.agu.org/fallmeeting/scientific-program/lectures/bowie-and-named-lectures/6dec/

U33C The Great 11 March 2011 Tohoku Earthquake I
Moscone South, Room 104, 1340h
http://sites.agu.org/fallmeeting/scientific-program/sessions-on-demand-7-december/

U34A The Great 11 March 2011 Tohoku Earthquake II
Moscone South, Room 104, 1600h
http://sites.agu.org/fallmeeting/scientific-program/sessions-on-demand-7-december/

U41D The Great 11 March 2011 Tohoku Earthquake III
Moscone South, Room 104, 0800h
http://sites.agu.org/fallmeeting/scientific-program/sessions-on-demand-8-december/

U42A The Great 11 March 2011 Tohoku Earthquake IV
Moscone South, Room 104, 1020h
http://sites.agu.org/fallmeeting/scientific-program/sessions-on-demand-8-december/

U23C Predicting Extreme Events in Natural and Socioeconomic Systems:
State-of-the-Art and Emerging Possibilities II
Moscone South, Room 103, 1340h
http://sites.agu.org/fallmeeting/scientific-program/sessions-on-demand-6-december/

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

M101-S-BELA.doc

Public Hearing Results

Committee Action: Disapproved

Committee Reason: The committee prefers to retain the appendix chapter on tsunami-generated flood hazard rather than delete it. The appendix may need improvement, but it also needs to stay in the code.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

James Bela, Oregon Earthquake Awareness, representing self, requests Approval as Submitted.

Commenter's Reason: The Committee Action for Disapproval: (a) incorrectly substitutes the committee's so-called "preference" for retaining Appendix M101 in the code for any actual, specific and defensible "reasons in support of that action; and (b) also ignores the very specific dangers to public safety embodied in this very deficient and impractical FEMA product. One of the saddest commentaries on FEMA funded projects is that, the more "public money" that is expended, the less knowledge is produced by FEMA, and the more unsafe the general public becomes.

The public does not and should not support the adoption into the IBC Structural Building Code of so-called "standards" that so prominently eschew responsibility and liability for the use of those very same products. "FEMA P646—08 Guidelines for Design of M101.4 Structures for Vertical Evacuation from Tsunamis," which was developed before the 2011 M 9 northern Japan giant megathrust earthquake and tsunami, and which further invalidated many (if not all) of its assumptions; is not truly a standard – it certainly is not maintained in any formal, qualified and practical way.

When you adopt a code, you are setting the level of risk to which the public is exposed. Furthermore, no one member of the Structural Code Committee is anywhere near to being qualified as an expert in tsunamis; as witnessed by the lack of any clear or reasonable objections to this Code Change Proposal, As Submitted. This was the same situation as in Japan (lack of tsunami expertise), which emboldened the committee evaluating the earthquake safety of the Tohoku nuclear power plants to "dismiss" the very real and documented threat from tsunamis along the entire Japan trench, and along the Sendai plane where the nuclear plants were located, in particular.

A few important points are:

1. The definition of the so-called Tsunami Hazard Zone is too ambiguous, or "free floating," if you will. It permits pretty much any criterion for its definition, including probabilistically defined tsunami hazard zones, which are not only unsafe, but a clear threat to public safety and survivability. Probabilistic hazard estimates, which are permitted, are not only mathematically flawed, as previously described in Public Comments S107-12 ASCE 7 and S110-12 Figs. 1613.3.1 MCEsubR Maps; but recent deadly tsunamis from the M 9 2004 Indonesian and M 9 2011 Japan subduction zone megathrust earthquakes have, unfortunately, provided clear examples of the "danger to public safety" where the hazard is defined by what seems "probable," as compared to what is "possible."
2. "TABLE 1604.5 RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES," although the only thing available; does not really work for tsunami inundation hazards – where debris generation, fire potential and high occupancy loads are all prime concerns. Risk Category II structures are not necessarily de-facto either wise or safe choices for designated Tsunami Hazard Zones.
3. Vertical evacuation structures are always dangerously subject to "overtopping," as, in fact, did occur in the March 11, 2011 M 9.0 Tohoku megathrust earthquake and tsunami. It is always best and safest to evacuate." There may be higher than planned for coastal inundation due to: (a) coastal configuration and offshore bathymetry; (b) tsunami wave resonance within a bay; (c) submarine landslides and their accompanying slide-generated waves; (d) multiple tsunami generating sources; (e) reflection and refraction of tsunami waves, along with "edge wave" effects; and (f) compounding effects of coastal subsidence combined with estuarine flooding.

In summary, since you cannot come close to assuring safety for any occupants seeking refuge in any such vertical evacuation structures; I believe it is unwise to codify their acceptance in this Appendix. In the final analysis, when you adopt a code you are really saying: "This is OK." Addressing the tsunami hazard requires much more thought . . . than that embodied in this abbreviated and cryptic Appendix M

And this is not OK! See the two attached photos.

So, to protect public safety against the threat of a tsunami . . . it takes a heart, it takes a brain, and it takes courage!

"PAY NO ATTENTION TO THAT MAN BEHIND THE CURTAIN!"

http://www.youtube.com/watch?v=XMsO4Mmtp5E&feature=related

S337-12

Final Action: AS AM AMPC____ D