# INTERNATIONAL BUILDING CODE – STRUCTURAL

## S1-07/08
### 1502.1

*Proposed Change as Submitted:*

**Proponent:** Mike Ennis, SPRI, Inc.

**Revise as follows:**

**1502.1 General.** The following words and terms shall, for the purposes of this chapter and as used elsewhere in this code, have the meanings shown herein.

**AGGREGATE (Supp).** In roofing, crushed stone, crushed slag or water-worn gravel used for surfacing for roof coverings <u>as defined in ASTM D 1863</u>.

**Reason:** This code change proposal clarifies the definition of aggregate, tying it into a current IBC reference standard: ASTM D1863–03 Specification for Mineral Aggregate Used on Built-up Roofs.

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

**Committee Action:** **Disapproved**

**Committee Reason:** The proposal would create a conflict between ASTM D448 and SPRI RP4 standards. Also under ASTM D 1863 ninety percent of aggregate would be finer than three-quarters on an inch.

**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 (SPRI), requests Approval as Submitted.**

**Commenter's Reason:** This proposed Code Change will clarify the definition of aggregate used in roofing applications. The proposal is to include ASTM D1863 Specification for Mineral Aggregate Used on Built-up Roofs, in the definition.

Extensive testing and field experience has demonstrated the need to subject aggregate used as a surfacing for built-up roofing assemblies and ballast stones used to hold the roof system in place to different requirements to mitigate the potential for blow-off. This modification will provide the necessary clarification so that the proper requirements can be implemented in the Code.

Final Action: AS AM AMPC____ D

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## S3-07/08
### 1502.1

*Proposed Change as Submitted:*

**Proponent:** Mark S. Graham, National Roofing Contractors Association, representing Technical Operations Committee of the National Roofing Contractors Association

**Add new definitions as follows:**

**SECTION 1502**
**DEFINITIONS**

<u>**LANDSCAPED ROOF:** See "Roof garden".</u>

**<u>ROOF GARDEN:</u>** <u>A roof area of plantings or landscaping installed above a waterproofed substrate at any building level that is separated from the ground beneath by a man-made structure.</u>

**Reason:** This proposed code change is intended to clarify the intent of the Code by adding definitions for landscaped roofs and roof gardens. The two terms are currently used in Sec. 1507.16 (FS210-06/07), Sec. 1607.11.2.2 and Sec. 1607.11.2.3. The specific definition for the term used here is taken from the National Roofing Contractors Association's (NRCA's) *The NRCA Green Roof Systems Manual—2007 Edition*.

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

**Committee Action:** **Disapproved**

**Committee Reason:** The proposal could lead to confusion over the application of roof live load criteria. The proposed definitions of landscaped roofs and roof garden may be necessary, but this potential confusion needs to be resolved.

**Assembly Action:** **None**

## *Individual Consideration Agenda*

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

*Public Comment:*

**Mark Graham, National Roofing Contractors Association (NRCA), Technical Operations Committee of the National Roofing Contractors Association, requests Approval as Modified by this Public Comment.**

**Replace proposal as follows:**

**SECTION 1502**
**DEFINITIONS**

**<u>ROOF GARDEN</u>**<u>: A roof area of plantings or landscaping installed above a waterproofed substrate over a roof deck that is designed for assembly purposes.</u>

**<u>LANDSCAPED ROOF</u>**<u>: A roof area of plantings or landscaping installed above a waterproofed substrate over a roof deck.</u>

**Commenter's Reason:** The proposed modification is intended to clarify the intent of the Code by adding specific definitions for the terms "landscaped roof" and "roof garden".

The two terms are currently used in Sec. 1507.16 (2007)—Roof gardens and landscaped roofs without specific definitions. The definitions provided here are based upon comments received at the Palm Springs public hearing.

Final Action: AS AM AMPC____ D

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# S5-07/08, Part I

## 1503.6 (New), Figure 1503.6 (New), 1503.6.1 (New), 1503.6.2 (New), 1507.2.6 (New), 1507.3.6 (New), 1507.4.4 (New), 1507.5.5 (New), 1507.7.6 (New), 1507.8.7 (New), 1507.9.8 (New), 1510.3;

***NOTE: PART II DID NOT RECEIVE A PUBLIC COMMENT AND IS ON THE CONSENT AGENDA. PART II IS REPRODUCED FOR INFORMATIONAL PURPOSES ONLY FOLLOWING <u>ALL</u> OF PART I.***

## *Proposed Change as Submitted:*

**Proponent:** Wanda D. Edwards, Institute for Business and Home Safety

**PART I – IBC STRUCTURAL**

**1. Add new text as follows:**

**<u>1503.6 Hail exposure.</u>** <u>Hail exposure, as specified in Sections 1503.6.1 and 1503.6.2, shall be determined using Figure 1503.6.</u>

**1503.6.1 Moderate hail exposure.** One or more hail days with hail diameters greater than 1.5 in (38 mm) in a twenty (20) year period.

**1503.6.2 Severe hail exposure.** One or more hail days with hail diameters greater than or equal to 2.0 in (50 mm) in a twenty (20) year period.

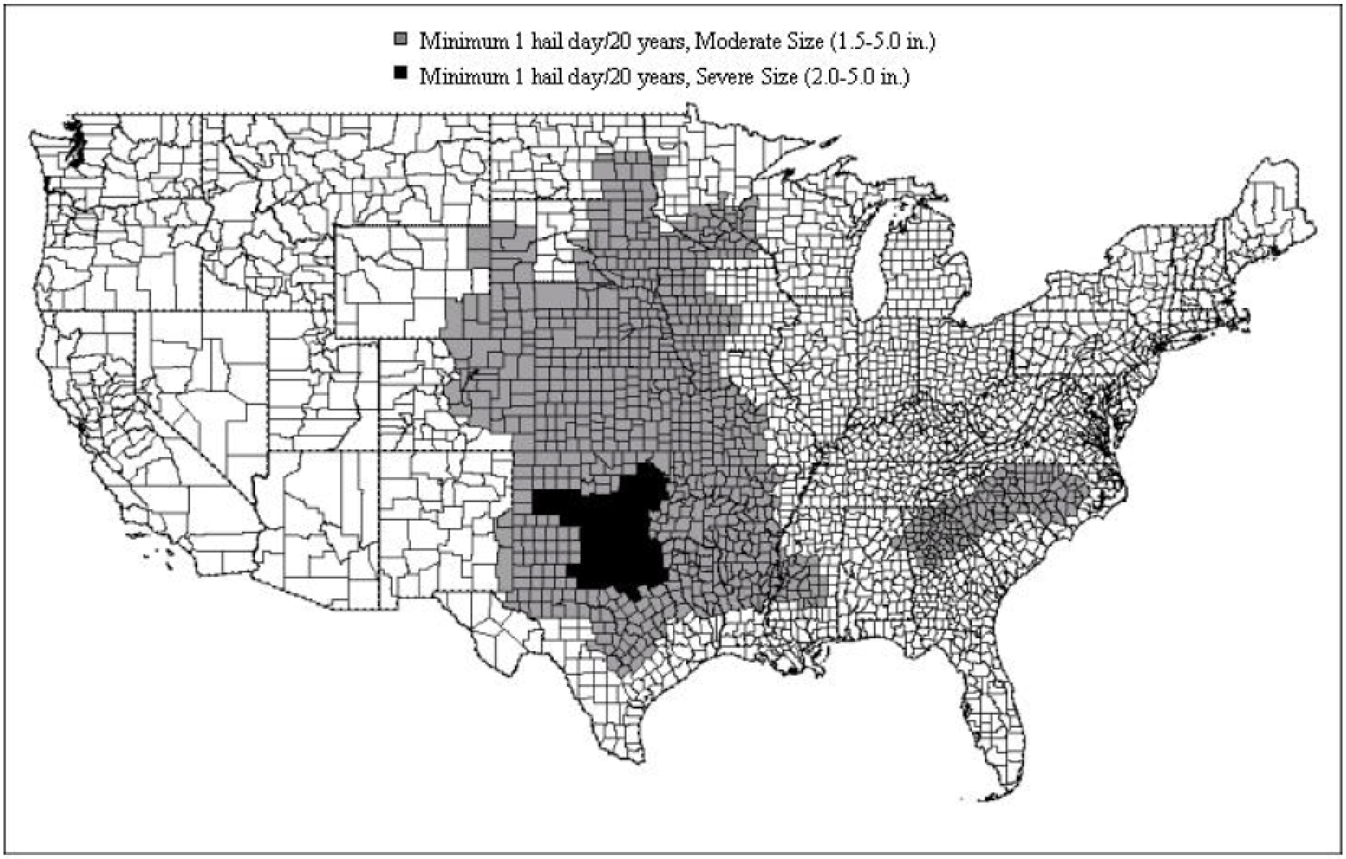


**FIGURE 1503.6**
**HAIL EXPOSURE**

**1507.2.6 Asphalt shingles subject to severe hail exposure.** Asphalt shingles used in regions where hail exposure is Severe, as determined in Section 1503.6, shall comply with this Section. Asphalt shingles used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with UL 2218-02.

**1507.3.6 Clay or concrete tile subject to severe hail exposure.** Clay or concrete tile used on roofs in regions where hail exposure is Severe, as determined in Section 1503.6, shall comply with this Section. Clay or concrete tile used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with FM 4473-05.

**1507.4.4 Metal roof panels subject to severe hail exposure.** Metal roof panels used in regions where hail exposure is Severe, as determined in Section 1503.6, shall comply with this Section. Metal roof panels used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with UL 2218-02.

**1507.5.5 Metal roof shingles subject to severe hail exposure.** Metal roof shingles used in regions where hail exposure is Severe, as determined in Section 1503.6, shall comply with this Section. Metal roof shingles used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with UL 2218-02.

**1507.7.6 Slate shingles subject to severe hail exposure.** Slate shingles used in regions where hail exposure is Severe, as determined in Section 1503.6, shall comply with this Section. Slate shingles used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with FM 4473-05.

**1507.8.7 Wood shingles subject to severe hail exposure.** Wood shingles used in regions where hail exposure is Severe, as determined in Section 1503.6, shall comply with this Section. Wood shingles used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with UL 2218-02.

**1507.9.8 Wood shakes subject to Severe hail exposure.** Wood shakes used in regions where hail exposure is Severe, as determined in Section 1503.6, shall comply with this Section. Wood shakes used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with UL 2218-02.

**Add standards to Chapter 35 as follows:**

**Underwriters Laboratories**
UL 2218-02 Standard for Impact Resistance of Prepared Roof Covering Materials

**Factory Mutual**
FM 4473-05 Specification Test Standard for Impact Resistance Testing of Rigid Roofing Materials by Impacting With Freezer Balls

**2. Revise as follows:**

**1510.3 (Supp) 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, concrete ~~cement~~ or asbestos-~~cement~~ concrete tile.
3. Where the existing roof has two or more applications of any type of roof covering.
4. For asphalt shingle roofs, metal roof panels, and metal roof shingles, when the building is located in an area subject to moderate or severe hail exposure according to Figure 1503.6 unless the roof covering has been successfully tested as required in Sections 1507.2.6, 1507.4.4, and 1507.5.5 for installation over an existing 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.

**Reason:** (IBC) Each year the United States experience 3,000 hail storms. Damages from these storms can run in the billions. This code change proposes to include in the Building Code a map showing moderate and severe hail prone areas. The map was developed by Haag Engineering utilizing data obtained from the National Climate Data Center (www. ncdc.noaa.gov/oa/ncdc.html). Data was compiled from the past twenty years, and included airport data as well as eye witness accounts. A computer program was created to analyze and map the data. The program considered that reports in rural areas were less than those in more populous areas, otherwise the data would conclude that it hailed more in populous areas versus sparsely populated areas. The program also considered that eyewitness accounts tend to overestimate the size of the hail. These considerations, plus others, were used to provide a conservative interpretation of the data. Areas shown on the map as severe hail exposure represent a risk of 1 or more hail days in 20 years with hail diameters greater than 2 inches. Areas shown as moderate risk represent a risk of 1 hail day in 20 years with hail diameters greater than 1.5 inches. It is important to note that the map is based upon **actual** meteorological data rather than modeling, such as the ASCE-7 wind map.

Also, this change requires that all existing roofing coverings be removed prior to installing new asphalt shingles in hail prone areas. The stiffness plays an important role in hail resistance. Too much flexibility in the system reduces the effectiveness of the of the system's resistance. Recovering over an existing roof system significantly reduces the impact resistance of the roof. Hailstones impacting a roof with two or more layers of asphalt shingles results in a "sponge" effect with the top layer being more susceptible to penetration by the hailstone, thus increasing the potential for water penetration under the roof covering. This sponge effect was observed and reported by the Roofing Industry Committee on Weather Issues (RICOWI) in its Hailstorm Investigation Report. The report confirmed that roofs with asphalt shingles overlaid over other roof coverings experienced damage at smaller size hail than roofs on solid decks.

Studies show that roofs in severe hail prone areas require replacement every seven years on average. With deductibles running as high as $2,000 to $5,000, costs for homeowners can be significant to replace roof covering every seven years. The additional cost of installing impact resistant roofing instead of conventional roofing is approximately $75/square installed. For a home requiring 30 squares of shingles, the additional cost for impact resistant roofing is $2,200. Combined with discounts on insurance premiums for using impact resistant roofing, it is easy to see if the impact roof lasts one storm without requiring replacement, it pays for itself.

The change also provides for two test standards, UL 2218 and FM 4473. Products classified in accordance with UL 2218 have been shown to sustain significantly less damage after being impacted by hailstones with diameters between 1.0 and 2.0 inches. FM 4473, which uses ice balls as an impact medium, allows relative comparisons of impact resistance between rigid roofing materials. In areas of the country where damaging hail is expected within the design life of a roof covering, building codes should mandate that such impact resistant roofing systems be used.

The hail exposure map is currently included in the International Residential Code. Including the map in the International Building Code would make the codes consistent with one another.

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

**Analysis:** Review of proposed new standard FM 4473 (2005) indicated that, in the opinion of ICC Staff, the standard did not comply with ICC standards criteria, Section 3.6.2.6.

**Analysis:** Review of proposed new standard UL 2218-96 indicated that, in the opinion of ICC Staff, the standard did not comply with ICC standards criteria, Section 3.6.2.3.

## PART I – IBC STRUCTURAL
## Committee Action: Disapproved

**Committee Reason:** Disapproval was requested by the proponent. Some of the proposed terminology is confusing. In addition, references to testing sections should be corrected.

## Assembly Action: None

## *Individual Consideration Agenda*

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

*Public Comment:*

### Wanda D. Edwards, Institute for Business & Home Safety, requests Approval as Modified by this Public Comment.

**Modify proposal as follows**:

**1503.6 Hail exposure.** Hail exposure, as specified in Sections 1503.6.1 and 1503.6.2, shall be determined using Figure 1503.6.

**1503.6.1 Moderate hail exposure.** One or more hail days with hail diameters greater than 1.5 in (38 mm) in a twenty (20) year period.

**1503.6.2 Severe hail exposure.** One or more hail days with hail diameters greater than or equal to 2.0 in (50 mm) in a twenty (20) year period.

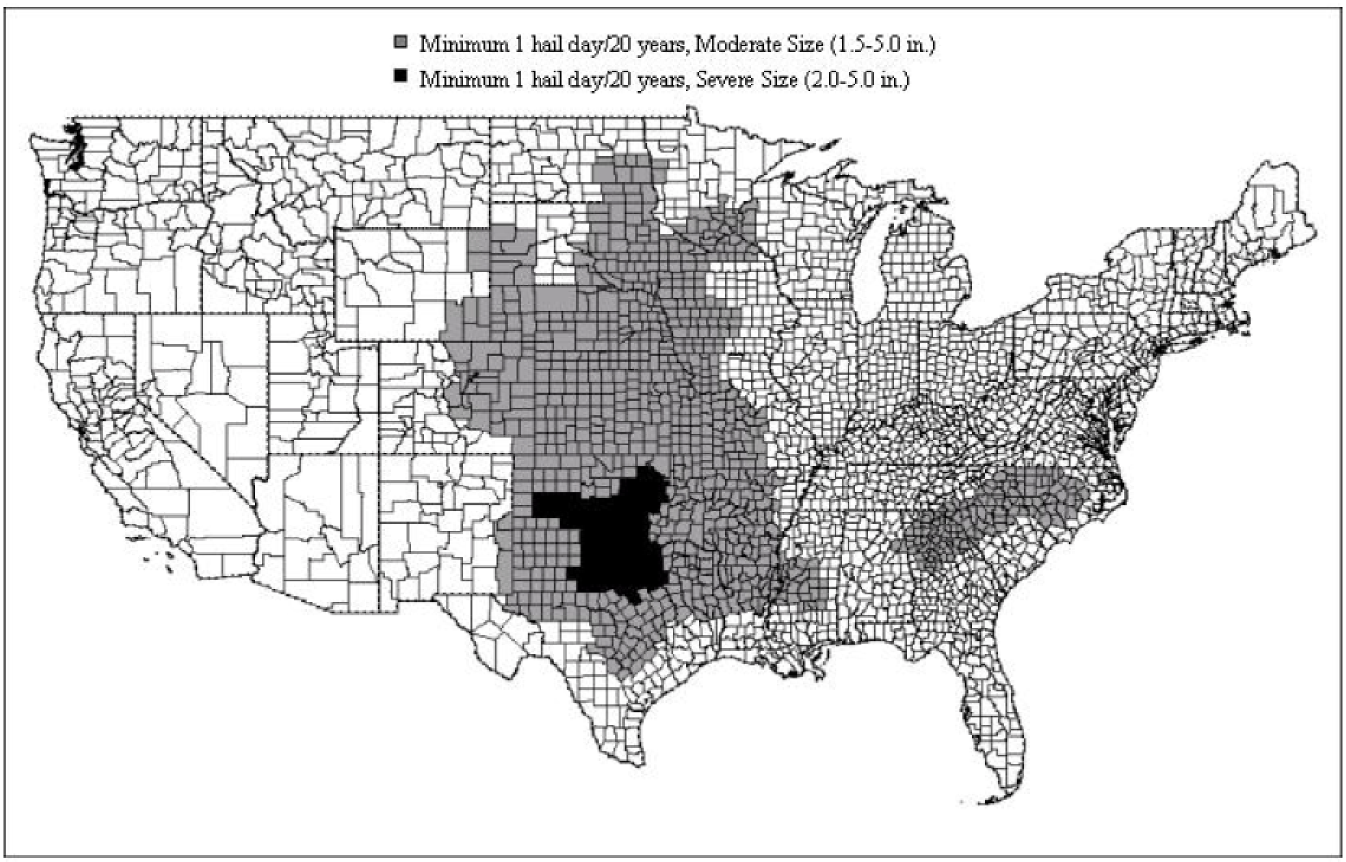


**FIGURE 1503.6**
**HAIL EXPOSURE**

~~**1507.2.6 Asphalt shingles subject to severe hail exposure.** Asphalt shingles used in regions where hail exposure is Severe, as determined in Section 1503.6, shall comply with this Section. Asphalt shingles used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with UL 2218.~~

~~**1507.3.6 Clay or concrete tile subject to severe hail exposure.** Clay or concrete tile used on roofs in regions where hail exposure is Severe, as determined in Section 1503.6, shall comply with this Section. Clay or concrete tile used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with FM 4473.~~

~~**1507.4.4 Metal roof panels subject to severe hail exposure.** Metal roof panels used in regions where hail exposure is Severe, as determined in Section 1503.6, shall comply with this Section. Metal roof panels used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with UL 2218.~~

~~**1507.5.5 Metal roof shingles subject to severe hail exposure.** Metal roof shingles used in regions where hail exposure is Severe, as determined in Section 1503.6, shall comply with this Section. Metal roof shingles used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with UL 2218.~~

~~**1507.7.6 Slate shingles subject to severe hail exposure.** Slate shingles used in regions where hail exposure is Severe, as determined in Section 1503.6, shall comply with this Section. Slate shingles used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with FM 4473.~~

~~**1507.8.7 Wood shingles subject to severe hail exposure.** Wood shingles used in regions where hail exposure is Severe, as determined in Section 1503.6, shall comply with this Section. Wood shingles used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with UL 2218.~~

~~**1507.9.8 Wood shakes subject to Severe hail exposure.** Wood shakes used in regions where hail exposure is Severe, as determined in Section 1503.6, shall comply with this Section. Wood shakes used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with UL 2218.~~

**1510.3 (Supp) 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, ~~concrete~~ cement or asbestos-cement ~~concrete~~ tile.
3. Where the existing roof has two or more applications of any type of roof covering.
4. ~~For asphalt shingle roofs, metal roof panels, and metal roof shingles, when the building is located in an area subject to moderate or severe hail exposure according to Figure 1503.6 unless the roof covering has been successfully tested as required in Sections 1507.2.6, 1507.4.4, and 1507.5.5 for installation over an existing 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.

**Commenter's Reason:** The modification deletes the requirement for the installation of impact resistant roofing in severe hail prone areas. The proposal will add the definitions for moderate and severe hail exposure areas and add the map. This change will make the IBC consistent with the IRC.

Final Action: AS AM AMPC____ D

***NOTE: PART II REPRODUCED FOR INFORMATIONAL PURPOSES ONLY – SEE ABOVE***

## S5-07/08, Part II – IRC BUILDING/ENERGY

**1. Add new text as follows:**

**R905.2.5 Asphalt shingles subject to severe hail exposure.** Asphalt shingles used in regions where hail exposure is Severe, as determined in Section R903.5, shall comply with this Section. Asphalt shingles used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with UL 2218.

**R905.3.6 Clay or concrete tile subject to severe hail exposure.** Clay or concrete tile used on roofs in regions where hail exposure is Severe, as determined in Section R903.5, shall comply with Section R905.3.6.2. Clay or concrete tile used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with FM 4473.

**R905.4.6 Metal roof shingles subject to severe hail exposure.** Metal roof shingles used in regions where hail exposure is Severe, as determined in Section R903.5, shall comply with this Section. Metal roof shingles used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with UL 2218.

**R905.6.6 Slate and slate-type shingles subject to Severe hail exposure.** Slate and slate-type shingles used in regions where hail exposure is Severe, as determined in Section R903.5, shall comply with this Section. Slate and slate-type shingles used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with FM 4473.

**R905.7.6 Wood shingles subject to severe hail exposure.** Wood shingles used in regions where hail exposure is Severe, as determined in Section R903.5, shall comply with that Section. Wood shingles used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with UL 2218.

**R905.8.8 Wood shakes subject to severe hail exposure.** Wood shingles used in regions where hail exposure is Severe, as determined in Section R903.5, shall comply with this Section. Wood shakes used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with UL 2218.

**R905.10.5 Metal roof panels subject to severe hail exposure.** Metal roof panels used in regions where hail exposure is Severe, as determined in Section R903.5, shall comply with this Section. Metal roof panels used in regions where hail exposure is Severe shall be tested, classified, and labeled as Class 4 in accordance with UL 2218.

**2. Revise text as follows:**

**R907.3 Re-covering versus replacement.** New roof coverings shall not be installed without first removing existing roof coverings 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~~ concrete or asbestos-~~cement~~ concrete tile.
3. Where the existing roof has two or more applications of any type of roof covering.
4. For asphalt shingles, metal roof shingles, and metal roof panels when the building is located in an area subject to moderate or severe hail exposure according to Figure R903.5 unless the roof covering has been successfully tested as required in Sections R905.2.5, R905.4.6, and R905.10.5 for installation over an existing 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. Installation of metal panel, metal shingle, and concrete and clay tile roof coverings over existing wood shake roofs shall be permitted when the application is in accordance with Section R907.4.
3. The application of new protective coating over existing spray polyurethane foam roofing systems shall be permitted without tear-off of existing roof coverings.

**Reason:** (IRC) This proposal will require the installation of impact resistant roof coverings in severe hail prone areas. A recent study conducted by the Institute for Business and Home Safety (IBHS) has shown that approximately 44% of all "non-impact resistant" single family residential roofs investigated in the study needed repair or replacement after being struck by hailstones with diameters between 1.0" and 2.0". Areas shown on the map as severe hail exposure represent a risk of 1 or more hail days in 20 years with hail diameters greater than 2 inches.

Other studies show that roofs in severe hail prone areas require replacement every seven years on average. With deductibles running as high as $2,000 to $5,000, costs for homeowners can be significant to replace roof covering every seven years. The additional cost of installing impact resistant roofing instead of conventional roofing is approximately $75/square installed. For a home requiring 30 squares of shingles, the additional cost for impact resistant roofing is $2,200. Combined with discounts of 15% to 30% on insurance premiums for using impact resistant roofing, it is easy to see that an impact roof lasting one storm without requiring replacement pays for itself.

Products classified in accordance with UL 2218 have been shown to sustain significantly less damage after being impacted by hailstones with diameters between 1.0 and 2.0 inches. This proposal allows the use of FM 4473, which uses ice balls as an impact medium and allows relative comparisons of impact resistance between rigid roofing materials. In areas of the country where damaging hail is expected within the design life of a roof covering, building codes should mandate that such impact resistant roofing systems be used.

This proposal also requires that all existing roofing coverings be removed prior to installing new asphalt shingles in hail prone areas. The stiffness plays an important role in hail resistance. Too much flexibility in the system reduces the effectiveness of the of the system's resistance. Recovering over an existing roof system significantly reduces the impact resistance of the roof. Hailstones impacting a roof with two or more layers of asphalt shingles results in a "sponge" effect with the top layer being more susceptible to penetration by the hailstone, thus increasing the potential for water penetration under the roof covering. This sponge effect was observed and reported by the Roofing Industry Committee on Weather Issues (RICOWI) in its Hailstorm Investigation Report. The report confirmed that roofs with asphalt shingles overlaid over other roof coverings experienced damage at smaller size hail than roofs on solid decks.

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

## PART II – IRC B/E Withdrawn by Proponent

# S8-07/08
## 1504.3, Chapter 35 (New)

*Proposed Change as Submitted:*

**Proponent:** Mike Ennis, SPRI, Inc.

**1. Revise as follows:**

**1504.3 (Supp) Wind resistance of nonballasted roofs.** Roof coverings installed on roofs in accordance with Section 1507 that are mechanically attached or adhered to the roof deck shall be designed to resist the design wind load pressures for components and cladding in accordance with Section 1609 <u>and shall be installed in accordance with ANSI/SPRI WD-1</u>.

**2. Add standard to Chapter 35 as follows:**

**Single-Ply Roofing Institute**
<u>ANSI/SPRI WD-1-07</u> <u>Wind Design Standard Practice for Roofing Assemblies</u>

**Reason:** ANSI-SPRI WD-1 Wind Design Standard Practice for Roofing Assemblies is based on calculations contained in ASCE-7, which is the standard used in Chapter 16 to calculate wind load pressures for cladding. The ANSI/SPRI standard uses these calculations and develops tables that can be used to determine the wind uplift pressure on the roof system based on location and building characteristics. The standard also provides design requirements for corner and perimeter areas of the roof that are subject to higher wind pressures. This standard is in the final stages of completing the ANSI Consensus standard process.

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

**Analysis:** Review of proposed new standard SPRI WD-1-07 indicated that, in the opinion of ICC Staff, the standard did not comply with ICC standards criteria, Section 3.6.2.1.

**Committee Action:** **Disapproved**

**Committee Reason:** There is no verification available that shows the testing is accurate for the extrapolation method of the proposed reference standard. The standard also needs to be readily available.

**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 (SPRI), requests Approval as Submitted.**

**Commenter's Reason:** This Code Change was proposed to include ANSI-SPRI WD-1, a national consensus standard entitled "Wind Design Standard Practice for Roofing Assemblies", in Section 1504.3. The wind load calculations in the standard are based on calculations contained in ASCE-7. The standard provides design requirements for corner and perimeter areas of the roof that are subject to higher wind load pressures.

Staff review of the Standard revealed one area in which permissive language was used. This language has been revised to a mandatory form and the standard is being re-canvassed through ANSI. It is expected that the re-canvassing will be complete prior to the September Final Action Hearings. The specific permissive language and edits are summarized below.

The last paragraph in Section 3.2 previously read:

> In some instances, the factored tested load capacity will exceed the field design load, but will be less than the perimeter and/or corner design loads. When this occurs, it may be permissible, as described below, to use an extrapolation method to meet the perimeter and corner design loads. The extrapolation process shall be completed as follows.

This has been changed to read:

> In some instances, the factored tested load capacity will exceed the field design load, but will be less than the perimeter and/or corner design loads. When this occurs, ~~it may be permissible, as described below, to~~ <u>the</u> use <u>of</u> an extrapolation method to meet the perimeter and corner design loads <u>is allowed subject to the limitations defined under "Extrapolation Method - Adhered System Assemblies" or "Extrapolation Method – Mechanically Fastened System Assemblies" depending upon the system being installed.</u> The extrapolation process shall be completed as follows.

Final Action: AS AM AMPC____ D

# S10-07/08

## 1504.4, 1504.4.1 (New), 1504.4.2 (New), 1504.4.3 (New), 1504.4.4 (New)

*Proposed Change as Submitted:*

**Proponent:** Mike Ennis, SPRI, Inc.

**1. Revise as follows:**

**1504.4 (Supp) 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 ~~Section 1504.8 and~~ ANSI/SPRI RP-4.

**2. Add new text as follows:**

**1504.4.1 Maximum building height.** When the maximum building height exceeds 150 feet, the roof design shall be based on an approved design method.

**1504.4.2 Maximum wind speed.** When the maximum wind speed exceeds 140 miles per hour, the roof design shall be based on an approved design method.

**1504.4.3 Wind borne debris regions.** In areas designated as wind borne debris regions, as defined in Section 1609.2, ballast designs using stone ballast shall use a minimum nominal stone diameter of 2 1/2 inches.

**1504.4.4 Use of stone ballast.** In hurricane-prone regions, as defined in Section 1609.2, stone ballast shall not be permitted at roof corners and perimeters as defined in ANSI/SPRI RP-4 in buildings exceeding 60 feet in height, unless the parapet height is greater than 36-inches at such locations.

Reason: **This code change proposal deletes the reference to Section 1504.8 from Section 1504.4, and adds new Sections 1504.4.1 to 1504.4.4 to highlight some of the restrictions to the use of ballasted roof systems that exist in ANSI/SPRI RP-4. The addition of these Sections will make it easier for the Code Official to enforce the requirements of this Section of the Code.**

A requirement to meet the provisions of Section 1504.8 was added to Section 1504.4 Ballasted low-slope roof systems. The reason provided for this addition was a concern about stone ballast blowing off of stone ballasted low-slope roofing systems. However no evidence was provided that this is an issue with ballasted systems designed in accordance with ANSI/SPRI RP-4 a National Consensus Standard that is already referenced in the Code**.** Section 1504.8 is redundant, conflicts with the National Consensus Standard and creates confusion for the code official and design professional.

**The ANSI/SPRI RP-4 Standard is based on extensive wind tunnel testing along with over 30-years of field experience. The Standard is designed to keep the roofing system in place and to prevent ballast stone blow-off when the roofing assembly is exposed to wind loads that are calculated for the specific building conditions. Please see the following references for additional information on studies that have been completed to support the use of ballasted roofing systems:**

www.spri.org/pdf/DesignofRooftopsAgainstGravelBlowOff.pdf
www.spri.org/pdf/WindDesignGuideBallastedRoofingSystemsSympofRoofTech.pdf
www.spri.org/pdf/LooseLaidRoofingSystemsforFlatRoofsSympofRoofTech.pdf
www.spri.org/pdf/WindResistanceofTwoLooseLaidRoofSystems.pdf
www.spri.org/pdf/FurtherWindTunnelTestsofLooseLaidRoofingSystems.pdf
www.spri.org/pdf/HugoEvalofWindPerformanceandWindDesignGuidelinesforAggregateBallastedSinglePlyMembrane.pdf
www.spri.org/pdf/ProceedingsofBallastedSinglePlySystemWindDesignConf1984.pdf
**www.spri.org/pdf/HurricanesCharleyandIvanWindInvestigationReportRICOWI.pdf**

Section 1504.8 is not based on a theoretical calculation and takes into account only the basic wind speed, building height and exposure category. It does not account for the important characteristics of ballast stone size, ballast load weight and building parapet height. Because of these deficiencies it unnecessarily restricts the use of these cost effective and time proven roofing systems. For example, this Section prohibits the use of these systems in hurricane prone regions even though, if properly designed, they have been used effectively in these regions for many years. Section 1504.8 also limits the use of ballasted systems in 90 mile per hour design wind zones to a maximum building height of 35 feet for exposure Category C. This represents the vast majority of the country. Again no examples were provided to substantiate the need for this restriction.

**Cost Impact:** The code change proposal will not increase the cost of construction. It will allow the use of a very cost effective roofing assembly in additional areas of the country and on a greater number of buildings.

### Committee Action: Disapproved

**Committee Reason:** Based on the varying sizes of stone and the excessive amounts of stone that can be there below the nominal size that is specified. In addition the removal of the reference to Section 1504.8 is a concern, since Table 1504.8 was added to the code to go beyond the standard. It would be wrong to remove it because of an apparent conflict. Referencing SPRI RP-4 solely, could create a conflict by referring to an outdated edition of ASCE 7. Also regarding the use of the "approved design method", it's not clear that Building Officials would want to approve such designs when they're brought to them.

### 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 (SPRI), requests Approval as Modified by this Public Comment.

**Replace proposal as follows:**

**1504.4 (Supp) 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 ~~Section 1504.8 and~~ ANSI/SPRI RP-4.

**Commenter's Reason:** The intent of Code Change Proposal S10 is to remove the reference to Section 1504.8 from Section 1504.4 Ballasted low-slope roof systems. The New Text was added in an attempt to provide clarifying language for the Code Official. Based on comments received at the Committee Hearings this language did not achieve the desired outcome. It would be clearer to just remove the reference to Section 1504.8 from Section 1504.4.

In 1988 the requirement for ballasted low-slope roof assemblies to be designed in accordance with ANSI/SPRI RP-4 was first included in the regional building codes. In 2003 Section 1504.8 was added to the code that prohibited the use of gravel surfaced ballasted roofs in hurricane zones, and limited their use to buildings of 35 foot maximum height in 90 mph wind zones under Exposure C conditions. This represents the majority of the United States. Further Section 1504.8 was cross referenced in Section 1504.4 requiring that ballasted roofs not only meet the requirements of ANSI/SPRI RP-4 but also the requirements of Section 1504.8. The justification for the inclusion of these restrictions by the proponents was a concern over wind borne debris. However no data was offered that demonstrated that gravel blow-off had occurred on stone ballasted roofs designed in accordance with ANSI/SPRI RP-4

**Section 1504.8 places unnecessary restrictions on the use of stone ballasted roof assemblies and greatly restricts the use of a cost effective roofing option that has been successfully used by designers and building owners for over 40 years. In fact SPRI members have supplied over 6 billion square feet of ballasted roofing applications over the last two decades. These systems have performed extremely well as evidenced by manufacturers inspections as well as third party reports in which the performance of ballasted roof systems exposed to hurricane conditions was documented by the Roofing Industry Committee on Weather Issues (RICOWI) and the Federal Emergency Management Agency (FEMA).**

One of the design criteria of ANSI/SPRI RP-4 is to prevent gravel blow-off. The technical basis for this standard is a series of wind tunnel tests conducted at the National Research Council Canada. In total 221 separate wind tunnel test were conducted evaluating 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:

$U_{c1}$ – the wind speed at which one or more stones were first observed to move an appreciable distance (i.e. several inches)
$U_{c2}$ – the wind speed above which scouring of stones would continue more or less indefinitely as long as the wind speed is maintained.
$U_{c3}$ – 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)
$U_{c4}$ – 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

In addition to the extensive wind tunnel test program, observed field performance was also a basis for the requirements included in ANSI/SPRI RP-4.

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. <u>Section 1504.8 does not even consider these critical factors</u>. A brief summary of the wind tunnel test program, and reports written as part of this program follows.

LTR-LA-142 Estimation of Critical Wind Speeds for Scouring of Gravel or Crushed Stone on Rooftops January 1974
<u>Objectives:</u>

- 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.

<u>Conclusions:</u>

- 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
<u>Objectives:</u>

- 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.

Final Action: AS AM AMPC____ D

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# S11-07/08
## 1504.5 (New)

*Proposed Change as Submitted:*

**Proponent:** Mark S. Graham, National Roofing Contractors Association, representing Technical Operations Committee of the National Roofing Contractors Association

**Add new text as follows:**

<u>**1504.5 Roof gardens and landscaped roofs.** The wind resistance of roof gardens and landscaped roofs shall be determined based upon the wind resistance of the roof covering and Section 1504.8.</u>

(Renumber subsequent sections)

**Reason:** This proposed code change is intended to provide a means for roof gardens and landscaped roofs to comply with the Code's requirements for wind resistance. The wind resistance of roof coverings used in roof gardens and landscaped roofs is already defined by Sec. 1504.3 and Sec. 1504.4. The inclusion of the requirements in Sec. 1504.8 is intended to address the potential for roof covering surfacings (aggregate, growth media, plants) to become windborne debris during high winds.

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

**Committee Action: Disapproved**

**Committee Reason:** The proposed section on landscaped roofs is poorly worded. A direct reference to Section 1504.1 should be provided. This is also consistent with the committee's disapproval of S3-07/08.

**Assembly Action: None**

*Individual Consideration Agenda*

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

*Public Comment:*

**Mark S Graham, National Roofing Contractors Association (NRCA), representing Technical Operations Committee of the National Roofing Contractors Association, requests Approval as Modified by this Public Comment.**

**Replace proposal as follows:**

<u>**1504.5 Roof gardens and landscaped roofs.** Roof gardens and landscaped roofs shall be designed in accordance with Sec. 1504.3 and Sec. 1504.8</u>

**Commenter's Reason:** This proposed code change and public comments is intended to provide a means for roof gardens and landscaped roofs to comply with the Code's requirements for wind resistance. The added text provides direct reference to Sec. 1504.3 for testing the roof covering's wind resistance and to Sec. 1504.8 for addressing the potential for the roof's surfacing (aggregate, growth media, plants) to become windborne debris during high winds.

Final Action: AS AM AMPC____ D

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# S12-07/08
## 1504.8

*Proposed Change as Submitted:*

**Proponent:** Edwin Huston, National Council of Structural Engineers Association (NCSEA), representing NCSEA Code Advisory Committee – General Engineering Subcommittee

**Revise as follows:**

**1504.8 (Supp) Aggregate.** Aggregate <u>used as surfacing for roof coverings and aggregate, gravel or stone used as ballast</u> shall not be used on the roof of a building located in a hurricane-prone region as defined in Section 1609.2, or on any other building with a mean roof height exceeding that permitted by Table 1504.8 based on the exposure category and basic wind speed at the site.

**Reason:** When Section 1504.8 was added to the IBC by code change S1-03/04, it was the intent of the change that small gravel or aggregate used as surfacing on built-up roofs be prohibited. It was also the intent that larger gravel or stones used as ballast to hold down single–ply membrane roof coverings also be prohibited. With the addition of the definitions of "aggregate" and "ballast" by code change FS185-06/07 and FS186/06/07, respectively, it is necessary to modify Section 1504.8 to clarify that aggregate is prohibited as well as larger stones used as ballast.

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

**Committee Action: Disapproved**

**Committee Reason:** The current wording already restricts the use of aggregate on roofs in hurricane-prone regions, regardless of what the aggregate is used for. It appears that the proposed clarifying verbiage may allow something to fall through a crack.

**Assembly Action: Approved as Submitted**

*Individual Consideration Agenda*

**This item is on the agenda for individual consideration because an assembly action was successful and a public comment was submitted.**

*Public Comment:*

**Edwin Huston, National Council of Structural Engineers Association (NCSEA), representing NCSEA Code Advisory Committee – General Engineering Subcommittee, requests Approval as Submitted.**

**Commenter's Reason:** Five Code Change Proposals were heard by the Structural Committee of the ICC in Palm Springs in February, 2008 that are interrelated.

In S1-07/08, the Single Ply Roofing Industry tried to tie the definition of "Aggregate" in Section 1502.1 to ASTM D 1863. This effort was defeated 12 to 1.

In S10-07/08, the Single Ply Roofing Industry tried to rewrite Section1504.4 of the IBC and in the process, delete Table 1504.8. The Structural Committee voted this down by a vote of 11-1.

In S13-07/08, the Single Ply Roofing Industry tried to delete Table 1504.8 which limits the maximum allowable mean roof height permitted for buildings with aggregate on the roof in areas outside a hurricane-prone region. This proposal would have lessened restrictions on use of aggregate on roof in high wind areas, and only restricted its use to Wind-Borne Debris Regions. This proposal was defeated by a vote of 12 to 1.

In S14-07/08, the Asphalt Roofing Manufacturers Association tried to tie the definition of "Aggregate" in Section 1502.1 to ASTM D 1863 and delete Table 1504.8 which limits the maximum allowable mean roof height permitted for buildings with aggregate on the roof in areas outside a hurricane-prone region. This proposal would have required that "wind-borne debris regions, a minimum of 50 percent of the total aggregate shall be embedded in the flood coat of bitumen". This proposal would have lessened restrictions on use of aggregate on roof in high wind areas, and only restricted its use to Wind-Borne Debris Regions. The Structural Committee voted this down by a vote of 12-1.

NCSEA opposed all four of these changes and was the proponent of S12-07/08. In S12-07/08 We defined aggregate as: "Aggregate <u>used as surfacing for roof coverings and aggregate, gravel or stone used as ballast</u> shall not be used on the roof of a building located in a hurricane-prone region as defined in Section 1609.2, or on any other building with a mean roof height exceeding that permitted by Table 1504.8 based on the exposure category and basic wind speed at the site."

The Structural Committee disapproved our action by a vote of 7-6. NCSEA sought an Assembly Action for "As Submitted" and was successful.

There is significant damage to adjacent structures in high wind storms due to wind-borne debris. Glazing is often damaged by aggregate which is blown off of adjacent or nearby roofs. The NIST report on Hurricanes Katrina and Rita states that, "In New Orleans, the ARA maps suggest that open-country equivalent sustained wind speeds reached as high as 90 mph". (NIST Technical Note 1476: Performance of Physical Structures in Hurricane Katrina and Hurricane Rita: A Reconnaissance Report June 2006). New Orleans was on the left side of the eye of Hurricane Katrina, and is sufficiently inland that the wind speeds decayed from those of the coast. Thus the reported 90 mph wind speeds were at levels that can be experienced by the majority of the United States. There was significant damage to high-rise buildings in downtown New Orleans and the NiST Reconnaissance Report tied that damage to aggregate from building rooftops.

Figure One shows the Hyatt Hotel in downtown New Orleans which was directly downwind from the Amaco Building on Poydrus Avenue. The Amaco Building has aggregate on its roof. Many other such photographs are available. Unlike other hotels in downtown New Orleans, the Hyatt hotel has never reopened after the damage it suffered in Hurricane Katrina.

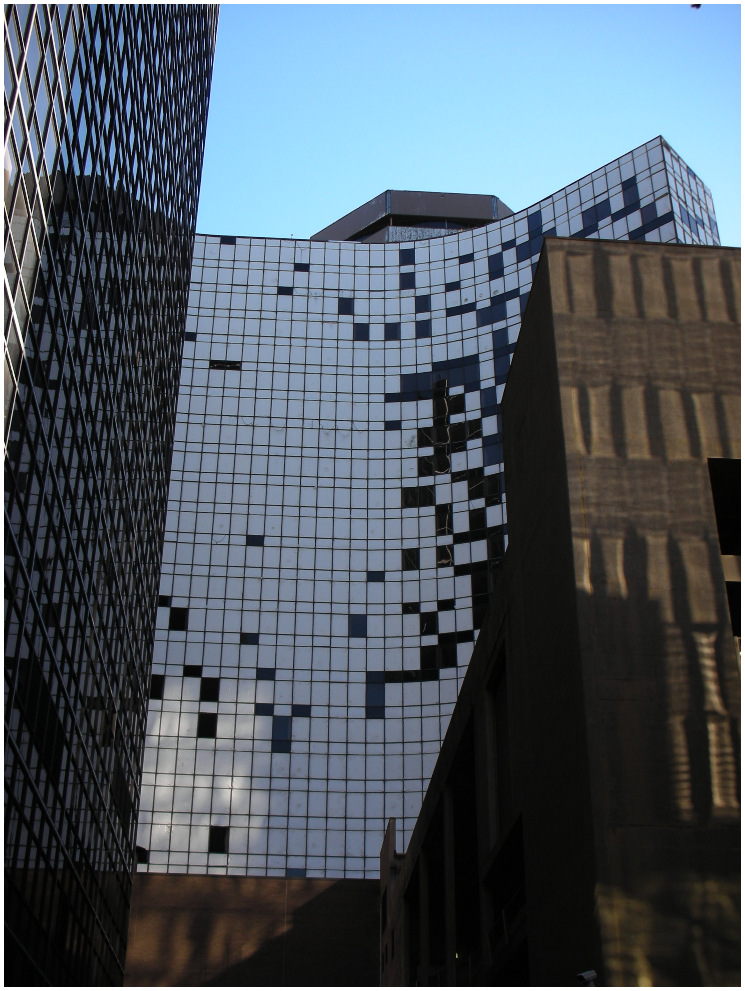

Figure One

ASTM D 1863-05 clearly indicates that aggregate is a surfacing material for built-up roofs.

Section 3.3 of ANSI/SPRI RP 4 – 2002 lists the Ballast Requirements for Ballasted Single-Ply Roofing Systems. A portion of Section 3 of this Standard is shown at right. Section 3.3.1 and Section 3.3.2 reference ASTM D-448, "Standard Sizes of Coarse Aggregate". Other ballast options, such as lightweight pavers are also listed.

Table 1 from ASTM D-448 is shown below. It clearly calls the ballast material "Aggregate" and lists the percentages by size that can pass various size screens. For the permitted sizes in Sections 3.3.1 or 3.3.2 of ANSI/SPRI RP 4 – 2002, Table 1 from ASTM D-448 allows the smaller gradations of aggregate to have 5% to 10% of small stones passing a ¾" screen or a ½" screen.

Thus "Aggregate used as surfacing for roof coverings" is permitted by D 1863 and "aggregate, gravel or stone used as ballast" is permitted by ANSI/SPRI RP 4 – 2002 directly and through the reference to Table 1 from ASTM D-448.

The definition of Aggregate in Section 1504.8 needs to apply to either roofing method, since they both clearly rely on aggregate.

The vast majority of the country is in a 90 mph, Exposure B wind zone. Exposure B is an urban area. In such areas, aggregate is only prohibited in the roof systems of buildings that are 110 feet or taller. If the building were in an open or rural area, it would be in Exposure C. In a 90 mph, Exposure C wind zone, aggregate is prohibited in the roof systems of buildings that are 35 feet or taller. This is hardly a blanket prohibition of aggregate surfaced or aggregate ballasted roofs. In higher wind speed or higher exposure areas, the allowable building height decreases. In lower wind speed or lower exposure areas, the allowable building height increases.

NCSEA does not have an interest in promoting any roofing product over any other roofing product, other than in the interest of public safety. NCSEA's efforts are to keep aggregate from becoming wind-borne debris by prohibiting it from use on roof tops in Wind-Borne Debris Regions and on roof tops of buildings in non-Wind-Borne Debris Regions where the wind speed, the wind exposure and the building height combine to create the same wind uplift force that would be present in a Wind-Borne Debris Region.

The Structural Committee clearly disapproved the use of aggregate in these limited cases by disapproving S1, S10, S13 and S14 by overwhelming majorities. They almost approved S12 "As Submitted". The Assembly did approve S12 as submitted. NCSEA urges you to support the Assembly Action on S12-07/08 and provide a definition of aggregate that keeps it off either type of roof in these limited situations. Such an action will reduce damage in future high wind events.

Final Action: AS AM AMPC____ D

# S14-07/08
## 1502.1, 1504.8, Chapter 35 (New)

*Proposed Change as Submitted:*

**Proponent:** Lorraine Ross, Intech Consulting Inc., representing the Asphalt Roofing Manufacturers Association

**1. Revise definitions as follows:**

**SECTION 1502**
**DEFINITIONS**

**(Supp) AGGREGATE.** In roofing, crushed stone, crushed slag or water-worn gravel used for surfacing for roof coverings, as defined in ASTM D 1863.

**(Supp) BALLAST.** ~~Ballast is any item having weight that is used to hold or steady an object.~~ In roofing, ballast comes in the form of large stones or paver systems or light-weight interlocking paver systems and is used to provide uplift resistance for roofing systems that are not adhered or mechanically attached to the roof deck.

**2. Delete and substitute as follows:**

~~**1504.8 (Supp) Aggregate.** Aggregate shall not be used on the roof of a building located in a hurricane-prone region as defined in Section 1609.2, or on any other building with a mean roof height exceeding that permitted by Table 1504.8 based on the exposure category and basic wind speed at the site.~~

**1504.8 Aggregate.** In wind-borne debris regions, a minimum of 50 percent of the total aggregate shall be embedded in the flood coat of bitumen. Aggregate shall comply with ASTM D 1863.

**~~TABLE 1504.8~~ (Supp)**
**~~MAXIMUM ALLOWABLE MEAN ROOF HEIGHT PERMITTED FOR BUILDINGS WITH AGGREGATE ON THE ROOF IN AREAS OUTSIDE A HURRICANE-PRONE REGION~~**

| ~~BASIC WIND SPEED FROMFIGURE1609 (mph)~~[b] | ~~MAXIMUM MEAN ROOF HEIGHT (ft)~~[a,c] | | |
|---|---|---|---|
| | ~~Exposure category~~ | | |
| | ~~B~~ | ~~C~~ | ~~D~~ |
| | | | |
| ~~85~~ | ~~170~~ | ~~60~~ | ~~30~~ |
| ~~90~~ | ~~110~~ | ~~35~~ | ~~15~~ |
| ~~95~~ | ~~75~~ | ~~20~~ | ~~NP~~ |
| ~~100~~ | ~~55~~ | ~~15~~ | ~~NP~~ |
| ~~105~~ | ~~40~~ | ~~NP~~ | ~~NP~~ |
| ~~110~~ | ~~30~~ | ~~NP~~ | ~~NP~~ |
| ~~115~~ | ~~20~~ | ~~NP~~ | ~~NP~~ |
| ~~120~~ | ~~15~~ | ~~NP~~ | ~~NP~~ |
| ~~Greater than 120~~ | ~~NP~~ | ~~NP~~ | ~~NP~~ |

~~For SI: 1 foot = 304.8 mm; 1 mile per hour = 0.447 m/s.~~
~~a. Mean roof height as defined in ASCE 7.~~
~~b. For intermediate values of basic wind speed, the height associated with the next higher value of wind speed shall be used, or direct interpolation is permitted.~~
~~c. NP = gravel and stone not permitted for any roof height.~~

**3. Add standard to Chapter 35 as follows:**

**ASTM**
D 1863-05 Standard Specification for Mineral Aggregate Used on Built-Up Roofs

**Reason:** This code change proposal clarifies the definition of aggregate, tying it into a current IBC reference standard: ASTM D1863—03 Specification for Mineral Aggregate Used on Built-up Roofs. This code change also delineates an safe appropriate use of aggregate roofs in wind borne debris regions as defined in IBC Chapter 1609.2.

The proposed language is taken from the 2004 Florida Building Code, High Velocity Hurricane Region where the ASCE 7 referenced wind Zone is 146 – 150 mph (3 sec. gust) and Exposure Category C. This area has recognized the advantages of built-up roofs, in terms of durability and fire test performance, and has developed requirements that allow its safe use. The last 15 years has proven the effectiveness of these requirements. There is no limitation on building height in this area and so the entire table has been deleted.

The current code is overly restrictive in it essentially bans the use of aggregate roofs (built-up roofs) in a major part of the US. However, these roofs have been used successfully for over a century in these wind zones and building heights. The severe limitation on the use of aggregate on built-up roofs shown in the current (2006 and 2007 Supplement) IBC was based solely on a probability calculation and had no empirical evidence. While there is, of course, concern with gravel blow-off in high wind conditions, actual experience shows that the requirements adopted by the Florida Miami-Dade County in its South Florida Building Code since Hurricane Andrew in 1992 and subsequent high wind events has proven to be effective in the use of this highly versatile roofing system.

The proposed requirements for aggregate roof systems for the wind borne debris regions is based on findings of the ASCE 7 Committee reflected in a distinction between Hurricane prone regions and the Wind Borne Debris Regions as illustrated in IBC Section 1609.2 as follows:

**HURRICANE-PRONE REGIONS.** Areas vulnerable to hurricanes defined as:

1. The U. S. Atlantic Ocean and Gulf of Mexico coasts where the basic wind speed is greater than 90 mph (40 m/s) and
2. Hawaii, Puerto Rico, Guam, Virgin Islands and American Samoa.

**WIND-BORNE DEBRIS REGION.** Portions of hurricane-prone regions that are within 1 mile (1.61 km) of the coastal mean high water line where the basic wind speed is 110 mph (48 m/s) or greater; or portions of hurricane-prone regions where the basic wind speed is 120 mph (53 m/s or greater; or Hawaii.

The concern about roof aggregate blow-off, if any, should be in the wind borne debris regions.

This code requirement has been used in the Miami-Dade county area for over 15 years and has a proven track record of success.

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

**Analysis:** Review of proposed new standard ASTM D 1863 indicated that, in the opinion of ICC Staff, the standard did comply with ICC standards criteria.

## Committee Action: Disapproved

**Committee Reason:** It is unclear how a building official can determine that the required 50 percent of the total aggregate is embedded in the flood coat. Furthermore regardless of the actual embedded percentage, there would seem to be a problem posed by the percentage of aggregate that is not embedded.

## Assembly Action: None

# *Individual Consideration Agenda*

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

*Public Comment:*

## Lorraine Ross, Intech Consulting Inc, representing Asphalt Roofing Manufacturers Association, requests Approval as Modified by this Public Comment.

**Modify proposal as follows:**

**(Supp) BALLAST.** Ballast is any item having weight that is used to hold or steady an object. In roofing, ballast comes in the form of large stones or paver systems or light-weight interlocking paver systems and is used to provide uplift resistance for roofing systems that are not adhered or mechanically attached to the roof deck.

**1504.8 Aggregate**. In wind-borne debris regions, ~~a minimum of 50 percent of the total aggregate shall be embedded in the flood coat of bitumen. Aggregate shall comply with ASTM D 1863~~ aggregate surfacing shall be permitted when installed on slopes of 3:12 or less, not less than 400 pound (182 kg) of roofing gravel or 300 pounds (145 kg) of slag per square shall be applied A minimum of 50 percent of the total aggregate shall be embedded in the flood coat of bitumen. Aggregate shall be dry and free from dirt and shall be in compliance with the sizing requirements set forth in ASTM D 1863.

(Portions of proposal not shown remain unchanged)

**Commenter's Reason:** Most roofing experts would agree that stones and gravel may become airborne during a high wind event. The same can be said of tile or asphalt shingles or any other type of roofing material. In fact, the Hurricane Charley investigation showed a great deal of damage caused by flying roofing tile. Yet tile was not banned in Florida or anywhere else in the hurricane prone regions. Measures were taken to improve the installation of tile products. This, in ARMA's opinion, was an appropriate response. Banning gravel and stone roofs is unwarranted. This modification is based on actual field results in the 150 mph wind zone in the State of Florida. This method of installation has proven to be successful.

A detailed risk analysis that examines the frequency of the high wind events, along with the probability of stones flying off roofs and the probability that those stones would cause damage has not been calculated. The decision by the IBC to ban stone and gravel roofs is based on a zero risk – an assumption that high winds always occur, stones always fly off buildings and always cause damage. No such data exists.

If this type of analysis is taken to every other building product or system, there will be little choices left for the consumer and building owner.

There is also an emerging environmental impact of eliminating stone and gravel roofs. It appears that a great deal of endangered nesting waterbirds have been using gravel roofs near the Atlantic and Gulf coasts because of loss of habitat due to beach development. However, the two articles from Dr. Forys at Eckerd College, along with work conducted by the Audubon Society shows a clear preference for gravel roofs by least terns because of the loss of habitat due to beachfront development. For example, in a paper published in Waterbirds 29(4): 501-506, 2006, Roof-top Selection by Least Terns in Pinellas County, Florida

ELIZABETH A. FORYS AND MONIQUE BORBOEN-ABRAMS
Natural Sciences Collegium,
Eckerd College 4200 54th Ave. South, St. Petersburg, FL 33711
Internet: forysea@eckerd.edu

St. Petersburg Audubon Society, St. Petersburg, FL 33701

" The first report of roof-nesting Least Terns came from Pensacola, Florida in 1957 (Fisk 1975). As many as 80% of Least Tern colonies in Florida are now found on roofs, where colonies can consist of fewer than five breeding pairs or contain as many as several hundred pairs (Burger 1988). A study in northwestern Florida found that rooftop colonies produce as many chicks and fledged young as beach colonies (Gore and Kinnison 1991). Until recently, Least Tern populations in the Southeastern United States were stable and possibly increasing due to the abundance of rooftop habitat (Gore 1996)." The full paper, along with others, will be made available in advance of the ICC Final Action Hearing.

Final Action: AS AM AMPC____ D

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# S15-07/08, Part II
## IRC R202

**THIS CODE CHANGE WILL BE HEARD ON THE IRC BUILDING PORTION OF THE HEARING ORDER.**

***NOTE: PART I DID NOT RECEIVE A PUBLIC COMMENT AND IS ON THE CONSENT AGENDA. PART I IS REPRODUCED FOR INFORMATIONAL PURPOSES ONLY FOLLOWING ALL OF PART I.***

*Proposed Change as Submitted:*

**PART II – IRC BUILDING/ENERGY**

**Proponent:** Philip Brazil, Reid Middleton, Inc., representing himself

**Revise as follows:**

**SECTION R202**
**DEFINITIONS**

**MEAN ROOF HEIGHT.** The vertical distance from grade plane to the average of the roof eave height and the height ~~to~~ of the highest point on the roof surface, except that, ~~eave height shall be used for roof angle of less than or equal to~~ for roof angles no greater than 10 degrees (0.18 rad), the mean roof height shall be the vertical distance from grade plane to the roof eave height.

**Reason:** There are several locations in the IBC where mean roof height is specified. Refer to Sections 1504.8, 1609.1.2 and 1609.4.3; and Tables 1504.8, 1507.3.7, 1609.1.2 and 2308.10.1. The IBC, however, does not define it. A definition was in 2003 IBC Section 1609.1.2 but it was deleted by Proposal S32-04/05-AM. This proposal restores the definition. The proposed definition is similar to the definition in Section 6.2 of ASCE 7-05 except it corrects the inadvertent omission in that definition of specifying what the mean roof height is measured from, which is grade plane in the proposed definition.

"Grade plane" was chosen for the definition over "grade" because of approved Proposal G44-04/05-AM, which successfully established the distinction between "grade plane" as a measurement of the height and number of stories of a building above the finished ground surface and "grade" as a measurement of the height of a component of the building above the finished ground surface. Grade plane is an imaginary horizontal reference plane representing the weighted average of the finished ground surface adjoining the building at its perimeter. The grade plane of each building is located at a single, unique elevation. Grade, however, is not imaginary but is the actual finished ground surface adjoining the building at its perimeter, which varies in elevation with the ground surface. Note that, in each case where "mean roof height" is specified in the IBC, the application is to a building or structure, not a component of a building or structure.

Footnote (a) of Table 1504.8 is deleted in coordination with the proposed definition.

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

**PART II – IRC B/E**
**Committee Action:** **Approved as Submitted**

**Committee Reason:** This change clarifies the code in regard to the measurement of mean roof height. Currently there is no reference in the code specific where to measure the mean roof height from. This change provides the reference.

**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 Committee – General Engineering Subcommittee, requests Disapproval.**

**Commenter's Reason:** The proposal modifies the definition of mean roof height, which is used with the wind design provisions of the IRC. The existing definition is taken from and is consistent with the same definition in ASCE 7. Part I of the proposal sought to add the definition of mean roof height to the IBC, but modified it from what is in ASCE 7. Part I was disapproved. If Part II is approved, there will be an inconsistency between the IRC, and IBC and ASCE 7.

The proponent has submitted a change to ASCE 7 to modify the definition of mean roof height in ASCE 7 based on the modifications in this code change. The change has been considered by the ASCE 7 Wind Subcommittee and rejected because it could result in unconservative application of the wind design provisions. It is the intent of ASCE 7 (see definition of "eave height") that the height be measured from the ground surface adjacent to the building on the side of the building being considered. In this proposal to the IBC and IRC, the height is measured from the grade plane, which on a hilly site can be a story or more above the ground surface on the downhill side of a building. By measuring from the grade plane instead of the ground surface, the height is reduced which reduces the velocity pressure exposure coefficient, which results in a reduction of the design wind pressure.

For the reasons cited, you are urged to vote against the motion to approve the proposal, and subsequently vote to disapprove the change.

Final Action: AS AM AMPC____ D

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***NOTE: PART I REPRODUCED FOR INFORMATIONAL PURPOSES ONLY – SEE ABOVE***

## S15–07/08, PART I – IBC STRUCTURAL

**TABLE 1504.8**
**MAXIMUM ALLOWABLE MEAN ROOF HEIGHT PERMITTED FOR BUILDINGS**
**WITH GRAVEL OR STONE ON THE ROOF IN AREAS OUTSIDE A HURRICANE-PRONE REGION**

| BASIC WIND SPEED FROM FIGURE 1609 (mph) [~~b~~ a] | MAXIMUM MEAN ROOF HEIGHT (ft) [~~a, c~~ b] | | |
|---|---|---|---|
| | Exposure Category | | |
| | B | C | D |
| 85 | 170 | 60 | 30 |
| 90 | 110 | 35 | 15 |
| 95 | 75 | 20 | NP |
| 100 | 55 | 15 | NP |
| 105 | 40 | NP | NP |
| 110 | 30 | NP | NP |
| 115 | 20 | NP | NP |
| 120 | 15 | NP | NP |
| Greater than 120 | NP | NP | NP |

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

~~a. Mean roof height as defined in ASCE 7.~~
a. ~~b.~~ For intermediate values of basic wind speed, the height associated with the next higher value of wind speed shall be used, or direct interpolation is permitted.
b. ~~c.~~ NP = gravel and stone not permitted for any roof height.

**Reason:** Same as Part I.

**PART I – IBC STRUCTURAL**
**Committee Action:** **Disapproved**

**Committee Reason:** See related code change proposal S82-07/08.

**Assembly Action:** **None**

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# S22-07/08
## 1505.7 (New)

**THIS CODE CHANGE WILL BE HEARD ON THE IBC FIRE SAFETY PORTION OF THE HEARING ORDER.**

*Proposed Change as Submitted:*

**Proponent:** Mark S. Graham, National Roofing Contractors Association, representing Technical Operations Committee of the National Roofing Contractors Association

**Add new text as follows:**

**1505.7 Roof gardens and landscaped roofs.** The fire resistance of roof gardens and landscaped roofs shall be determined based upon the fire resistance of the roof covering.

(Renumber subsequent sections)

**Reason:** This proposed code change is intended to provide a means for roof gardens and landscaped roofs to comply with the Code's requirements for external fire resistance. The fire resistance roof coverings, such as those used in roof gardens and landscaped roofs, is already defined in Sec. 15005—Fire Classification.

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

**This code change was heard by the IBC Fire Safety Code Development Committee.**

**Committee Action: Disapproved**

**Committee Reason:** Based on incorrect terminology (fire resistance should be fire classification) and the lack of testing criteria addressing roof gardens and landscaped roofs.

**Assembly Action: None**

*Individual Consideration Agenda*

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

*Public Comment:*

**Mark S. Graham, National Roofing Contractors Association (NRCA), Technical Operations Committee of the National Roofing Contractors Association, requests Approval as Modified by this Public Comment.**

**Replace proposal as follows:**

**1505.7 Roof gardens and landscaped roofs**. The fire classification of roof gardens and landscaped roofs shall be based upon the fire classification of the roof covering.

**Commenter's Reason:** The proposed code change and this proposed modification is intended to provide a means for roof gardens and landscaped roofs to comply with the Code's requirements for external fire resistance. This modification takes into consideration suggestions for rewording provided by committee members.

The fire classification of roof coverings, such as those used in roof gardens and landscaped roofs, is already defined in Sec. 1505—Fire Classification.

Final Action: AS AM AMPC____ D

# S35-07/08, Part II
## IRC R905.14.2

**THIS CODE CHANGE WILL BE HEARD ON THE IRC BUILDING PORTION OF THE HEARING ORDER.**

***NOTE: PART I DID NOT RECEIVE A PUBLIC COMMENT AND IS ON THE CONSENT AGENDA. PART I IS REPRODUCED FOR INFORMATIONAL PURPOSES ONLY FOLLOWING ALL OF PART II.***

*Proposed Change as Submitted:*

**PART II – IRC BUILDING/ENERGY**

**Proponent:** Mason Knowles, Mason Knowles Consulting LLC, representing himself

**Revise as follows:**

**R905.14.2 Material standards.** Spray-applied polyurethane foam insulation shall comply with Types III and IV as defined in ASTM C 1029.

**Reason:** This code change corrects a mistake made a few years ago. Originally this code change referenced types III and IV as spray foams acceptable for SPF roofing applications but in the 2004 supplement, the reference to types III an IV were deleted.

ASTM C 1029 classifies spray polyurethane foam into 4 types based on minimum compressive strength. Types I & II compressive strengths are 15 and 25 respectively. Types I and II as defined by ASTM C 1029 are not suitable for spray polyurethane foam roofing systems. Types III and IV compressive strengths are 40 and 60 psi respectively.

ASTM D 5469, Standard Guideline for New Spray Polyurethane Foam Roofing Systems and technical documents SPFA list the minimum compressive strength for spray polyurethane roofing systems as 40 psi which would correspond to types III and IV in ASTM C 1029.

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

**Committee Action:** **Approved as Submitted**

**Committee Reason:** This change provides the correct reference to the types of acceptable spray polyurethane insulation, as well as correcting an omission.

**Assembly Action:** **None**

*Individual Consideration Agenda*

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

*Public Comment:*

**Mark S. Graham, National Roofing Contractors Association (NRCA), Technical Operations Committee of the National Roofing Contractors Association, requests Approval as Modified by this Public Comment.**

**Modify proposal as follows:**

**R905.14.2 Material standards.** Spray-applied polyurethane foam insulation shall comply with Types III ~~and~~ or IV as defined in ASTM C 1029.

**Commenter's Reason:** This modification is intended to make the IRC language consistent with the action taken by the IBC Structural Committee on the same item (Part I) and maintain consistency between the IBC and IRC.

An "or" is appropriate here to permit the use of either Type III or Type IV. Type III provides for SPF with a minimum compressive strength of 40 psi, while Type IV provides for a minimum compressive strength of 60 psi. Both 40 PSI and 60 psi SPF are appropriate for roofing applications.

Final Action: AS AM AMPC____ D

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***NOTE: PART I IS REPRODUCTED FOR INFORMATIONAL PURPOSES ONLY – SEE ABOVE***

***S35-07/08, PART I – IBC STRUCTURAL***

**1507.14.2 Material standards**. Spray-applied polyurethane foam insulation shall comply with Types III and IV as defined in ASTM C 1029.

**Reason**: This code change corrects a mistake made a few years ago. Originally this code change referenced types III and IV as spray foams acceptable for SPF roofing applications but in the 2004 supplement, the reference to types III an IV were deleted.

ASTM C 1029 classifies spray polyurethane foam into 4 types based on minimum compressive strength. Types I & II compressive strengths are 15 and 25 respectively. Types I and II as defined by ASTM C 1029 are not suitable for spray polyurethane foam roofing systems. Types III and IV compressive strengths are 40 and 60 psi respectively.

ASTM D 5469, Standard Guideline for New Spray Polyurethane Foam Roofing Systems and technical documents SPFA list the minimum compressive strength for spray polyurethane roofing systems as 40 psi which would correspond to types III and IV in ASTM C 1029.

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

**PART I – IBC STRUCTURAL**
**Committee Action:** **Approved as Modified**

Modify proposal as follows:

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

**Committee Reason**: This code change establishes product standards. The modification is editorial and makes it clear that the insulation should comply with one or the other rather than both.

**Assembly Action:** **None**

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# S41-07/08
## 1507.16

*Proposed Change as Submitted:*

**Proponent:** Scott Poster, Los Angeles County Fire Department, CA

**Revise as follows:**

**1507.16 (Supp) Roof gardens and landscaped roofs.** Roof gardens and landscaped roofs shall comply with the requirements of this chapter and Sections 1607.11.2.2 and 1607.11.2.3. <u>When over 50 percent of the roof is utilized as such, provision shall be made for firefighting ventilation activities utilizing smoke and heat vents, sawtooth roof construction, mechanical ventilation or other approved means.</u>

**Reason:** Roof gardens and landscaped roofs will hinder vertical firefighting ventilation. Ventilation allows the escape of smoke and heat creating a more tenable situation for interior crews to reach the seat of a fire and extinguish. Rapid ventilation may save the lives of building occupants.

In order to cut holes in the roof, truck firefighters utilize chain saws and rotary saws. In order to traverse the roof to position themselves as near above the fire as possible, truck firefighters utilize sounding tools, such as rubbish hooks, and infrared cameras to monitor structural integrity. Adding layers of waterproofing, building material, soil, and vegetation to the surface of a roof will delay if not preclude ventilation operations.

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

**Committee Action:** **Disapproved**

**Committee Reason:** Section 1507.16 currently provides a cross reference to live load requirements for landscaped roofs. Landscaped roofs are not exempt from smoke and heat vent requirements in Section 910. There is no reason for adding such requirements in this section of 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:*

**Scott Poster, Los Angeles County Fire Department, requests Approval as Modified by this Public Comment.**

**Replace proposal as follows:**

**SECTION 911**
**ROOF GARDENS AND LANDSCAPED ROOFS**

**911.1 Roof garden and landscaped roofs**. When over 50 percent of a building's roof is covered by the installation of roof gardens and roof landscaping, provisions shall be made for firefighting ventilation activities utilizing smoke and heat vents, saw-tooth roof construction, mechanical ventilation, or other approved means.

(Renumber subsequent sections)

**Commenter's Reason:** This public comment is being submitted to address the Hearing Committee's criticism of the original S-41proposal which was denied at the Palm Springs Hearings due to it being placed in a wrong section of the Building Code.

This public comment moves the original S-41 code proposal from Chapter 15, Roof Assemblies and Rooftop Structures, to Chapter 9, Fire Protection Systems, of the Building Code. It also creates a new section in Chapter 9 following the existing smoke and heat venting section. Two existing sections of Chapter 9 will be renumbered.

The purpose of this new section is to make provisions to buildings in which the roof's surface has been significantly obstructed by roof gardens and landscaped roofs. These provisions provide for fire fighter's operational need to rapidly vertically ventilate a building during a fire emergency.

Roof gardens and landscaped roofs hinder or prevent vertical firefighting ventilation efforts. In order to cut holes in the roof and to ventilate the interior space of a building, firefighters utilize chain saws and rotary saws. In order to traverse the roof and to position themselves as near above the fire as possible, firefighters utilize sounding tools, such as rubbish hooks, and infrared cameras to monitor structural integrity of the building. Adding layers of waterproofing, building material, soil, and vegetation to the surface of a roof will delay if not preclude ventilation operations.

Roof top ventilation allows the escape of smoke and heat within a building that can lengthen the time the interior of the building will remain tenable for occupant and firefighters. Firefighting crews could be allowed to conduct rapid search and rescue operations and to more easily find the seat of a fire and extinguish it. Rapid vertical ventilation could be a major factor in saving the lives of building occupants by increasing smoke compartment visibility to aid in exiting, lessen the likelihood of a fatal flashover, and offer firefighters a ventilation tactic of drawing an interior building fire in a particular direction away from life and fire hazards within the building.

Final Action: AS AM AMPC____ D

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# S42-07/08, Part I

## 1502.1, 1509.2, 1509.2.1 through 1509.2.3, 1509.2.4 (New)

**THIS CODE CHANGE WILL BE HEARD ON THE IBC GENERAL PORTION OF THE HEARING ORDER.**

***NOTE: PART II DID NOT RECEIVE A PUBLIC COMMENT AND IS ON THE CONSENT AGENDA. PART II IS REPRODUCED FOR INFORMATIONAL PURPOSES ONLY FOLLOWING ALL OF PART I.***

*Proposed Change as Submitted:*

**PART I – IBC GENERAL**

**Proponent:** Sarah A. Rice, CBO, Schirmer Engineering Corporation

**Revise as follows:**

**SECTION 1502**
**DEFINITIONS**

**PENTHOUSE.** An enclosed, unoccupied structure above the roof of a building, other than a tank, tower, spire, dome cupola or bulkhead~~, occupying not more than one-third of the roof area~~.

**1509.2 Penthouses.** A penthouse or penthouses in compliance with Section 1509.2 shall not be considered as a portion of the story below.

**1509.2.1 Height above roof.** A penthouse or other projection above the roof in structures of other than Type I construction shall not exceed 28 feet (8534 mm) above the roof where used as an enclosure for tanks or for elevators that run to the roof and in all other cases shall not extend more than 18 feet (5486 mm) above the roof.

**1509.2.2 Area limitation.** The aggregate area of penthouses and other rooftop structures shall not exceed one-third the area of the supporting roof. Such penthouses shall not contribute to either the building area or number of stories as regulated by Section 503.1. The area of the penthouse shall not be included in determining the fire area defined in Section 702.

**1509.2.3 Use limitations.** A penthouse, bulkhead or any other similar projection above the roof shall not be used for purposes other than shelter of mechanical equipment or shelter of vertical shaft openings in the roof. Provisions such as louvers, louver blades or flashing shall be made to protect the mechanical equipment and the building interior from the elements. Penthouses or bulkheads used for purposes other than permitted by this section shall conform to the requirements of this code for an additional story. The restrictions of this section shall not prohibit the placing of wood flagpoles or similar structures on the roof of any building.

**Reason:** This proposal seeks to clean up the provisions on penthouses so that among other things:

- code requirements are not in the definition,
- it is clear on what kind of means of egress is to be provided,
- it is clear what types of means of egress must be available.
- A penthouse is similar to a mezzanine in many ways, thus much of the proposed language is similar to what is found in the provisions for mezzanines in Section 505.

The only element that will vary for these tenant spaces is how they contribute to the design of the means of egress system of the covered mall building. All other elements remain the same, e.g., fire alarm, sprinkler, etc.

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

**PART I – IBC GENERAL**
**Committee Action:** **Approved as Modified**

**Modify the proposal as follows:**

**1509.2 Penthouses.** A penthouse or penthouses in compliance with Sections ~~1509.2~~ 1509.2.1 through 1509.2.3 shall ~~not~~ be considered as a portion of the story below.

(Portions of proposal not shown remain unchanged)

**Committee Reason:** Clarifies application of the provisions for penthouses and removes requirements from the definition of penthouse. The modification simply revises the phrase "in compliance with section 1509.2" to "in compliance with Section 1509.2.1 through 1509.2.3" to clarify the applicability of the subsections and be consistent with code style. Additionally the term "not" in section 1509.2 was deleted as it would nullify the revisions the proponent intended.

**Assembly Action:** **None**

*Individual Consideration Agenda*

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

*Public Comment:*

**Sarah A. Rice, C.B.O., Schirmer Engineering Corporation, requests Approval as Modified by this Public Comment.**

**Further modify proposal as follows:**

**1509.2 Penthouses**. A penthouse or penthouses in compliance with Sections 1509.2.1 through 1509.2.3 shall be considered as a portion of the story below. Penthouses shall not contribute to the building area, building height in feet or number of stories as regulated by Section 503.1. The area of the penthouse shall not be included in determining the fire area defined in Section 702.

**1509.2.1 Height above roof.** A penthouse or other projection above the roof in structures of other than Type I construction shall not exceed 28 feet (8534 mm) above the roof where used as an enclosure for tanks or for elevators that run to the roof and in all other cases shall not extend more than 18 feet (5486 mm) above the roof.

**1509.2.2 Area limitation**. The aggregate area of penthouses and other rooftop structures shall not exceed one-third the area of the supporting roof. ~~Such penthouses shall not contribute to either the building area or number of stories as regulated by Section 503.1. The area of the penthouse shall not be included in determining the fire area defined in Section 702.~~

**1509.2.3 Use limitations.** A penthouse, bulkhead or any other similar projection above the roof shall not be used for purposes other than shelter of mechanical equipment or shelter of vertical shaft openings in the roof. Provisions such as louvers, louver blades or flashing shall be made to protect the mechanical equipment and the building interior from the elements. Penthouses or bulkheads used for purposes other than permitted by this section shall conform to the requirements of this code for an additional story. The restrictions of this section shall not prohibit the placing of wood flagpoles or similar structures on the roof of any building.

**Commenter's Reason:** The term "building height in feet" is needed to clarify that a penthouse complying with Section 1509.2 contributes to neither building height in feet nor in stories.

Final Action: AS AM AMPC____ D

***NOTE: PART II IS REPRODUCED FOR INFORMATIONAL PURPOSES ONLY – SEE ABOVE***

**S42-07/08, PART II**

**Add new text as follows:**

<u>**1509.2.4 Egress.** Each occupant of a penthouse shall have access to at least two independent means of egress where the common path of egress travel exceeds the limitations of Section 1014.3. Where a stairway provides a means of exit access from a penthouse, the maximum travel distance includes the distance traveled on the stairway measured in the plane of the tread nosing. Accessible means of egress shall be provided in accordance with Section 1007.</u>

<u>**Exception:** A single means of egress shall be permitted in accordance with Section 1015.1.</u>

(Renumber subsequent sections)

**Reason:** This proposal seeks to clean up the provisions on penthouses so that among other things:
- code requirements are not in the definition,
- it is clear on what kind of means of egress is to be provided,
- it is clear what types of means of egress must be available.
- A penthouse is similar to a mezzanine in many ways, thus much of the proposed language is similar to what is found in the provisions for mezzanines in Section 505.

The only element that will vary for these tenant spaces is how they contribute to the design of the means of egress system of the covered mall building. All other elements remain the same, e.g., fire alarm, sprinkler, etc.

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

**PART II – IBC MEANS OF EGRESS**
**Committee Action:** **Disapproved**

**Committee Reason:** The proposal is too restrictive. The definition for means of egress deals with occupied spaces; therefore, since penthouses are considered unoccupied spaces, this could be a possible conflict. A penthouse typically has a very low or no occupants – the proposal would result in two means of egress being required for a penthouse. In addition, this requirement could result in at least two stairways to extend to the roof.

**Assembly Action:** **None**

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# S43-07/08
## 1509.2.1

**THIS CODE CHANGE WILL BE HEARD ON THE IBC GENERAL PORTION OF THE HEARING ORDER.**

*Proposed Change as Submitted:*

**Proponent:** Joseph T. Holland, III, Hoover Treated Wood Products

**Revise as follows:**

**1509.2.1 (Supp) Type of construction.** Penthouses shall be constructed with walls, floors and roof as required for the building.

**Exceptions:**

1. On buildings of Type I<u>A</u> construction, the exterior walls and roofs of penthouses with a fire separation distance of more than 5 feet (1524 mm) and less than 20 feet (6096 mm) shall be of at least 1-hour fire resistance rated noncombustible construction. Walls and roofs with a fire separation distance of 20 feet (6096 mm) or greater shall be of noncombustible construction. Interior framing and walls shall be of noncombustible construction
2. On buildings of Type I<u>A construction</u> two stories above grade plane or less in height<u>, and Type IB</u> and Type II construction, the exterior walls and roofs of penthouses with a fire separation distance of more than 5 feet (1524 mm) and less than 20 feet (6096 mm) shall be of at least 1-hour fire-resistance-rated noncombustible or fire-retardant-treated wood construction. Walls and roofs with a fire separation distance of 20 feet (6096 mm) or greater shall be of noncombustible or fire-retardant-treated wood construction. Interior framing and walls shall be of noncombustible or fire retardant-treated wood construction.

3. On buildings of Type III, IV and V construction, the exterior walls of penthouses with a fire separation distance of more than 5 feet (1524 mm) and less than 20 feet (6096 mm) shall be at least 1-hour fire resistance- rated construction. Walls with a fire separation distance of 20 feet (6096 mm) or greater from a common property line shall be of Type IV, noncombustible, or fire-retardant-treated wood construction. Roofs shall be constructed of materials and fire-resistance rated as required in Table 601and Section 601 Item 1.3. Interior framing and walls shall be Type IV, noncombustible, or fire-retardant-treated wood construction.
4. On buildings of Type I unprotected noncombustible enclosures housing only mechanical equipment and located with a minimum fire separation distance of 20 feet (6096 mm) shall be permitted.
5. On buildings of Type IA construction two stories or less above grade plane in height, or Type IB, ~~Type~~ II, III, IV, and V construction, unprotected noncombustible or fire-retardant-treated wood enclosures housing only mechanical equipment and located with a minimum fire separation distance of 20 feet (6096 mm) shall be permitted.
6. On one-story buildings, combustible unroofed mechanical equipment screens, fences or similar enclosures are permitted where located with a fire separation distance of at least 20 feet (6096 mm) from adjacent property lines and where not exceeding 4 feet (1219 mm) in height above the roof surface.
7. Dormers shall be of the same type of construction as the roof on which they are placed, or of the exterior walls of the building.

**Reason:** Update section to incorporate provisions adopted last cycle for roof construction on Type I buildings. FS211-06/07 added Type IB construction to the types of construction where FRTW can be used in the roof construction over two stories. This proposal will allow FRTW and for the penthouse on Type IB over two stories to be of the same materials. With the adoption of FS211 the roof can be FRTW it would be inconsistent to not allow roof structures to be of the same material as allowed for the roof construction.

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

**Committee Action:** **Disapproved**

**Committee Reason:** Sufficient justification was not provided to allow the additional use of fire retardant treated wood in Type IB construction for penthouses.

**Assembly Action:** **None**

*Individual Consideration Agenda*

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

*Public Comment:*

**Joseph Holland, Hoover Treated Wood Products, requests Approval as Submitted.**

**Commenter's Reason:** The membership approved a change to allow the roof construction in Type IB construction to be fire-retardant-treated wood. This change is consistent with the previous allowable uses. When the roof can be FRTW the structures on the roof can also be FRTW as in Type IIA and IIB.

Final Action: AS AM AMPC____ D

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# S47-07/08

## 1603.1, 1603.1.6 (New), 1802.6

*Proposed Change as Submitted:*

**Proponent:** Edwin T. Huston, Smith & Huston, Inc., representing National Council of Structural Engineering Associations

**1. Revise as follows:**

**1603.1 General.** Construction documents shall show the size, section and relative locations of structural members with floor levels, column centers and offsets dimensioned. The design loads and other information pertinent to the structural design required by Sections 1603.1.1 through ~~1603.1.8~~ 1603.1.9 shall be indicated on the construction documents.

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

1. Floor and roof live loads.
2. Ground snow load, *Pg*.
3. Basic wind speed (3-second gust), miles per hour (mph) (km/hr) and wind exposure.
4. Seismic design category and site class.
5. Flood design data, if located in flood hazard areas established in Section 1612.3.
6. <u>Design load-bearing values of soils.</u>

**2. Add new text as follows:**

<u>**1603.1.6 Geotechnical information.** The soil classification and design load-bearing values shall be shown on the construction documents.</u>

(Renumber subsequent sections)

**Revise as follows:**

**1802.6 Reports.** ~~The soil classification and design load-bearing capacity shall be shown on the construction documents.~~ Where required by the building official, a written report of the investigation shall be submitted that includes, but need not be limited to, the following information:

1. A plot showing the location of test borings and/or excavations.
2. A complete record of the soil samples.
3. A record of the soil profile.
4. Elevation of the water table, if encountered.
5. Recommendations for foundation type and design criteria, including but not limited to: bearing capacity of natural or compacted soil; provisions to mitigate the effects of expansive soils; mitigation of the effects of liquefaction, differential settlement and varying soil strength; and the effects of adjacent loads.
6. Expected total and differential settlement.
7. Pile and pier foundation information in accordance with Section 1808.2.2.
8. Special design and construction provisions for footings or foundations founded on expansive soils, as necessary.
9. Compacted fill material properties and testing in accordance with Section 1803.5.

**Reason:** Code clarification. Moves the requirement to show “soil classification and design load-bearing capacity” on the construction documents to the appropriate section. Since the requirement presently exists for all structures, it must apply to both the general case and the conventional light-frame construction case. Even in the simplest case, where the presumptive load-bearing values are used, some classification and design load-bearing capacities are known. In the exceptional case where Section 2308 is being used, the soil classification may be irrelevant, so the requirement is relaxed to reduce the burden on designers.

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

**Committee Action:** **Approved as Modified**

**Modify proposal as follows:**

**1603.1.6 Geotechnical information.** The ~~soil classification and~~ design load-bearing values <u>of soils</u> shall be shown on the construction documents.

(Portions of proposal not shown remain unchanged)

**Committee Reason:** This proposal relocates information that is already in the code to a more appropriate section in Chapter 16, resulting in a clarification of requirements for construction documents. The modification removes “soils classification” in recognition that in some instances the allowable soil pressure may be determined without borings. Where a soils report is performed it is still part of the submittals required by Chapter 1.

**Assembly Action:** **None**

## *Individual Consideration Agenda*

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

*Public Comment:*

**Can Xiao, City of Phoenix, representing herself requests Approval as Modified by this public comment.**

Modify proposal further:

**1603.1.6 Geotechnical information**. The <u>soil classification and</u> design load-bearing values of soils shall be shown on the construction documents.

(Portions of proposal not shown remain unchanged)

**Commenter's Reason:** The proposed code change removed the requirement of soil classification information as a part of construction document. The proposed reason was sometimes soil bearing capacity was determined without borings but per Table 1804.2. But please notice that the soil bearing capacity in Table 1804.2 is still based on soil classification. If the structural engineer of record decides not to have a soil report to determine the soil bearing capacity, he/she will then assume the responsibility to go to the field and verify the soil type then use the corresponding bearing capacity from Table 1804.2 per that soil classification. He or she will be responsible for the soil classification and bearing capacity information used for the design. Engineer of record can not just assume a soil bearing capacity even without knowing the soil classification. Soil classification information is especially important in areas that potentially have expansive/collapsible or saturated clay soil. Certain types of soil classifications also require special engineering fill, excavation and compaction procedures. To remove this important information from construction documents requirement will create potential liability issues that nobody can be held responsible for.

Final Action: AS AM AMPC____ D

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# S53-07/08
## Table 1604.5

*Proposed Change as Submitted:*

**Proponent:** Gary J. Ehrlich, PE, National Association of Home Builders, representing National Association of Home Builders

**Revise table as follows:**

**TABLE 1604.5 (Supp)**
**OCCUPANCY CATEGORY OF BUILDINGS AND OTHER STRUCTURES**

| OCCUPANCY CATEGORY | NATURE OF OCCUPANCY |
|---|---|
| I | Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to:<br>• Agricultural facilities.<br>• Certain temporary facilities.<br>• Minor storage facilities.<br>• <u>Solid signs and freestanding walls.</u> |
| II | Buildings and other structures except those listed in Occupancy Categories I, III and IV |

(Portions of table not shown remain unchanged)

**Reason:** ASCE 7-05 revised the wind load provisions for solid freestanding walls and solid signs to specify increases at the ends of walls and signs to resist winds coming from a 45-degree angle to the wall. These new provisions increase the design loads at the ends of long walls as much as 2-1/2 times over the loads previously obtained (per ASCE 7-05, Figure 6-20, Case C). In order to obtain reasonable results, it needs to be clarified that these structures can be placed in Category I and use the $I_w$ = 0.87 importance factor (or $I_w$ = 0.77 for hurricane-prone regions) assigned to Category I structures.

An example design for two masonry screen walls was provided to us by Curt McDonald of Wright Engineering in Las Vegas (see attached). The first wall was a 6'-0" high, 6" CMU wall with length/height ratio exceeding 45. Under the old provisions, the reinforcing required was #4@48" on center. Under the new provisions, #6@8" are required over the first 6'-0" of wall, from each end. The second wall was an 8'-0" high, 8" CMU wall with length/height ratio exceeding 45. Under the old provisions, the reinforcing required was #4@48" on center. Under the new provisions, #7@8" are required over the first 8'-0" of wall, from each end. Both new requirements strain the limit of constructability given current masonry lap splice length requirements and are certainly not cost-effective.

With the overly conservative nature of the new wall and sign provisions, it makes sense to insure that engineers make use of all the available factors (Iw, Kz, Kd) to reduce the loads to an appropriate level for design. This clarification will result in a 13% reduction in the wind load (or 23% in hurricane-prone regions). Since the actual performance of properly-designed and constructed walls and signs does not justify a 250% increase in the wind loads, this decrease is a small but needed step towards a reasonable design.

This change was also discussed in the ASCE 7-05 Wind Subcommittee's task group for Windows, Doors, Signs and Other Structures where there was agreement the change should be submitted to the General Provisions subcommittee, and to ICC as well so users will not need to wait until 2012 for this simple clarification. NAHB asks for your support of this proposal. Proposal will decrease the cost of constructing a wall under the mew provisions.

**Cost Impact:** The code change proposal will not increase the cost of construction. In fact, it will decrease the cost of constructing a wall under new provisions.

**Committee Action:** **Disapproved**

**Committee Reason:** The application of the additional item under Category I was not clear and a proposed floor modification was not necessarily an improvement. The proponent is encouraged to address this in another manner via the public comment process.

**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 proposal as follows:**

**TABLE 1604.5 (Supp)**
**OCCUPANCY CATEGORY OF BUILDINGS AND OTHER STRUCTURES[a]**

| OCCUPANCY CATEGORY | NATURE OF OCCUPANCY |
|---|---|
| I | Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to:<br>• Agricultural facilities.<br>• Certain temporary facilities.<br>• Minor storage facilities.<br>• ~~Solid signs and freestanding walls~~. |
| II | Buildings and other structures except those listed in Occupancy Categories I, III and IV |

a. Solid freestanding walls not exceeding 10 feet in height and adjacent to R-1, R-2, R-3 or R-4 occupancies not exceeding four stories in height shall be permitted to be designed as Occupancy Category I structures.

(Portions of table not shown remain unchanged)

**Commenter's Reason:** In disapproving this proposal, the IBC-S committee was concerned it would permit a solid freestanding sign or solid freestanding wall adjacent to a building with a high occupant load, or an essential facility, to take the load reduction for a Category I structure. They felt the wall or sign should generally be designed using the same occupancy category as the structure to which it is adjacent. However, they were sympathetic to the issue of the design of masonry screen walls adjacent to residential buildings, which was the reason for the proposal, and suggested a more targeted public comment be developed. This proposed revision to the proposal eliminates solid freestanding signs entirely and moves the proposed language regarding solid freestanding walls to a footnote. Further, the ability to design the wall as a Category I structure is limited to walls 10 feet or less in height and to walls adjacent to a residential occupancy. NAHB asks for your support in approving this proposal as modified and overturning the committee's action.

Final Action: AS AM AMPC____ D

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# S57-07/08
## 1604.8.2

## *Proposed Change as Submitted:*

**Proponent:** Joseph J. Messersmith, Jr., PE, Portland Cement Association, representing Portland Cement Association

**Revise as follows:**

**1604.8.2 ~~Concrete and masonry~~ Walls.** ~~Concrete and~~ masonry Walls shall be anchored to floors, roofs and other structural elements that provide lateral support for the wall. Such anchorage shall provide a positive direct connection capable of resisting the horizontal forces specified in this chapter but not less than ~~a~~ the minimum strength design horizontal force ~~of 280 plf (4.10 kN/m) of wall~~ specified in Section 11.7.3 of ASCE 7, substituted for "*E*" in the load combinations of Section 1605.2 or 1605.3. Concrete and masonry walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet (1219 mm). Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall. See Sections 1609 for wind design requirements and see Section 1613 for earthquake design requirements.

**Reason:** The requirement that anchors attaching concrete and masonry walls to supporting construction be designed for a lower bound value of 280 pounds per linear foot is excessive and discriminatory considering that anchorage of walls of other materials is required to be designed for a horizontal force of 5% of the weight of the wall tributary to the anchor by Section 11.7.3 of ASCE 7-05.

To illustrate the punitive nature of the provision, consider two 10-foot high walls that are representative of walls used in single family dwellings and small commercial buildings; one a 5.5-inch thick concrete wall, the other a light-framed wall with 4-inch thick nominal masonry veneer anchored to the wall framing. The weight of the light-framed wall, including veneer is estimated to be 45 psf; therefore, the weight tributary to an anchor at the top of the wall is 225 plf (45 * (10/2) = 225). The required design anchorage force for this wall is 11 plf (225 * 0.05 = 11). For the 5.5-inch concrete wall (weight 69 psf), the weight tributary to an anchor at the top of the wall is 345 plf (69 * (10/2) = 345). Based on the requirement that applies to walls of other than concrete or masonry, the required anchorage design force for the concrete wall should be 17 plf (345 * 0.05 = 17); however, 280 plf must be used to design the anchorage.

Let's examine how the 280 plf requirement compares to wind design. ASCE 7, Section 6.1.4.2 requires that components and cladding be designed for a minimum service level design wind pressure of 10 psf. For our example walls using this minimum design wind pressure, the strength level (factored) force at the top of the wall due to wind is 80 plf (10 * (10/2) * 1.6 = 80). Now let's determine what basic wind speed is required to result in a factored design force at the top of the wall of 280 plf (strength level). The service level (unfactored) force comparable to 280 plf is 175 plf (280/1.6 = 175). Since 175 plf is based on a tributary wall height of 5 feet, the design wind pressure is 35 psf (175/5 = 35). From ASCE 7, Figure 6-3, for a building in exposure B, height of 30 feet, $K_{zt}$ of 1.0, and effective wind area of less than or equal to 10 square feet for wall area 4, the negative design pressure for a basic wind speed of 140 mph is 38.2 psf. Therefore, the requirement that the 5.5-inch, 10-foot high concrete wall be anchored against a force of 280 plf is the same as requiring that the connection be designed for a basic wind speed of approximately 135 mph in exposure B.

It is obvious that in a world that is rapidly embracing performance-based design, the requirement that anchorages for concrete and masonry walls be designed for a force of 280 plf is not necessary and discriminates against these products. In addition, by singling our walls of concrete and masonry, walls of other materials that could have comparable mass per unit area are exempt from the requirement. Based on the foregoing, the requirement that anchorages for concrete and masonry walls be designed for 280 plf should be deleted. In its place will be a reference to Section 11.7.3 of ASCE 7-05 which has attachments/anchorage requirements that apply too all buildings.

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

**Committee Action:** **Approved as Submitted**

**Committee Reason:** The minimum horizontal force of 280 pounds per foot that has been carried over from legacy codes is arbitrary and overly conservative.

**Assembly Action:** **None**

## *Individual Consideration Agenda*

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

*Public Comment:*

**Can Xiao, City of Phoenix, AZ, representing herself, requests Disapproval.**

**Commenter's Reason:** The 280plf minimum horizontal force design was intent to improve the connections between masonry/concrete walls and floor/roof diaphragm. This is a structural detailing requirement that was intent to achieve "strong connection weak members" seismic design concept. Because masonry/concrete walls have larger self weight, they tend to attract more seismic forces during earthquake. It is important to make sure that the connections between horizontal diaphragm and masonry/concrete walls are strong enough to withstand the seismic forces attracted by masonry/concrete walls, so the building will not collapse and cause human being casualties. This requirement was actually based on several post-earthquake investigations where most damages were found to occur. This requirement is a very important seismic detailing requirement that is for protecting life safety. It shall not be removed from the building code solely because of economic reason. The No.1 priority of building code is to protect public life safety. This priority can not be compromised by economic reason at any time.

The similar proposal to ASCE 7 has been disapproved. ASCE 7 is one of the national standards that are referred by IBC. By approving this code proposal, it makes IBC to be conflicting with ASCE 7. This will definitely create difficulties and problems in code enforcement.

Final Action: AS AM AMPC____ D

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# S58-07/08
## 1604.11, 1605 (New)

## *Proposed Change as Submitted:*

**Proponent:** Edwin Huston, National Council of Structural Engineers Association (NCSEA), representing NCSEA Code Advisory Committee – General Engineering Subcommittee

**Revise as follows:**

**1604.8.3 Decks.** Where supported by attachment to an exterior wall, decks shall be positively anchored to the primary structure and designed for both vertical and lateral loads as applicable. Such attachment shall not be accomplished by the use of toenails or nails subject to withdrawal. Where positive connection to the primary building structure cannot be verified during inspection, decks shall be self-supporting. ~~For~~ In addition to the normal downward acting dead and live load reactions, decks with cantilevered framing members, connections to exterior walls or other framing members shall be designed and constructed to resist uplift resulting from the full live load specified in Table 1607.1 or snow load specified in Section 1608, whichever is greater, acting on the cantilevered portion of the deck, and no live load or snow load on the remaining portion of the span.

**Reason:** The existing last sentence is attempting to address the situation where the load on the cantilevered portion of the span may result in uplift at the support remote from the support at the cantilever. It is accepted engineering practice that for a cantilever, the full live load (or snow load) is placed on the cantilever, with no live or snow load on the remaining portion of the span. This may or may not cause uplift at the support, depending upon many factors. The proposal will clarify the intent and is consistent with the intent of Section 1607.10. Also, see Section 4.6 of ASCE 7-05. In addition, the proposal adds snow load since it is conceivable that snow load could control the design of the deck, especially where snow sliding or drifting from a higher roof must be considered.

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

**Committee Action:** **Approved as Modified**

**Modify proposal as follows:**

**1604.8.3 Decks.** Where supported by attachment to an exterior wall, decks shall be positively anchored to the primary structure and designed for both vertical and lateral loads as applicable. Such attachment shall not be accomplished by the use of toenails or nails subject to withdrawal. Where positive connection to the primary building structure cannot be verified during inspection, decks shall be self-supporting. In addition to the normal downward acting dead and live load reactions, ~~decks with cantilevered framing members,~~ connections to exterior walls or other framing members of decks with cantilevered framing members shall be designed and constructed to resist uplift resulting from the full live load specified in Table 1607.1 or snow load specified in Section 1608, whichever is greater, acting on the cantilevered portion of the deck, and no live load or snow load on the remaining portion of the ~~span~~ deck.

**Committee Reason:** This code change improves the code by clarifying the current required loading condition that affects the critical portion of the deck. The modification is editorial and further clarifies the intention of this code section.

**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 Committee General Engineering Subcommittee, requests Approval as Modified by this Public Comment.**

**Further modify proposal as follows:**

**1604.8.3 Decks.** Where supported by attachment to an exterior wall, decks shall be positively anchored to the primary structure and designed for both vertical and lateral loads as applicable. Such attachment shall not be accomplished by the use of toenails or nails subject to withdrawal. Where positive connection to the primary building structure cannot be verified during inspection, decks shall be self-supporting. ~~In addition to the normal downward acting dead and live load reactions~~, Connections of decks with cantilevered framing members ~~connections~~ to exterior walls or other framing members ~~of decks with cantilevered framing members~~ shall be designed ~~and constructed to resist uplift resulting from the full live load specified in Table 1607.1 or snow load specified in Section 1608, whichever is greater, acting on the cantilevered portion of the deck, and no live load or snow load on the remaining portion of the deck~~ for both of the following:

1. The reactions resulting from the dead, live load specified in Table 1607.1, or the snow load specified in Section 1608, in accordance with Section 1605, acting on all portions of the deck.
2. The reactions resulting from the dead load and live load specified in Table 1607.1 or the snow load specified in Section 1608, in accordance with Section 1605, acting on the cantilevered portion of the deck, and no live load or snow load on the remaining portion of the deck.

**Commenter's Reason:** NCSEA has received the suggested Public Comment from WTCA. They indicated another possible way this section could be interpreted. We agree that the above language is an editorial improvement to section 1604.8.3

Final Action: AS AM AMPC____ D

# S61-07/08

## 1605.1.1 (New)

### *Proposed Change as Submitted:*

**Proponent:** William Sherman, CH2M HILL, representing himself

**Add new text as follows:**

**<u>1605.1.1 Stability.</u>** <u>Where overall structure stability (such as stability against overturning, sliding, or buoyancy) is checked using the load combinations in Section 1605.2, strength reduction factors applicable to soil resistance shall be provided by a qualified geotechnical engineer and consideration shall be given to acceptable behavior at service loads. Where structural elements are designed for strength using the load combinations in 1605.2, it is permissible to check overall structure stability using the load combinations in 1605.3. Where the load combinations in 1605.3 are used to check overall structure stability, the dead load factor in each load combination shall be taken as 1.0 where the factors of safety in Section 1806.1 are applied.</u>

**Reason:** Clarification of code provisions – existing code language implies that it applies to all design conditions but is not clear as to how load combinations are to be used when performing stability analysis of structures. Structure stability safety factors would be different using factored loads than using unfactored loads. Some factors applied to loads in the ASD provisions may duplicate the purpose of separately applied safety factors, such as the "safety factor of 1.5 against overturning or sliding" in Section 1806.1. If factored loads are used, soil resistance must consider strength reduction factors that are not provided by standard building codes.

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

**Committee Action:** **Approved as Modified**

**Modify proposal as follows:**

**1605.1.1 Stability.** <u>Regardless of which load combinations are used to design for strength, w</u>here overall structure stability (such as stability against overturning, sliding, or buoyancy) is ~~checked using~~ <u>being verified, use of</u> the load combinations <u>specified</u> in Section 1605.2~~,~~ <u>or 1605.3 shall be permitted. Where the load combinations specified in Section 1605.2 are used,</u> strength reduction factors applicable to soil resistance shall be provided by a <u>registered design professional.</u> ~~qualified geotechnical engineer and consideration shall be given to acceptable behavior at service loads. Where structural elements are designed for strength using the load combinations in 1605.2, it is permissible to check overall structure stability using the load combinations in 1605.3. Where the load combinations in 1605.3 are used to check overall structure stability, the dead load factor in each load combination shall be taken as 1.0 where the factors of safety in Section 1806.1 are applied.~~ <u>The stability of retaining wall shall be verified in accordance with Section 1807.2.3.</u>

**Committee Reason:** This proposal provides a clarification of the code with respect to stability. The modification improves the wording so that the intent is clearer and also coordinates with Chapter 18 requirements.

**Assembly Action:** **None**

### *Individual Consideration Agenda*

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

*Public Comment:*

**Can Xiao, City of Phoenix, AZ, representing herself, requests Disapproval.**

**Commenter's Reason:** It is general engineering practice to use service (un-factored) load to do structural stability, deflection and other serviceability-related analysis, no matter using allowable stress or LRFD method. The proposed code changes also lack of academic and engineering justifications.

Final Action: AS AM AMPC____ D

# S69-07/08
## Table 1607.1

*Proposed Change as Submitted:*

**Proponent:** Kirk Grundahl, WTCA, representing the Structural Building Components Industry

**Revise table as follows:**

**TABLE 1607.1 (Supp)**
**MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS AND MINIMUM CONCENTRATED LIVE LOADS[g]**

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 $mm^2$,
1 square foot = 0.0929 $m^2$,
1 pound per square foot = 0.0479 $kN/m^2$, 1 pound = 0.004448 kN,
1 pound per cubic foot = 16 $kg/m^3$

j. For attics with limited storage and constructed with trusses, this live load need only be applied to those portions of the bottom chord where there are two or more adjacent trusses with the same web configuration capable of containing a rectangle 42 inches high by 2 feet wide or greater, located within the plane of the truss. The rectangle shall fit between the top of the bottom chord and the bottom of any other truss member, provided that each of the following criteria is met:

i. The attic area is accessible by a pull-down stairway or framed opening in accordance with Section 1209.2, ~~and~~
ii. The truss has ~~shall have~~ a bottom chord pitch less than 2:12~~.~~, and
iii. Required insulation depth is less than the bottom chord member depth.

Bottom chords of trusses meeting the above criteria for limited storage shall be designed for the ~~greater of~~ actual imposed dead load or 10 psf, uniformly distributed over the entire span.

(Portions of table and footnotes not shown remain unchanged)

**Reason:** Purpose: The purpose of the code change is to update and harmonize the code language between the IBC and the IRC by introducing the depth of insulation applied at the bottom chord as a third criterion for evaluating uninhabitable attics with limited storage. This proposal also clarifies that the bottom chords of these trusses are to be designed to for a uniformly distributed actual imposed dead load or 10 psf.

Justification and Substantiation: It makes no good sense for Table 1607.1 and Table 301.5 to be different in the application of truss bottom chord loading. The goal is to provide a uniform loading approach that will be consistently applied in the marketplace.

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

**Committee Action:** **Disapproved**

**Committee Reason:** The committee disapproved this proposal at the recommendation of the proponent.

**Assembly Action:** **None**

*Individual Consideration Agenda*

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

*Public Comment 1:*

**Larry Wainright, WTCA, representing The Structural Building Components Industry, requests Approval as Modified by this public comment.**

**Replace proposal as follows:**

**TABLE 1607.1 (Supp)**
**MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS AND MINIMUM CONCENTRATED LIVE LOADS[g]**

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

1 square foot = 0.0929 m$^2$,
1 pound per square foot = 0.0479 kN/m$^2$, 1 pound = 0.004448 kN,
1 pound per cubic foot = 16 kg/m$^3$

j. For attics with limited storage and constructed with trusses, this live load need only be applied to those portions of the bottom chord where there are two or more adjacent trusses with the same web configuration capable of containing a rectangle 42 inches high by 2 feet wide or greater, located within the plane of the truss. The rectangle shall fit between the top of the bottom chord and the bottom of any other truss member, provided that each of the following criteria is met:

i. The attic area is accessible by a pull-down stairway or framed opening in accordance with Section 1209.2 and
ii. The truss shall have a bottom chord pitch less than 2:12~~.~~, and
iii. Required insulation depth is less than the bottom chord member depth.

Bottom chords of trusses meeting the above criteria for limited storage shall be designed for the greater of actual imposed dead load or 10 psf, uniformly distributed over the entire span.

(Portions of table and footnotes not shown remain unchanged)

**Commenter's Reason:** The purpose of the code change is to update and harmonize the code language between the IBC and the IRC by introducing the depth of insulation applied at the bottom chord as a third criterion for evaluating uninhabitable attics with limited storage.

*Public Comment 2:*

**Larry Wainright, WTCA, representing The Structural Building Components Industry, requests Approval as Modified by this public comment.**

Replace **proposal as follows:**

**TABLE 1607.1 (Supp)**
**MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS AND MINIMUM CONCENTRATED LIVE LOADS[g]**

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm$^2$,
1 square foot = 0.0929 m$^2$,
1 pound per square foot = 0.0479 kN/m$^2$, 1 pound = 0.004448 kN,
1 pound per cubic foot = 16 kg/m$^3$

j. For attics with limited storage and constructed with trusses, this live load need only be applied to those portions of the bottom chord where there are two or more adjacent trusses with the same web configuration capable of containing a rectangle 42 inches high by 2 feet wide or greater, located within the plane of the truss. The rectangle shall fit between the top of the bottom chord and the bottom of any other truss member, provided that each of the following criteria is met:

i. The attic area is accessible by a pull-down stairway or framed opening in accordance with Section 1209.2 and
ii. The truss shall have a bottom chord pitch less than 2:12.
~~iii. Bottom chords of trusses shall be designed for the greater of actual imposed dead load or 10 psf, uniformly distributed over the entire span.~~

(Portions of table and footnotes not shown remain unchanged)

**Commenter's Reason:** Table 1607.1 is a live load table. Discussion of dead loads is not appropriate here. Also, the language does not provide the building designer the option of designing a building for a bottom chord dead load of 10 PSF.

*Public Comment 3:*

**Larry Wainright, WTCA, representing The Structural Building Components Industry, requests Approval as Modified by this public comment.**

**Replace proposal as follows:**

**TABLE 1607.1 (Supp)**
**MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS AND MINIMUM CONCENTRATED LIVE LOADS[g]**

For SI: 1 inch = 25.4 mm, 1 square inch = 645.16 mm$^2$,
1 square foot = 0.0929 m$^2$,
1 pound per square foot = 0.0479 kN/m$^2$, 1 pound = 0.004448 kN,
1 pound per cubic foot = 16 kg/m$^3$

j. For attics with limited storage and constructed with trusses, this live load need only be applied to those portions of the bottom chord where there are two or more adjacent trusses with the same web configuration capable of containing a rectangle 42 inches high by 2 feet wide or greater, located within the plane of the truss. The rectangle shall fit between the top of the bottom chord and the bottom of any other truss member, provided that each of the following criteria is met:

i. The attic area is accessible by a pull-down stairway or framed opening in accordance with Section 1209.2 and
ii. The truss shall have a bottom chord pitch less than 2:12.

~~iii.~~ Bottom chords of trusses meeting the above criteria for limited storage shall be designed for the ~~greater of~~ actual imposed dead load, or if not specified, 10 psf, uniformly distributed over the entire span.

(Portions of table and footnotes not shown remain unchanged)

**Commenter's Reason:** This change will give the building designer the flexibility to design buildings in a more efficient manner by specifying the appropriate loads on the building while still recognizing that these loads are not always specified.

Final Action: AS AM AMPC____ D

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# S75-07/08

## 1607.9.1, Table 1607.9.1, 1607.9.1.2, 1607.9.1.3, 1607.9.1.4, 1607.9.1.5 (New)

*Proposed Change as Submitted:*

**Proponent:** Philip Brazil, PE, SE, Reid Middleton, Inc., representing himself

**Revise as follows:**

**1607.9.1 General.** Subject to the limitations of Sections 1607.9.1.1 through 1607.9.1.4, members for which a value of $K_{LL}A_T$ is 400 square feet (37.16 $m^2$) or more are permitted to be designed for a reduced live load in accordance with the following equation:

(No changes to equation 16-24)

where:

$L$ = Reduced design live load per square foot (meter) of area supported by the member.

$L_o$ = Unreduced design live load per square foot (meter) of area supported by the member (see Table 1607.1).

$K_{LL}$ = Live load element factor (see Table 1607.9.1).

$A_T$ = Tributary area, in square feet (square meters).

~~L shall not be less than 0.50Lo for members supporting one floor and L shall not be less than 0.40Lo for members supporting two or more floors.~~

*L* shall not be less than 0.50 $L_o$ for members supporting one floor and *L* shall not be less than 0.40 $L_o$ for members supporting two or more floors.

**TABLE 1607.9.1**
**LIVE LOAD ELEMENT FACTOR, $K_{LL}$**

| ELEMENT | $K_{LL}$ |
|---|---|
| Interior columns<br>Exterior columns without cantilever slabs | 4<br>4 |
| Edge columns with cantilever slabs | 3 |
| Corner columns with cantilever slabs<br>Edge beams without cantilever slabs<br>Interior beams | 2<br>2<br>2 |
| All other members not identified above including:<br>Edge beams with cantilever slabs<br>Cantilever beams<br>One-way slabs<br>Two-way slabs<br>Members without provisions for continuous shear transfer normal to their span | 1 |

~~**1607.9.1.4**~~ **1607.9.1.1** ~~**Special structural elements**~~ **One-way slabs.** ~~Live loads shall not be reduced for one-way slabs except as permitted in Section 1607.9.1.1.~~ The tributary area, $A_T$, for one-way slabs shall not exceed an area defined by the slab span times a width normal to the span of 1.5 times the slab span. ~~Live loads of 100 psf (4.79 kN/m²) or less shall not be reduced for roof members except as specified in Section 1607.11.2~~

**1607.9.1.~~1.~~2 Heavy live loads.** Live loads that exceed 100 psf (4.79 kN/m2) shall not be reduced.

**Exceptions:**

1. The live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall not be less than *L* as calculated in Section 1607.9.1.
2. 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.

**1607.9.1.~~2.~~3 Passenger vehicle garages.** The live loads shall not be reduced in passenger vehicle garages. ~~except the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall not be less than *L* as calculated in Section 1607.9.1.~~

**Exception:** The live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall not be less than *L* as calculated in Section 1607.9.1.

**1607.9.1.~~3.~~4 ~~Special~~ Group A occupancies.** Live loads of 100 psf (4.79 kN/m$^2$) ~~or less~~ shall not be reduced in ~~public assembly~~ Group A occupancies.

**1607.9.1.5 Roofs members.** Live loads of 100 psf (4.79 kN/m$^2$) or less shall not be reduced for roof members except as specified in Section 1607.11.2.

**Reason:** The purpose of this proposal is to align IBC Section 1607.9.1 on the general method for floor live load reductions with similar provisions in Section 4.8 of ASCE 7-05 and to make related editorial revisions.

"One way slabs" is added to Table 1607.9.1 for consistency with Table 4-2 of ASCE 7.05. The second part of Section 1607.9.1.2 (Section 1607.9.1.3 in proposal) is reformatted into an exception for consistency with Section 4.8.3 of ASCE 7-05.

Section 1607.9.1.3 (Section 1607.9.1.4 in proposal) on public assembly occupancies is changed to Group A occupancies, thus, replacing language that is vague and unenforceable with a classification that is defined by the IBC (refer to Section 303). Public assembly occupancies could be interpreted as other than Group A occupancies but they would typically have an occupant load of less than 50 (i.e., Exception 1 to Section 303.1) and a prohibition on live load reduction is not judged to be warranted in such cases.

Also in Section 1607.9.1.3, "or less" is deleted, which reduces the scope of the section to live loads of 100 psf. Several items in Table 1607.1 list live loads for areas of public assembly that could be classified as a Group A occupancy, including Items #3 (armories and drill rooms), #4 (assembly areas and theaters), #6 (balconies), #8 (dance halls and ballrooms), #10 (dining rooms and restaurants), #19 (gymnasiums), #26 (lobbies of office buildings) and #28 (residential public rooms). A live load of at least 100 psf is specified for all but areas of fixed seats at Item #4. Prohibiting reductions in live loads at areas of fixed seats is not judged to be warranted. Live loads greater than 100 psf are currently covered by Section 1607.9.1.1 on heavy live loads.

Section 1607.9.1.4 is split into two parts. The first part on one-way slabs is relocated to a new Section 1607.9.1.1 and is changed from a general prohibition on live load reduction (except for heavy live loads) to a limit on the determination of tributary area, $A_T$, in the same manner as specified in Section 4.8.4 of ASCE 7-05. The relocation to Section 1607.9.1.1 is proposed because the subject matter of tributary area logically follows the calculation of reduced live load, which is based on tributary area. The sections that follow Section 1607.9.1.1 are largely prohibitions on live load reduction. The second part of Section 1607.9.1.4 is renumbered as Section 1607.9.1.5.

This proposal was prepared in conjunction with related proposals on reduction of roof live loads, live loads at marquees, and reduction of live loads at roofs used for assembly purposes.

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

**Committee Action:** **Approved as Modified**

**Modify proposal as follows:**

**1607.9.1.4 Group A occupancies.** Live loads of 100 psf (4.79 kN/m$_2$) and at areas where fixed seats are located shall not be reduced in Group A occupancies.

(Portions of proposal not shown remain unchanged)

**Committee Reason:** This code change makes adjustments in the live load requirements that further align the IBC with the ASCE 7 live load provisions. The modification clarifies that the restriction on live load reductions in Group A occupancies also includes areas with fixed seating.

**Assembly Action:** **None**

## *Individual Consideration Agenda*

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

*Public Comment 1:*

**Edwin Huston, National Council of Structural Engineers Association (NCSEA), representing NCSEA Code Advisory Committee – General Engineering Subcommittee, requests Approval as Modified by this public comment.**

**Further modify proposal as follows:**

**1607.9.1.1 One-way slabs.** The tributary area, $A_T$, for use in Equation 16-24 for one-way slabs shall not exceed an area defined by the slab span times a width normal to the span of 1.5 times the slab span.

(Portions of proposal not shown remain unchanged)

**Commenter's Reason:** There are two methods of live load reduction in the IBC, a Basic Method in Section 1607.9.1 which uses Equation 16-24 and an Alternate Method in Section 1607.9.2 which uses Equation 16-25. S75-07/08 defines $A_T$ to agree with ASCE 7 to allow an area reduction for one-way slabs in the Basic Method. The Alternate Method does not have this definition. In the absence of guidance, NCSEA is concerned that Registered Design Professionals might misapply the definition of $A_T$ in S75-07/08 for the Basic Method to the Alternate Method. In the Basic Method the maximum reduction occurs at a slab span of about 49 feet. If the definition of $A_T$ in S75-07/08 for the Basic Method was applied to the Alternate Method the maximum reduction of 40% for the Alternate Method would occur at a slab span of less than 21 feet!! We consider this a possible life safety issue.

Given the potential life safety risks associated with this potential gap in the IBC, we urge you to approve this code change.

## *Public Comment 2:*

## Edwin Huston, National Council of Structural Engineers Association (NCSEA), representing NCSEA Code Advisory Committee, General Engineering Subcommittee requests Approval as Modified by this Public Comment.

**Further modify proposal by adding new item as follows:**

**1607.9.2 Alternate floor live load reduction.** As an alternative to Section 1607.9.1, floor 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 in Group A occupancies.
2. 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 20 percent.
3. 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.
4. 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 the following equation:
5. For one-way slabs, the area, A, for use in Equation 16-25 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 16-25)**

For SI: R = 0.861 (A -13.94)

Such reduction shall not exceed the smallest of:

1. 40 percent for horizontal members;
2. 60 percent for vertical members; or
3. R as determined by the following equation.

R= 23.1 (1 + D/Lo) **(Equation 16-26)**

where:

A = Area of floor supported by the member, square feet (m2).
D = Dead load per square foot (m2) of area supported.
Lo = Unreduced live load per square foot (m2) of area supported.
R = Reduction in percent.

**Commenter's Reason:** There are two methods of live load reduction in the IBC, a Basic Method in Section 1607.9.1 which uses Equation 16-24 and an Alternate Method in Section 1607.9.2 which uses Equation 16-25. S75-07/08 defines $A_T$ to agree with ASCE 7 to allow an area reduction for one-way slabs in the Basic Method. The Alternate Method does not have this definition. In the absence of guidance, NCSEA is concerned that Registered Design Professionals might misapply the definition of $A_T$ in S75-07/08 for the Basic Method to the Alternate Method. In the Basic Method the maximum reduction occurs at a slab span of about 49 feet. If the definition of $A_T$ in S75-07/08 for the Basic Method was applied to the Alternate Method the maximum reduction of 40% for the Alternate Method would occur at a slab span of less than 21 feet!! We consider this a possible life safety issue. NCSEA submitted a Code Change Proposal on S77 which we are hereby modifying to address this issue.

NCSEA has looked at the relation ship between Equations 16-24 and 16-25 and finds that the best agreement between the two methods of live load reduction would be to use a definition of A in Equation 16-25 of the slab span times a width normal to the span of 0.5 times the slab span. A comparison table is shown below.

| Summary % LL Reduction | | | | | |
|---|---|---|---|---|---|
| Span | Eq 16-24 | Eq 16-25 | | | |
| | | 0.5 span | .75 span | Span | 1.5 Span |
| 20 | 14 | 4 | 12 | 20 | 36 |
| 25 | 26 | 13 | 26 | 38 | 40 max |
| 30 | 34 | 24 | 40 max | 40 max | 40 max |
| 40 | 44 | 40 max | 40 max | 40 max | 40 max |
| 49 | 50 max | 40 max | 40 max | 40 max | 40 max |

Given the potential life safety risks associated with this potential gap in the IBC, we urge you to approve this code change.

Final Action: AS AM AMPC____ D

# S79-07/08, Part I

## 1609.1.1, 1609.1.1.1, 2308.2.1, Chapter 35

***NOTE: PART II DID NOT RECEIVE A PUBLIC COMMENT AND IS ON THE CONSENT AGENDA. PART II IS REPRODUCED FOR INFORMATIONAL PURPOSES ONLY FOLLOWING ALL OF PART I.***

*Proposed Change as Submitted:*

**Proponent:** Med Kopczynski, City of Keene, NH, representing ICC IS-HRC Standards Committee

**PART I – IBC STRUCTURAL**

**1. Revise as follows:**

**1609.1.1 (Supp) Determination of wind loads.** Wind loads on every building or structure shall be determined in accordance with Chapter 6 of ASCE 7. The type of opening protection required, the basic wind speed 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 ~~SBCCI SSTD 10~~ 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 the AF&PA WFCM.
3. Designs using NAAMM FP 1001.
4. Designs using TIA-222 for antenna-supporting structures and antennas.
5. Wind Tunnel tests in accordance with Section 6.6 of ASCE 7, subject to the limitations in Section 1609.1.1.2.

**1609.1.1.1 Applicability**. The provisions of ~~SSTD 10~~ ICC-600 are applicable only to buildings located within Exposure B or C as defined in Section 1609.4. The provisions of ~~SBCCI SSTD 10~~ ICC-600 and the AF&PA WFCM shall not apply to buildings sited on the upper half of an isolated hill, ridge or escarpment meeting the following conditions:

1. The hill, ridge or escarpment is 60 feet (18 288 mm) or higher if located in Exposure B or 30 feet (9144 mm) or higher if located in Exposure C;
2. The maximum average slope of the hill exceeds 10 percent; and
3. The hill, ridge or escarpment is unobstructed upwind by other such topographic features for a distance from the high point of 50 times the height of the hill or 1 mile (1.61 km), whichever is greater.

**2308.2.1 Basic wind speed greater than 100 mph (3-second gust).** Where the basic wind speed exceeds 100 mph (3-second gust), the provisions of either AF&PA WFCM, or the ~~SBCCI SSTD 10~~ ICC-600 are permitted to be used.

**2. Revise standards as follows:**

**International Code Council (ICC)**

~~SBCCI SSTD 10-99 Standard for Hurricane Resistance Residential Construction~~

ICC-600 Standard for Residential Construction in High Wind Regions

**Reason:** This proposal is to delete the current ICC legacy Standard SSTD 10 – 99 and replace with the new ICC– 600 Standard for Residential Construction in High Wind Regions.

The ICC legacy standard SSTD 10 – 99 and its predecessors were the first US standards for high wind construction of residential structures. The SSTD 10 is based on the Standard Building Code wind loads and which used fastest-mile wind speeds. Although dated, the SSTD 10 is referenced by the IBC and IRC.

The new ICC– 600 standard provides a set of specifications that is consistent with the International Building Code and ASCE 7 wind loads, wind speed maps, and conventions. The primary focus of the update effort has been to provide a contemporary set of prescriptive requirements that supplement the International Residential Code provisions.

The ICC– 600 was developed by the ICC Consensus Committee on Hurricane Resistant Construction (IS-HRC) that operates under ANSI Approved ICC Consensus Procedures. A copy of a draft of the standard has been submitted to the ICC as allowed by ICC Council Policy; CP#28.ANSI certification of the standard is expected to be received prior to the ICC Final Action Hearings in September 2008.

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

**PART I – IBC STRUCTURAL**

**Committee Action:** **Disapproved**

**Committee Reason:** While the new reference standard will replace an outdated standard, it is not yet complete. It is hoped that a public comment can be submitted to allow this standard to be referenced by the code.

**Assembly Action:** **None**

## *Individual Consideration Agenda*

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

*Public Comment 1:*

**Med Kopczynski, City of Keene, NH, representing ICC IS-HRC Standards Committee, requests Approval as Submitted.**

**Commenter's Reason:** This proposal is to delete the current ICC legacy Standard SSTD 10 – 99 and replace with the new ICC– 600 Standard for Residential Construction in High Wind Regions. The SSTD 10 is based on the Standard Building Code wind loads and which used fastest-mile wind speeds. The new ICC– 600 standard provides a set of specifications that is consistent with the International Building Code and ASCE 7 wind loads, wind speed maps, and conventions.

The IBC-Structural Committee disapproved this code change because the standard proposed for inclusion in the code, ICC 600, was not finalized. Since that time, the ICC/IS-HRC Standards Committee has completed its work, submitted the standard to ANSI for approval, received approval, and published the standard. Other than the objections stated regarding the completion of the standard, the committee had no other technical concerns or issues regarding this proposed code change. Given that the standard proposed for reference is now complete and in compliance with the ICC criteria for referenced standards given in ICC Council Policy no. CP 28, the ICC/IS- HRC Standards Committee request approval of the proposed code change S79-07/08 Part I "As Submitted."

*Public Comment 2:*

**Joseph J. Messersmith, Jr., P.E., Portland Cement Association, requests Approval as Submitted.**

**Commenter's Reason:** As indicated in the committee reason, the change to reference ICC-600 in the IBC was disapproved because the standard was not complete. It should be noted that Part II of this change to reference this new standard in the IRC was approved in Palm Springs. Since the Palm Springs hearings, the standard has been completed and the membership is urged to overturn the motion for disapproval and subsequently vote to approve Part I of the change.

Final Action: AS AM AMPC____ D

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***NOTE: PART II REPRODUCED FOR INFORMATIONAL PURPOSES ONLY – SEE ABOVE***

S79-07/08, Part II – IRC B/E

**1. Revise as follows:**

**R301.2.1.1 (Supp) Design criteria.** In regions where the basic wind speeds from Figure R301.2(4) equal or exceed 100 miles per hour (45 m/s) in hurricane-prone regions, or 110 miles per hour (49 m/s) elsewhere, the design of buildings shall be in accordance with one of the following methods. The elements of design not addressed by those documents in Items 1 through 4 shall be in accordance with this code.

1. American Forest and Paper Association (AF&PA) *Wood Frame Construction Manual for One- and Two- Family Dwellings* (WFCM); or
2. ~~*Southern Building Code Congress International Standard for Hurricane Resistant Residential Construction* (SSTD 10);~~ *International Code Council (ICC) Standard for Residential Construction in High Wind Regions (ICC-600);* or
3. *Minimum Design Loads for Buildings and Other Structures* (ASCE-7); or
4. American Iron and Steel Institute (AISI), *Standard for Cold-Formed Steel Framing—Prescriptive Method For One- and Two-Family Dwellings (COFS/PM) with Supplement to Standard for Cold-Formed Steel Framing—Prescriptive Method For One- and Two-Family Dwellings*.
5. Concrete construction shall be designed in accordance with the provisions of this code.
6. Structural insulated panels shall be designed in accordance with the provisions of this code.

**2. Revise standards as follows:**

**International Code Council (ICC)**

~~SBCCI SSTD 10-99 Standard for Hurricane Resistance Residential Construction~~
ICC-600 Standard for Residential Construction in High Wind Regions

**Reason/Cost Impact:** Same as Part I

**PART II – IRC B/E**
**Committee Action:** **Approved as Submitted**

**Committee Reason:** This new standard, ICC-600 Standard for Residential Construction in High Wind Regions, will be a great improvement over the legacy standard SSTD 10. The standard provides a set of specifications that is consistent with the IBC and ASCE 7 wind loads, wind speed maps, and conventions.

**Assembly Action:** **None**

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# S81-07/08

## 1609.1.1, 1609.1.1.2, Chapter 35 (New)

*Proposed Change as Submitted:*

**Proponent:** Paul K. Heilstedt, P.E., Chair, representing ICC Code Technology Committee (CTC)

**1. Revise as follows:**

**1609.1.1 (Supp) Determination of wind loads:** Wind loads on every building or structure shall be determined in accordance with Chapter 6 of ASCE 7. The type of opening protection required, the basic wind speed 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 SBCCI SSTD 10 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 the AF&PA WFCM.
3. Designs using NAAMM FP 1001.
4. Designs using TIA/EIA-222 for antenna-supporting structures and antennas.
5. Wind tunnel tests in accordance with Section 6.6 of ASCE 7, subject to the limitations in Section 1609.1.1.2.
6. Wind tunnel tests in accordance with ASCE/SEI 49, subject to the limitations in Section 1609.1.1.2.

**1609.1.1.2 (Supp) Wind tunnel test limitations.** The lower limit on pressures for main wind-force-resisting systems and components and cladding shall be in accordance with Sections 1609.1.1.2.1 and 1609.1.1.2.2. The minimum design wind load shall not be less than the minimum prescribed in Chapter 6 of ASCE 7.

**2. Add standard to Chapter 35 as follows:**

**American Society of Civil Engineers/Structural Engineering Institute**

ASCE/SEI 49-07 Wind Tunnel Testing for Buildings and Other Structures

**Reason:** The ICC Board established the ICC Code Technology Committee (CTC) as the venue to discuss contemporary code issues in a committee setting which provides the necessary time and flexibility to allow for full participation and input by any interested party. The code issues are assigned to the CTC by the ICC Board as "areas of study". Information on the CTC, including: meeting agendas; minutes; reports; resource documents; presentations; and all other materials developed in conjunction with the CTC effort can be downloaded from the following website: **http://www.iccsafe.org/cs/cc/ctc/index.html** Since its inception in April/2005, the CTC has held twelve meetings - all open to the public.

**This proposed change is a follow-up to S16-06/07 which was a result of the CTC's investigation of the area of study entitled "Review of NIST WTC Recommendations". The scope of the activity is noted as:**

**Review the recommendations issued by NIST in its report entitled "Final Report on the Collapse of the World Trade Center Towers", issued September 2005, for applicability to the building environment as regulated by the I-Codes.**

This proposal is intended to address NIST recommendation 2. For this specific proposed change, CTC is working in cooperation with the NIBS/MMC Committee to Translate the NIST World Trade Center Investigation Recommendations for the Model Codes. The CTC notes in their investigation that many of the recommendations contained in the NIST report require additional information for the CTC to further investigate. As such, CTC intends to continue to study the other NIST recommendations.

**NIST Recommendation 2 recommends that nationally accepted performance standards be developed for: (1) conducting wind tunnel testing of prototype structures based on sound technical methods that result in repeatable and reproducible results among testing laboratories; and (2) estimating wind loads and their effects on tall buildings for use in design, based on wind tunnel testing data and directional wind speed data.**

The IBC requires that wind loads be determined in accordance with Chapter 6 of ASCE 7, with specific exceptions depending on the size, configuration and location of the building. Section 6.1 of ASCE 7-05 provides three procedures to determine design wind loads: Method 1- Simplified Procedure; Method 2- Analytical Procedure; and Method 3- Wind Tunnel Procedure. Due to unique wind load considerations for certain building configurations and locations, Section 6.5.2 of ASCE 7 - 05 further mandates compliance with either the wind tunnel procedure of Section 6.6 of ASCE 7 or requires the design to be based on recognized literature documenting the wind load effects. Section 6.6 of ASCE does not currently prescribe specific wind tunnel test procedures. These are being developed by an ASCE Wind Tunnel Testing standard committee.

The purpose of this change is not to mandate wind tunnel testing in the IBC, but rather to achieve uniformity in results where the design involves wind tunnel testing – either as required by ASCE 7 or where the designer determines that wind tunnel testing is to be used to determine the wind loads.

The proposed revision that stipulates that the minimum design loads can not be less than the minimums of ASCE 7 (10 psf) is in response to the committees concern stated in the reason for disapproval of S16 -06/07. It is CTC's understanding that the standard will have been completed by the 2008 Palm Springs Code Development Hearings.

**References:**

Interim Report No. 1 of the CTC, Area of Study – Review of NIST WTC Recommendations, March 9, 2006.

National Institute of Standards and Technology. Final Report of the National Construction Safety Team on the Collapses of the World Trade Center Towers. United States Government Printing Office: Washington, D.C. September 2005.

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

### Committee Action: Disapproved

**Committee Reason:** The proposed standard has not been completed. It is hoped that a public comment can be submitted to allow this standard to be referenced by 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:*

### Paul K. Heilstedt, P.E., AIA, representing ICC Code Technology Committee (CTC), requests Approval as Submitted.

**Commenter's Reason:** The reason this code change was not approved was due to the lack of completion/availability of the standard ASCE/SEI 49 entitled "Wind Tunnel Testing for Buildings and Other Structures". At the time this public comment is submitted, the standard is still under development. As such, this comment is submitted in anticipation of the standard being completed by the Final Action Hearings. If it is not completed, this public comment will not be pursued, with the proposed reference held until the 2009/2010 Cycle.

Code issues are assigned to the CTC by the ICC Board as "areas of study". Information on the CTC, including: meeting agendas; minutes; reports; resource documents; presentations; and all other materials developed in conjunction with the CTC effort can be downloaded from the following website: http://www.iccsafe.org/cs/cc/ctc/index.html. Since its inception in Aprril/2005, the CTC has held fifteen meetings - all open to the public. This public comment is a result of the CTC's investigation of the area of study entitled "NIST World Trade Center Recommendations". The CTC web page for this area of study is: http://www.iccsafe.org/cs/cc/ctc/WTC.html

Final Action: AS AM AMPC____ D

# S83-07/08
## 1612.3.2 (New)

*Proposed Change as Submitted:*

**Proponent:** John Woestman, The Kellen Company, representing Door Safety Council

**1. Add new text as follows:**

<u>**1609.1.2.1 Side-hinged doors.** Side-hinged door assemblies shall be permitted to meet the impact testing requirements of ANSI/SDI A250.13.</u>

**2. Add standard to Chapter 35 as follows:**

**ANSI**

<u>ANSI/SDI A250.13-XX</u> <u>Testing and Rating of Severe Windstorm Resistant Components for Swinging Door Assemblies</u>

**Reason:** This proposed change allows an alternative method to demonstrate performance to impact-resistant requirements for side-hinged doors by requiring doors to be tested per ANSI/SDI A250.13-XX. A250.13-XX, which is under development to update A250.13-03, will contain language that prescribes how door components are to be selected to create door assemblies expected to perform equivalently to a door assembly tested to ASTM E 1996 / E 1886 for impact resistance.

This proposal helps resolve performance and code compliance issues when doors are assembled from components from multiple sources and include interchangeable elements.

Through the ASTM standards development process, members of the Steel Door Institute (SDI) and members of the Builders Hardware Manufacturers' Association (BHMA) developed a national standard for a component-based approach to testing for windstorm resistance of swinging door assemblies. The test procedures used in this standard represent the most severe requirements found in the windstorm resistance standards in use in building codes. However, the procedures are designed to isolate, as much as possible, the loads and conditions that a particular component is subjected to in the full assembly test and duplicate these specific conditions. Using a combination of worst-case assembly design and safety factors, this testing was designed to provide a component rating that related directly to the component's ability to withstand the conditions that occur in a full assembly test.

**Prior to releasing the current ANSI/SDI A250.13 standard, the BHMA/SDI task group conducted validation testing where components were expected to be rated at three design-load target values. Those components were tested to establish their ratings by the proposed procedure. Following this process, complete assemblies were tested in accordance with the ASTM E1886 test method. The results of this process confirmed that assemblies made up of rated components would perform as expected. In addition, the validation test showed that where a component was identified as the weakest element of an assembly, based on the component tests, the same component would fail in a similar manner when tested as part of an assembly to levels exceeding the component's rated capacity.**

**Building designers will use performance criteria of door components, per ANSI/SDI A250.13, to select appropriate components to create door assemblies by conducting an opening-by-opening design analysis, specify components, verify code compliance, and submit the results through the normal plans review process. Code Authorities will thus need only to verify that the design load and compliance analysis has been correctly carried out and that the specified components are actually installed during construction in accordance with the manufacturer's instructions and project specifications.**

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

**Analysis:** Review of proposed new standard SDI A250.13 indicated that, in the opinion of ICC Staff, the standard did not comply with ICC standards criteria, Section 3.6.2.1.

**Committee Action:** **Disapproved**

**Committee Reason:** The newest edition of the standard is not yet completed, while the former edition did not meet ICC requirements for referenced standards.

**Assembly Action:** **None**

*Individual Consideration Agenda*

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

*Public Comment 1:*

**John Woestman, The Kellen Company, representing Door Safety Council (DSC), requests Approval as Modified by this Public Comment.**

**Modify proposal as follows:**

~~**1609.1.2.1 Side-hinged doors**. Side-hinged door assemblies shall be permitted to meet the impact testing requirements of ANSI/SDI A250.13.~~

**1609.1.2 Protection of openings**. In wind-borne debris regions, glazing in buildings shall be impact-resistant or protected with an impact-resistant covering meeting the requirements of an approved impact-resisting standard or ASTM E 1996 and ASTM E 1886 referenced therein as follows:

1. Glazed openings located within 30 feet (9144 mm) of grade shall meet the requirements of the Large Missile Test of ASTM E 1996.
2. Glazed openings located more than 30 feet (9144 mm) above grade shall meet the provisions of the Small Missile Test of ASTM E 1996.

**Exceptions:**

1. Wood structural panels with a minimum thickness of 7/16 inch (11.1 mm) and maximum panel span of 8 feet (2438 mm) shall be permitted for opening protection in one- and two-story buildings. Panels shall be precut so that they shall be attached to the framing surrounding the opening containing the product with the glazed opening. Panels shall be secured with the attachment hardware provided. Attachments shall be designed to resist the components and cladding loads determined in accordance with the provisions of ASCE 7. Attachment in accordance with Table 1609.1.2 is permitted for buildings with a mean roof height of 33 feet (10058 mm) or less where wind speeds do not exceed 130 mph (57.2 m/s).
2. Glazing in Occupancy Category I buildings as defined in Section 1604.5, including greenhouses that are occupied for growing plants on a production or research basis, without public access shall be permitted to be unprotected.
3. Glazing in Occupancy Category II, III or IV buildings located over 60 feet (18 288 mm) above the ground and over 30 feet (9144 mm) above aggregate surface roofs located within 1,500 feet (458 m) of the building shall be permitted to be unprotected.
4. Components of side-hinged door assemblies shall be permitted to be interchangeable provided the interchange components are tested in accordance with ANSI/SDI A250.13 and meet the required impact performance criteria of ANSI/SDI A250.13.

**Add standard to Chapter 35 as follows:**

**Steel Door Institute**
ANSI/SDI A250.13-~~XX~~08 Testing and Rating of Severe Windstorm Resistant Components for Swinging Door Assemblies
**Commenter's Reason:** The committee's reason for recommending disapproval was that the newest edition of ANSI/SDI A250.13 was not yet completed the ANSI ballot process at the time of the committee hearings. Subsequent to the committee hearings, ANSI/SDI A250.13-2008 was approved May 21, 2008 with a publication date of May 30, 2008.

The updated A250.13 standard incorporates requirements for testing swinging door components (i.e. side-hinge door components) for impact resistance. This updated standard also incorporates requirements for selecting the tested components to create a swinging door assembly (side-hinged door assembly) to meet the impact testing requirements of the code. The proposed modification more appropriately places the provisions of A250.13 regarding side-hinged door components as an exception.

This code change will allow builders, designers, and architects to select a more extensive range of door components while maintaining the impact resistance requirements of the code.

The Door Safety Council recommends "Approval as Modified by this Public Comment" at the Final Action Hearing.

*Public Comment 2:*

## John Woestman, The Kellen Company, representing Door Safety Council (DSC), requests Approval as Modified by this Public Comment.

**Modify proposal as follows:**

ANSI/SDI A250.13-~~XX~~08 Testing and Rating of Severe Windstorm Resistant Components for Swinging Door Assemblies

(Portions of proposal not shown remain unchanged)

**Commenter's Reason:** The committee's reason for recommending disapproval was that the newest edition of ANSI/SDI A250.13 was not yet completed the ANSI ballot process at the time of the committee hearings. Subsequent to the committee hearings, ANSI/SDI A250.13-2008 was approved May 21, 2008 with a publication date of May 30, 2008.

The updated A250.13 standard incorporates requirements for testing swinging door components (i.e. side-hinge door components) for impact resistance. This updated standard also incorporates requirements for selecting the tested components to create a swinging door assembly (side-hinged door assembly) to meet the impact testing requirements of the code.

This code change will allow builders, designers, and architects to select a more extensive range of door components while maintaining the impact resistance requirements of the code.

The Door Safety Council recommends "Approval as Modified by this Public Comment" at the Final Action Hearing.

Final Action: AS AM AMPC____ D

# S84-07/08
## 1609.1.1, 1609.6

*Proposed Change as Submitted:*

**Proponent:** Edwin Huston, National Council of Structural Engineers Associations (NCSEA), representing NCSEA Code Advisory Committee – General Engineering Subcommittee

**1. Revise as follows:**

**1609.1.1 (Supp) Determination of wind loads.** Wind loads on every building or structure shall be determined in accordance with Chapter 6 of ASCE 7 or provisions of the Alternate All-heights Method in Section 1609.6. The type of opening protection required, the basic wind speed 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 SBCCI SSTD 10 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 the AF&PA WFCM.
3. Designs using NAAMM FP 1001.
4. Designs using TIA-222 for antenna-supporting structures and antennas.
5. Wind Tunnel tests in accordance with Section 6.6 of ASCE 7, subject to the limitations in Section 1609.1.1.2.

**2. Add new text as follows:**

**1609.6 Alternate All-Heights method.** The alternate wind design provisions in this section are simplifications of the ASCE 7 Method 2-Analytical Procedure.
**1609.6.1 Scope.** As an alternate to ASCE 7 Section 6.5, the following provisions are permitted to be used to determine the wind effects on regularly shaped buildings, or other structures which meet all of the following conditions:

1. The building or other structure is less than 100 feet (30480 mm) in height, with a height to least width ratio of 4 or less.
2. The building or other structure is not sensitive to dynamic effects.
3. The building or other structure is not located on a site for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.

**1609.6.1.1 Modifications**. The following modifications shall be made to certain subsections in ASCE 7: Section 1609.6.3 Symbols and Notations that are specific to this section are used in conjunction with the Symbols and Notations in ASCE 7 Section.6.3.

**1609.6.2 Symbols and notations.** Coefficients and variables used in the Alternate All-Heights Method equations are as follows:

$C_{net}$ = net-pressure coefficient based on $K_d$ [$(G)$ $(C_p)$ – $(GC_{pi})$], Ref Table 1609.6.2(2)
$G$ = Gust effect factor equal to 0.85 for rigid structures per ASCE 7 Section 6.5.8.1.
$K_d$ = Wind directionality factor per ASCE 7 Table 6-4.
$P_{net}$ = Design wind pressure to be used in determination of wind loads on buildings or other structures or their components and cladding, in lb/ft$^2$ (N/m$^2$).
$q_s$ = Wind velocity pressure in lb/ft$^2$ (N/m$^2$). (Per Table 1609.6.2(1))

**1609.6.3 Design equations.** When using the Alternate All-Heights Method, the Main-Wind-Force-Resisting System, (MWFRS) and Components and Cladding of every structure shall be designed to resist the effects of wind pressures on the building envelope in accordance with Equation (16-36).

$$P_{net} = q_s K_z C_{net} [I K_{zt}] \qquad \textbf{(Equation 16-36)}$$

Design wind forces for the MWFRS shall not be less than 10 $lb/ft^2$ (0.48 $KN/_m{}^2$) multiplied by the area of the structure projected on a plane normal to the assumed wind direction. See ASCE Section 6.1.4 for criteria. Design net wind pressure for components and cladding shall not be less than 10 $lb/ft^2$ (0.48 $KN/_m{}^2$) acting in either direction normal to the surface.

**1609.6.4 Design procedure.** The MWFRS and the components and cladding of every building or other structure shall be designed for the pressures calculated using Equation (16-36).

**1609.6.4.1 Main Wind-Force-Resisting Systems.** The MWFRS shall be investigated for the torsional effects identified in ASCE 7 Figure 6-9.

**1609.6.4.2 Determination of $K_z$ and $K_{zt}$. Velocity Pressure Exposure Coefficient,** $K_z$, shall be determined in accordance with ASCE 7 Section 6.5.6.6 and the Topographic Factor, $K_{zt}$ shall be determined in accordance with ASCE 7 Section 6.5.7.

1. For the windward side of a structure, $K_{zt}$ and $K_z$ shall be based on height z.
2. For leeward and side walls, and for windward and leeward roofs, $K_{zt}$ and $K_z$ shall be based on mean roof height *h*.

**1609.6.4.3 Determination of net pressure coefficients, $C_{net}$.** For the design of the Main Wind-Force-Resisting-System and for Components and Cladding, the sum of the internal and external net pressure shall be based on the net pressure coefficient $C_{net}$.

1. The pressure coefficient, $C_{net}$, for walls and roofs shall be determined from Table 1609.6.2(2).
2. Where $C_{net}$ may have more than one value, the more severe wind load combination shall be used for design.

**1609.6.4.4 Application of wind pressures.** When using the Alternate All-Heights Method, wind pressures shall be applied simultaneously on, and in a direction normal to, all building envelope wall and roof surfaces.

**1609.6.4.4.1 Components and cladding.** Wind pressure for each component or cladding element is applied as follows using $C_{net}$ values based on the effective wind area, *A* contained within the zones in areas-of-discontinuity of width and/or length “a”, “2a” or “4a” at: corners of roofs and walls; edge strips for ridges, rakes and eaves; or field areas on walls or roofs as indicated in Figures in Table 1609.6.2(2) in accordance with the following:

1. Calculated pressures at local discontinuities acting over specific edge strips or corner boundary areas.
2. Include “field” (zone 1, 2 or 4, as applicable) pressures applied to areas beyond the boundaries of the areas-of-discontinuity.
3. Where applicable, the calculated pressures at discontinuities (zones 2 or 3) shall be combined with design pressures that apply specifically on rakes or eave overhangs.

**TABLE 1609.6.2(1)**
**WIND VELOCITY PRESSURE ($q_s$) AT STANDARD HEIGHT OF 33 FEET [a, b, c]**

| **BASIC WIND SPEED, V (mph)** | 85 | 90 | 100 | 105 | 110 | 120 | 125 | 130 | 140 | 150 | 160 | 170 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **PRESSURE, $q_s$ (psf)** | 18.5 | 20.7 | 25.6 | 28.2 | 31.0 | 36.9 | 40.0 | 43.3 | 50.2 | 57.6 | 65.5 | 74.0 |

a. For Wind Speeds not shown, use $q_s = 0.00256\ V^2$
b. Multiply by 1.61 to convert to km/h
c. Multiply by 0.048 to convert to $kN/m^2$

**TABLE 1609.6.2(2)**
**NET PRESSURE COEFFICIENTS, $C_{net}$[a, b, c]**

| STRUCTURE OR PART THEREOF | DESCRIPTION | | $C_{net}$ FACTOR | | |
|---|---|---|---|---|---|
| 1. Main Wind Force Resisting Frames and Systems | WALLS: | | Enclosed | Part Enclosed | |
| | Windward Wall | | 0.43 | 0.11 | |
| | Leeward Wall | | -0.53 | -0.83 | |
| | Side Wall | | -0.66 | -0.97 | |
| | Parapet Wall | Windward | 1.28 | 1.28 | |
| | | Leeward | -0.85 | -0.85 | |
| | | | | | |
| | ROOFS: | | Enclosed | Part Enclosed | |
| | Wind perpendicular to ridge | | | | |
| | Leeward roof or flat roof | | -0.66 | -0.97 | |
| | Windward roof slopes: | | | | |
| | Slope < 2:12 ( 10°) | | -1.09 | -1.41 | |
| | Slope = 4:12 ( 18°) | | -0.73 | -1.04 | |
| | Slope = 5:12 ( 23°) | | -0.58 | -0.90 | |
| | Slope = 6:12 ( 27°) | Case 1 | -0.47 | -0.78 | |
| | | Case 2 | 0.20 | 0.51 | |
| | Slope = 7:12 ( 30°) | Case 1 | -0.37 | -0.68 | |
| | | Case 2 | 0.30 | 0.61 | |
| | Slope 9:12 ( 37°) | Case 1 | -0.27 | -0.58 | |
| | | Case 2 | 0.31 | 0.63 | |
| | Slope 12:12 ( 45°) | | 0.37 | 0.68 | |
| | Wind parallel to ridge and flat roofs | | -1.09 | -1.41 | |
| | | | | | |
| | Non Building Structures: Chimneys, Tanks and Similar Structures: | | | | |
| | | | h/D | | |
| | | | 1 | 7 | 25 |
| | Square (Wind normal to face) | | 0.99 | 1.07 | 1.53 |
| | Square (Wind on diagonal) | | 0.77 | 0.84 | 1.15 |
| | Hexagonal or Octagonal | | 0.81 | 0.97 | 1.13 |
| | Round | | 0.65 | 0.81 | 0.97 |
| | | | | | |
| | Open Signs and Lattice Frameworks | | Ratio of solid to gross area | | |
| | | | < 0.1 | 0.1 to 0.29 | 0.3 to 0.7 |
| | Flat | | 1.45 | 1.30 | 1.16 |
| | Round | | 0.87 | 0.94 | 1.08 |
| 2. Components and Cladding not in areas of discontinuity – Roofs and overhangs | Roof Elements and slopes | | Enclosed | Partially Enc. | |
| | Gable or Hipped Configurations (Zone 1) | | | | |
| | Flat < Slope < 6:12 ( 27°) | | | | |
| | Positive | 10 SF or less | 0.58 | 0.89 | |
| | | 100 SF or more | 0.41 | 0.72 | |
| | Negative | 10 SF or less | -1.00 | -1.32 | |

| | | | | |
|---|---|---|---|---|
| | | 100 SF or more | -0.92 | -1.23 |
| | Overhang: Flat < Slope < 6:12 ( 27°) | | | |
| | Negative | 10 SF or less | -1.45 | |
| | | 100 SF or more | -1.36 | |
| | | 500 SF or more | -0.94 | |
| | 6:12 (27°) < Slope < 12:12 ( 45°) | | | |
| | Positive | 10 SF or less | 0.92 | 1.23 |
| | | 100 SF or more | 0.83 | 1.15 |
| | Negative | 10 SF or less | -1.00 | -1.32 |
| | | 100 SF or more | -0.83 | -1.15 |
| | Monosloped Configurations (Zone 1) | | Enclosed | Partially Enc. |
| | Flat < Slope < 7:12 ( 30°) | | | |
| | Positive | 10 SF or less | 0.49 | 0.81 |
| | | 100 SF or more | 0.41 | 0.72 |
| | Negative | 10 SF or less | -1.26 | -1.57 |
| | | 100 SF or more | -1.09 | -1.40 |
| | Tall flat topped roofs h> 60' | | Enclosed | Partially Enc. |
| | Flat <slope < 2:12 (10°) (Zone 1) | | | |
| | Negative | 10 SF or less | -1.34 | -1.66 |
| | | 500 SF or more | -1.00 | -1.32 |
| 3. Components and Cladding in areas of discontinuities – Roofs and overhangs | Roof Elements and slopes | | Enclosed | Partially Enc. |
| | Gable or Hipped Configurations at Ridges, Eaves and Rakes (Zone 2) | | | |
| | Flat < Slope < 6:12 ( 27°) | | | |
| | Positive | 10 SF or less | 0.58 | 0.89 |
| | | 100 SF or more | 0.41 | 0.72 |
| | Negative | 10 SF or less | -1.68 | -2.00 |
| | | 100 SF or more | -1.17 | -1.49 |

| | | | | |
|---|---|---|---|---|
| | Overhang for Slope Flat < Slope < 6:12 ( 27°) | | | |
| | Negative | 10 SF or less | -1.87 | |
| | | 100 SF or more | -1.87 | |
| | 6:12 (27°) < Slope < 12:12 ( 45°) | | Enclosed | Partially Enc. |
| | Positive | 10 SF or less | 0.92 | 1.23 |
| | | 100 SF or more | 0.83 | 1.15 |
| | Negative | 10 SF or less | -1.17 | -1.49 |
| | | 100 SF or more | -1.00 | -1.32 |
| | Overhang for 6:12 ( 27°) < Slope < 12:12 ( 45°) | | | |
| | Negative | 10 SF or less | -1.70 | |
| | | 100 SF or more | -1.53 | |
| | Monosloped Configurations at Ridges, Eaves and Rakes (Zone 2) | | | |
| | Flat < Slope < 7:12 ( 30°) | | | |
| | Positive | 10 SF or less | 0.49 | 0.81 |
| | | 100 SF or more | 0.41 | 0.72 |
| | Negative | 10 SF or less | -1.51 | -1.83 |
| | | 100 SF or more | -1.43 | -1.74 |
| | Tall flat topped roofs h> 60' | | Enclosed | Partially Enc. |
| | Flat <slope < 2:12 (10°) (Zone 2) | | | |
| | Negative | 10 SF or less | -2.11 | -2.42 |
| | | 500 SF or more | -1.51 | -1.83 |
| | Gable or Hipped Configurations at Corners (Zone 3) | | | |
| | Flat < Slope < 6:12 ( 27°) | | Enclosed | Partially Enc. |
| | Positive | 10 SF or less | 0.58 | 0.89 |
| | | 100 SF or more | 0.41 | 0.72 |
| | Negative | 10 SF or less | -2.53 | -2.85 |
| | | 100 SF or more | -1.85 | -2.17 |
| | Overhang for Slope Flat < Slope < 6:12 ( 27°) | | | |
| | Negative | 10 SF or less | -3.15 | |

| | | | | |
|---|---|---|---|---|
| | | 100 SF or more | -2.13 | |
| | 6:12 (27°) < Slope < 12:12 ( 45°) | | | |
| | Positive | 10 SF or less | 0.92 | 1.23 |
| | | 100 SF or more | 0.83 | 1.15 |
| | Negative | 10 SF or less | -1.17 | -1.49 |
| | | 100 SF or more | -1.00 | -1.32 |
| | Overhang for 6:12 ( 27°) < Slope < | | Enclosed | Partially Enc. |
| | Negative | 10 SF or less | -1.70 | |
| | | 100 SF or more | -1.53 | |
| | Monosloped Configurations at corners (Zone 3) | | | |
| | Flat < Slope < 7:12 ( 30°) | | | |
| | Positive | 10 SF or less | 0.49 | 0.81 |
| | | 100 SF or more | 0.41 | 0.72 |
| | Negative | 10 SF or less | -2.62 | -2.93 |
| | | 100 SF or more | -1.85 | -2.17 |
| | Tall flat topped roofs h> 60' | | Enclosed | Partially Enc. |
| | Flat <slope < 2:12 (10°) (Zone 3) | | | |
| | Negative | 10 SF or less | -2.87 | -3.19 |
| | | 500 SF or more | -2.11 | -2.42 |
| 4. Components and Cladding not in areas of discontinuity - Walls and parapets | Wall Elements: h ≤ 60' (Zone 4) | | Enclosed | Partially Enc. |
| | Positive | 10 SF or less | 1.00 | 1.32 |
| | | 500 SF or more | 0.75 | 1.06 |
| | Negative | 10 SF or less | -1.09 | -1.40 |
| | | 500 SF or more | -0.83 | -1.15 |
| | Wall Elements: h > 60' (Zone 4) | | | |
| | Positive | 20 SF or less | 0.92 | 1.23 |
| | | 500 SF or more | 0.66 | 0.98 |
| | Negative | 20 SF or less | -0.92 | -1.23 |
| | | 500 SF or more | -0.75 | -1.06 |

| | | | | |
|---|---|---|---|---|
| | Parapet Walls | | | |
| | Positive | | 2.87 | 3.19 |
| | Negative | | -1.68 | -2.00 |
| 5. Components and Cladding in areas of discontinuity - Walls and parapets | Wall Elements: h ≤ 60' (Zone 5) | | Enclosed | Partially Enc. |
| | Positive | 10 SF or less | 1.00 | 1.32 |
| | | 500 SF or more | 0.75 | 1.06 |
| | Negative | 10 SF or less | -1.34 | -1.66 |
| | | 500 SF or more | -0.83 | -1.05 |
| | Wall Elements: h > 60' (Zone 5) | | | |
| | Positive | 20 SF or less | 0.92 | 1.23 |
| | | 500 SF or more | 0.66 | 0.98 |
| | Negative | 20 SF or less | -1.68 | -2.00 |
| | | 500 SF or more | -1.00 | -1.32 |
| | Parapet Walls | | | |
| | Positive | | 3.64 | 3.95 |
| | Negative | | -2.45 | -2.76 |

a. Linear interpolation between values in the table is acceptable.
b. For open buildings, multispan gable roofs, stepped roofs, sawtooth roofs, domed roofs, solid free standing walls and solid signs apply ASCE 7.
c. Some $C_{net}$ values have been grouped together. Less conservative results may be obtained by applying ASCE 7.

**Reason:** The all heights wind provisions of ASCE 7 are time consuming and confusing. Many engineers make significant errors in their use of this method. There is a simplified method in ASCE 7, but it is limited in use. Member Organizations of NCSEA have brought forward an alternate method which is in full compliance with ASCE 7. This method is being considered by the ASCE 7 Wind Committee, but it won't be able to be placed in the standard until the 2012 IBC is adopted. To speed this transition, this method is proposed for the IBC first.

The derivation of this method from ASCE 7 Chapter 6 is as follows:
$C_{net}$ values
$q_z = 0.00256\ K_z\ K_{zt}\ K_d\ V^2\ I$ Eqn 6-15
$p = q\ G\ C_p - q_i\ (GC_{pi})$ Eqn 6-17

$p = 0.00256\ K_h\ K_{zt}\ K_d\ V^2\ I\ G\ C_p - 0.00256\ K_z\ K_{zt}\ K_d\ V^2\ I\ (GC_{pi})$

Rearranging terms:
$p = (\ 0.00256\ V^2\ K_h\ K_d\ G\ C_p - 0.00256\ V^2\ K_z\ K_d\ (GC_{pi}))\ K_{zt}\ I$

Define: $q_z = 0.00256\ V^2$
so: $p = (q_s\ K_h\ K_d\ G\ C_p - q_s\ K_z\ K_d\ (GC_{pi}))\ K_{zt}\ I$
and: $p = q_s\ K_d\ (\ K_h\ G\ C_p - K_z\ (GC_{pi}))\ K_{zt}\ I$

For leeward wall and roof elements
$K_h = K_z$
so: $p = q_s\ K_z\ (\ K_d\ (G\ C_p - (GC_{pi})))\ K_{zt}\ I$
Substitute $C_{net} = K_d\ (G\ C_p - (GC_{pi}))$
and we get: $p = q_s\ K_z\ C_{net}\ K_{zt}\ I$

which is Eqn. 16-xx in the draft.
For windward roof elements
$K_h \approx K_z$ and the same relationship holds.

For buildings: $K_d$ = 0.85
For rigid structures: G = 0.85
so: $C_{net}$ = 0.85 (0.85 $C_p$ − $(GC_{pi})$)

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

## Committee Action: Approved as Modified

**Modify proposal as follows:**

**1609.6.1 Scope.** As an alternate to ASCE 7 Section 6.5, the following provisions are permitted to be used to determine the wind effects on regularly shaped buildings, or other structures which meet all of the following conditions:

1. The building or other structure is less than or equal to 75 ~~100~~ feet (~~30480~~ 22 860 mm) in height, with a height to least width ratio of 4 or less, or the building or other structure has a fundamental frequency greater than or equal to 1 hertz.
2. The building or other structure is not sensitive to dynamic effects.
3. The building or other structure is not located on a site for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.
4. The building shall meet the requirements of a simple diaphragm building as defined in ASCE 7 Section 6.2.

**TABLE 1609.6.2(2)**
**NET PRESSURE COEFFICIENTS, $C_{net}$ [a, b, c]**

| STRUCTURE OR PART THEREOF | DESCRIPTION | | $C_{net}$ FACTOR | |
|---|---|---|---|---|
| 1. Main Wind Force Resisting Frames and Systems | WALLS: | | Enclosed | Part Enclosed |
| | Windward Wall | | 0.43 | 0.11 |
| | Leeward Wall | | ~~-0.53~~ -0.51 | -0.83 |
| | Side Wall | | -0.66 | -0.97 |
| | Parapet Wall | Windward | 1.28 | 1.28 |
| | | Leeward | -0.85 | -0.85 |
| | | | | |
| | ROOFS: | | Enclosed | Part Enclosed |
| | Wind perpendicular to ridge | | | |
| | Leeward roof or flat roof | | -0.66 | -0.97 |
| | Windward roof slopes: | | | |
| | Slope < 2:12 ( 10°) | Case 1 | -1.09 | -1.41 |
| | | Case 2 | -0.28 | -0.60 |
| | Slope = 4:12 ( 18°) | Case 1 | -0.73 | -1.04 |
| | | Case 2 | -0.05 | -0.37 |
| | Slope = 5:12 ( 23°) | Case 1 | -0.58 | -0.90 |
| | | Case 2 | 0.03 | -0.29 |
| | Slope = 6:12 ( 27°) | Case 1 | -0.47 | -0.78 |
| | | Case 2 | ~~0.20~~ 0.06 | ~~0.51~~ -0.25 |
| | Slope = 7:12 ( 30°) | Case 1 | -0.37 | -0.68 |
| | | Case 2 | ~~0.30~~ 0.07 | ~~0.61~~ -0.25 |
| | Slope 9:12 ( 37°) | Case 1 | -0.27 | -0.58 |
| | | Case 2 | ~~0.31~~ 0.14 | ~~0.63~~ -0.18 |
| | Slope 12:12 ( 45°) | | ~~0.37~~ 0.14 | ~~0.68~~ -0.18 |
| | Wind parallel to ridge and flat roofs | | -1.09 | -1.41 |

(Portions of proposal not shown remain unchanged)

**Committee Reason:** This proposal provides a needed alternative that addresses concerns over the complexity of the ASCE 7 Method 2 wind analysis. Exam committees have considered this complexity to be the primary cause in the disparity in answers given to test questions on Method 2. Adding this now is similar to the approach taken when adding the simplified wind method to the SBC and the 2000 IBC, which prompted the ASCE 7 committee to get it into their document. With the floor modifications this proposal is superior to S85-07/08. The modification places additional limitations on buildings that can qualify for this method of analysis, by eliminating flexible buildings and requiring simple diaphragm building. These address some of the concerns that were raised in connection with S85-07/08.

There were still some concerns with assumptions regarding internal pressurization. The resulting values are not necessarily conservative for Components and Cladding as well as MWFRS for some nonsymmetrical configurations. The same comment concerning the use of ASCE 7 figures that is noted on S85-07/08 applies to this change as well. It would have been preferable to include necessary figures in this proposal. There is also a concern with diaphragms. The method should probably be limited to rigid diaphragms. It is possible that horizontal truss elements could be unconservatively designed on the windward side because of the preference given to maximum negative values in the selection of these coefficients. The same concern in hurricane prone regions discussed in S85-07/08 applies to this proposal as well. The scope of application is too broad. There are possible issues down the road if we've begun using these values and ASCE 7 introduces a version of this method which provides different values.

**Assembly Action:** **None**

## *Individual Consideration Agenda*

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

*Public Comment 1:*

**Edwin Huston, National Council of Structural Engineers Association (NCSEA), representing NCSEA Code Advisory Committee, requests Approval as Modified by this Public Comment.**

**Further modify proposal as follows:**

**TABLE 1609.6.2(2)**
**NET PRESSURE COEFFICIENTS, $C_{NET}$[a,b,c]**

| STRUCTURE OR PART THEREOF | DESCRIPTION | | $C_{net}$ FACTOR | | | |
|---|---|---|---|---|---|---|
| 1. Main Wind Force Resisting Frames and Systems | | | Enclosed | | Partially Enclosed | |
| | WALLS: | | + Internal Pressure | - Internal Pressure | + Internal Pressure | - Internal Pressure |
| | Windward Wall | | 0.43 | 0.73 | 0.11 | 1.05 |
| | Leeward Wall | | -0.51 | -0.21 | -0.83 | 0.11 |
| | Side Wall | | -0.66 | -0.35 | -0.97 | -0.04 |
| | Parapet Wall | Windward | 1.28 | | 1.28 | |
| | | Leeward | -0.85 | | -0.85 | |
| | ROOFS: | | Enclosed | | Partially Enclosed | |
| | Wind perpendicular to ridge | | + Internal Pressure | - Internal Pressure | + Internal Pressure | - Internal Pressure |
| | Leeward roof or flat roof | | -0.66 | -0.35 | -0.97 | -0.04 |
| | Windward roof slopes: | | | | | |
| | Slope < 2:12 ( 10°) | Case 1 | -1.09 | -0.79 | -1.41 | -0.47 |
| | | Case 2 | -0.28 | 0.02 | -0.60 | 0.34 |
| | Slope = 4:12 ( 18°) | Case 1 | -0.73 | -0.42 | -1.04 | -0.11 |
| | | Case 2 | -0.05 | 0.25 | -0.37 | 0.57 |
| | Slope = 5:12 ( 23°) | Case 1 | -0.58 | -0.28 | -0.90 | 0.04 |
| | | Case 2 | 0.03 | 0.34 | -0.29 | 0.65 |
| | Slope = 6:12 ( 27°) | Case 1 | -0.47 | -0.16 | -0.78 | 0.15 |
| | | Case 2 | 0.06 | 0.37 | -0.25 | 0.68 |
| | Slope = 7:12 ( 30°) | Case 1 | -0.37 | -0.06 | -0.68 | 0.25 |
| | | Case 2 | 0.07 | 0.37 | -0.25 | 0.69 |
| | Slope 9:12 ( 37°) | Case 1 | -0.27 | 0.04 | -0.58 | 0.35 |
| | | Case 2 | 0.14 | 0.44 | -0.18 | 0.76 |
| | Slope 12:12 ( 45°) | | 0.14 | 0.44 | -0.18 | 0.76 |
| | Wind parallel to ridge and flat roofs | | -1.09 | -0.79 | -1.41 | -0.47 |
| | - | | | | | |

(Portions of table not shown remain unchanged)

a) Linear interpolation between values in the table is acceptable.
b) For open buildings, multispan gable roofs, stepped roofs, sawtooth roofs, domed roofs, solid free standing walls and solid signs apply ASCE 7.
c) Some $C_{net}$ values have been grouped together. Less conservative results may be obtained by applying ASCE 7.

**(Portions of proposal not shown remain unchanged)**

**Commenter's Reason:** At the Code Change Hearings in Palm Springs, Mr. Chock, who served on the ICC Structural Committee, expressed some concerns which are printed in the second paragraph of the Committee Reason Statement in the Report on the Hearings. NCSEA asked for more detailed information on these concerns and we were invited to attend an ASCE/SEI 7 Wind Load Subcommittee meeting which was held in San Francisco, CA on March 12. Subsequent to that meeting Mr. Chock forwarded a list of technical concerns from ASCE/SEI 7 to NCSEA.

On June 6, 2008 NCSEA again attended an ASCE/SEI 7 Wind Load Subcommittee meeting which was held at ASCE Headquarters in Reston, VA to present a response to those technical concerns. Based on the discussions from that meeting, NCSEA has drafted this Public Comment to address a substantial technical concern from ASCE/SEI 7. This Public comment adds another set of pressure coefficients to include the case of negative internal pressurization of the building for the Main-Wind-Force-Resisting System.

In the NCSEA reason statement included with S84, the following statement appears:

The all heights wind provisions of ASCE 7 are time consuming and confusing. Many engineers make significant errors in their use of this method. There is a simplified method in ASCE 7, but it is limited in use. Member Organizations of NCSEA have brought forward an alternate method which is in full compliance with ASCE 7. This method is being considered by the ASCE 7 Wind Committee, but it won't be able to be placed in the standard until the 2012 IBC is adopted. To speed this transition, this method is proposed for the IBC first.

At the time this Code Change Proposal was submitted that statement was correct. Since that time, ASCE/SEI 7 has begun consideration of a different format for wind simplification. If this proposal is adopted by the Wind Load Subcommittee and the main committee it would appear in the ASCE/SEI 7-10 Standard and could be considered for adoption into the 2012 IBC. In the meantime, modification of S84 will permit an alternative wind load methodology, which is in full compliance with the Method 2 procedure for all-heights, to be included in the 2009 IBC.

*Public Comment 2:*

## James Riemenschneider, Structural Engineers Association of Oregon (SEAO)/Kramer Gehlen & Associates, representing, Structural Engineers Association of Oregon (SEAO), Wind Committee requests Approval as Modified by this Public Comment.

**Further modify proposal as follows:**

**1609.6.4.4.1 Components and Cladding**. Wind pressure for each component or cladding element is applied as follows using $C_{net}$ values based on the effective wind area, A contained within the zones in areas-of-discontinuity of width and/or length "a", "2a" or "4a" at: corners of roofs and walls; edge strips for ridges, rakes and eaves; or field areas on walls or roofs as indicated in Figures in the Tables in ASCE 7 as referenced in Table 1609.6.2(2) in accordance with the following:

1. Calculated pressures at local discontinuities acting over specific edge strips or corner boundary areas.
2. Include "field" (zone 1, 2 or 4, as applicable) pressures applied to areas beyond the boundaries of the areas-of-discontinuity.
3. Where applicable, the calculated pressures at discontinuities (zones 2 or 3) shall be combined with design pressures that apply specifically on rakes or eave overhangs.

**TABLE 1609.6.2(2) – NET PRESSURE COEFFICIENTS, $C_{NET}$[a,b,c]**

| STRUCTURE OR PART THEREOF | DESCRIPTION | | $C_{net}$ FACTOR | |
|---|---|---|---|---|
| 1. Main Wind Force Resisting Frames and Systems | WALLS: | | Enclosed | Partially Enclosed |
| | Windward Wall | | 0.43 | 0.11 |
| | Leeward Wall | | -0.51 | -0.83 |
| | Side Wall | | -0.66 | -0.97 |
| | Parapet Wall | Windward | 1.28 | 1.28 |
| | | Leeward | -0.85 | -0.85 |
| | | | | |
| | ROOFS: | | Enclosed | Partially Enclosed |
| | Wind perpendicular to ridge | | | |
| | Leeward roof or flat roof | | -0.66 | -0.97 |
| | Windward roof slopes: | | | |
| | Slope < 2:12 ( 10°) | Case 1 | -1.09 | -1.41 |
| | | Case 2 | -0.28 | -0.60 |
| | Slope = 4:12 ( 18°) | Case 1 | -0.73 | -1.04 |
| | | Case 2 | -0.05 | -0.37 |
| | Slope = 5:12 ( 23°) | Case 1 | -0.58 | -0.90 |
| | | Case 2 | 0.03 | -0.29 |
| | Slope = 6:12 ( 27°) | Case 1 | -0.47 | -0.78 |
| | | Case 2 | 0.06 | -0.25 |
| | Slope = 7:12 ( 30°) | Case 1 | -0.37 | -0.68 |
| | | Case 2 | 0.07 | -0.25 |

| | | | | | |
|---|---|---|---|---|---|
| | Slope 9:12 ( 37°) | Case 1 | -0.27 | -0.58 | |
| | | Case 2 | 0.14 | -0.18 | |
| | Slope 12:12 ( 45°) | | 0.14 | -0.18 | |
| | Wind parallel to ridge and flat roofs | | -1.09 | -1.41 | |
| | - | | | | |
| | Non Building Structures: Chimneys, Tanks and Similar Structures: | | | | |
| | | | h/D | | |
| | | | 1 | 7 | 25 |
| | Square (Wind normal to face) | | 0.99 | 1.07 | 1.53 |
| | Square (Wind on diagonal) | | 0.77 | 0.84 | 1.15 |
| | Hexagonal or Octagonal | | 0.81 | 0.97 | 1.13 |
| | Round | | 0.65 | 0.81 | 0.97 |
| | | | | | |
| | Open Signs and Lattice Frameworks | | Ratio of solid to gross area | | |
| | | | < 0.1 | 0.1 to 0.29 | 0.3 to 0.7 |
| | Flat | | 1.45 | 1.30 | 1.16 |
| | Round | | 0.87 | 0.94 | 1.08 |
| | | | | | |
| | | | | | |
| | | | | | |
| 1. Components and cladding not in areas of discontinuity – Roofs and overhangs | Roof Elements and slopes | | Enclosed | Partially Enclosed | |
| | Gable or Hipped Configurations (Zone 1) | | | | |
| | Flat < Slope < 6:12 ( 27°) See ASCE 7 Figure 6-11C Zone 1 | | | | |
| | Positive | 10 SF or less | 0.58 | 0.89 | |
| | | 100 SF or more | 0.41 | 0.72 | |
| | Negative | 10 SF or less | -1.00 | -1.32 | |
| | | 100 SF or more | -0.92 | -1.23 | |
| | Overhang: Flat < Slope < 6:12 ( 27°) See ASCE 7 Figure 6-11B Zone 1 | | | | |
| | Negative | 10 SF or less | -1.45 | | |
| | | 100 SF or more | -1.36 | | |
| | | 500 SF or more | -0.94 | | |
| | 6:12 (27°) < Slope < 12:12 ( 45°) See ASCE 7 Figure 6-11D Zone 1 | | | | |
| | Positive | 10 SF or less | 0.92 | 1.23 | |
| | | 100 SF or more | 0.83 | 1.15 | |
| | Negative | 10 SF or less | -1.00 | -1.32 | |
| | | 100 SF or more | -0.83 | -1.15 | |

| | | | Enclosed | Partially Enclosed |
|---|---|---|---|---|
| | Monosloped Configurations (Zone 1) | | Enclosed | Partially Enclosed |
| | Flat < Slope < 7:12 ( 30°) See ASCE 7 Figure 6-14B Zone 1 | | | |
| | Positive | 10 SF or less | 0.49 | 0.81 |
| | | 100 SF or more | 0.41 | 0.72 |
| | Negative | 10 SF or less | -1.26 | -1.57 |
| | | 100 SF or more | -1.09 | -1.40 |
| | Tall flat topped roofs h> 60’ | | Enclosed | Partially Enclosed. |
| | Flat <slope < 2:12 (10°) (Zone 1) See ASCE 7 Figure 6-17 Zone 1 | | | |
| | Negative | 10 SF or less | -1.34 | -1.66 |
| | | 500 SF or more | -0.92 | -1.23 |
| 2. Components and cladding in areas of discontinuities – Roofs and overhangs | Roof Elements and slopes | | Enclosed | Partially Enclosed. |
| | Gable or Hipped Configurations at Ridges, Eaves and Rakes (Zone 2) | | | |
| | Flat < Slope < 6:12 ( 27°) See ASCE 7 Figure 6-11C Zone 2 | | | |
| | Positive | 10 SF or less | 0.58 | 0.89 |
| | | 100 SF or more | 0.41 | 0.72 |
| | Negative | 10 SF or less | -1.68 | -2.00 |
| | | 100 SF or more | -1.17 | -1.49 |
| | Overhang for Slope Flat < Slope < 6:12 ( 27°) See ASCE 7 Figure 6-11C Zone 2 | | | |
| | Negative | 10 SF or less | -1.87 | |
| | | 100 SF or more | -1.87 | |
| | 6:12 (27°) < Slope < 12:12 ( 45°) Table 6-11D | | Enclosed | Partially Enclosed |
| | Positive | 10 SF or less | 0.92 | 1.23 |
| | | 100 SF or more | 0.83 | 1.15 |
| | Negative | 10 SF or less | -1.17 | -1.49 |
| | | 100 SF or more | -1.00 | -1.32 |
| | Overhang for 6:12 ( 27°) < Slope < 12:12 ( 45°) See ASCE 7 Figure 6-11D Zone 2 | | | |
| | Negative | 10 SF or less | -1.70 | |
| | | 100 SF or more | -1.53 | |

| | | | | |
|---|---|---|---|---|
| | Monosloped Configurations at Ridges, Eaves and Rakes (Zone 2) | | | |
| | Flat < Slope < 7:12 ( 30°) See ASCE 7 Figure 6-14B Zone 2 | | | |
| | Positive | 10 SF or less | 0.49 | 0.81 |
| | | 100 SF or more | 0.41 | 0.72 |
| | Negative | 10 SF or less | -1.51 | -1.83 |
| | | 100 SF or more | -1.43 | -1.74 |
| | Tall flat topped roofs h> 60’ | | Enclosed | Partially Enclosed |
| | Flat <slope < 2:12 (10°) (Zone 2) See ASCE 7 Figure 6-17 Zone 2 | | | |
| | Negative | 10 SF or less | -2.11 | -2.42 |
| | | 500 SF or more | -1.51 | -1.83 |
| | Gable or Hipped Configurations at Corners (Zone 3) See ASCE 7 Figure 6-11C Zone 3 | | | |
| | Flat < Slope < 6:12 ( 27°) | | Enclosed | Partially Enclosed |
| | Positive | 10 SF or less | 0.58 | 0.89 |
| | | 100 SF or more | 0.41 | 0.72 |
| | Negative | 10 SF or less | -2.53 | -2.85 |
| | | 100 SF or more | -1.85 | -2.17 |
| | Overhang for Slope Flat < Slope < 6:12 ( 27°) See ASCE 7 Figure 6-11C Zone 3 | | | |
| | Negative | 10 SF or less | -3.15 | |
| | | 100 SF or more | -2.13 | |
| | 6:12 (27°) < Slope < 12:12 ( 45°) See ASCE 7 Figure 6-11D Zone 3 | | | |
| | Positive | 10 SF or less | 0.92 | 1.23 |
| | | 100 SF or more | 0.83 | 1.15 |
| | Negative | 10 SF or less | -1.17 | -1.49 |
| | | 100 SF or more | -1.00 | -1.32 |
| | Overhang for 6:12 ( 27°) < Slope < 12:12 ( 45°) | | Enclosed | Partially Enclosed. |
| | Negative | 10 SF or less | -1.70 | |
| | | 100 SF or more | -1.53 | |
| | Monosloped Configurations at corners (Zone 3) See ASCE 7 Figure 6-14B Zone 3 | | | |

| | | | | |
|---|---|---|---|---|
| | Flat < Slope < 7:12 ( 30°) | | | |
| | Positive | 10 SF or less | 0.49 | 0.81 |
| | | 100 SF or more | 0.41 | 0.72 |
| | Negative | 10 SF or less | -2.62 | -2.93 |
| | | 100 SF or more | -1.85 | -2.17 |
| | | | | |
| | Tall flat topped roofs h> 60' | | Enclosed | Partially Enclosed |
| | Flat <slope < 2:12 (10°) (Zone 3) See ASCE 7 Figure 6-17 Zone 3 | | | |
| | Negative | 10 SF or less | -2.87 | -3.19 |
| | | 500 SF or more | -2.11 | -2.42 |
| 3. Components and Cladding not in areas of discontinuity - Walls and parapets | Wall Elements: h ≤ 60' (Zone 4) Table 6-11A | | Enclosed | Partially Enclosed |
| | Positive | 10 SF or less | 1.00 | 1.32 |
| | | 500 SF or more | 0.75 | 1.06 |
| | Negative | 10 SF or less | -1.09 | -1.40 |
| | | 500 SF or more | -0.83 | -1.15 |
| | Wall Elements: h > 60' (Zone 4) See ASCE 7 Figure 6-17 Zone 4 | | | |
| | Positive | 20 SF or less | 0.92 | 1.23 |
| | | 500 SF or more | 0.66 | 0.98 |
| | Negative | 20 SF or less | -0.92 | -1.23 |
| | | 500 SF or more | -0.75 | -1.06 |
| | Parapet Walls | | | |
| | Positive | | 2.87 | 3.19 |
| | Negative | | -1.68 | -2.00 |
| 4. Components and Cladding in areas of discontinuity - Walls and parapets | Wall Elements: h ≤ 60' (Zone 5) Table 6-11A | | Enclosed | Partially Enclosed |
| | Positive | 10 SF or less | 1.00 | 1.32 |
| | | 500 SF or more | 0.75 | 1.06 |
| | Negative | 10 SF or less | -1.34 | -1.66 |
| | | 500 SF or more | -0.83 | -1.05 |
| | Wall Elements: h > 60' (Zone 5) See ASCE 7 Figure 6-17 Zone 4 | | | |
| | Positive | 20 SF or less | 0.92 | 1.23 |

| | | | | |
|---|---|---|---|---|
| | | 500 SF or more | 0.66 | 0.98 |
| | Negative | 20 SF or less | -1.68 | -2.00 |
| | | 500 SF or more | -1.00 | -1.32 |
| | Parapet Walls | | | |
| | Positive | | 3.64 | 3.95 |
| | Negative | | -2.45 | -2.76 |

a. Linear interpolation between values in the table is acceptable.
b. For open buildings, multispan gable roofs, stepped roofs, sawtooth roofs, domed roofs, solid free standing walls and solid signs apply ASCE 7.
c. Some $C_{net}$ values have been grouped together. Less conservative results may be obtained by applying ASCE 7.

(Portions of proposal not shown remain unchanged)

**Commenter's Reason:** The S84-07/08 code change proposal stated incorrectly that the zones and thus the definitions of the "a", "2a", or 4"a" distance over which they apply were in Table 1609.6.2(2). They are not. This Public comment corrects the text in Section 1609.6.4.4.1 and adds the appropriate references to the Figures in ASCE 7 in Table 1609.6.2(2).

*Public Comment 3:*

## Don Scott, Structural Engineers of Washington (SEAW), representing SEAW Wind Engineering Committee, requests Approval as Modified by this Public Comment.

**Further modify proposal as follows:**

**1609.6.1 Scope:** As an alternate to ASCE 7 Section 6.5, the following provisions are permitted to be used to determine the wind effects on regularly shaped buildings, or regularly shaped other structures which meet all of the following conditions:

1. The building or other structure is less than or equal to 75 feet in height, with a height to least width ratio of 4 or less, or the building or other structure has a fundamental frequency greater than or equal to 1 hertz.
2. The building or other structure is not sensitive to dynamic effects.
3. The building or other structure is not located on a site for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.
4. The building shall meet the requirements of a simple diaphragm building as defined in ASCE 7 Section 6.2, where wind loads are only transmitted to the Main-Wind-Force-Resisting System at the diaphragms.
5. For open buildings, multispan gable roofs, stepped roofs, sawtooth roofs, domed roofs, roofs with slopes greater than 45°, solid free standing walls and solid signs, and roof top equipment apply ASCE 7.

**1609.6.2 Symbols and Notations**. Coefficients and variables used in the Alternate All-Heights Method equations are as follows:

$C_{net}$ = net-pressure coefficient based on $K_d$ [(G) ($C_p$) – ($GC_{pi}$)], Ref Table 1609.6.2(2)
G = Gust effect factor ~~equal to 0.85~~ for rigid structures per ASCE 7 Section 6.5.8.1.
$K_d$ = Wind directionality factor per ASCE 7 Table 6-4.
$P_{net}$ = Design wind pressure to be used in determination of wind loads on buildings or other structures or their components and cladding, in lb/ft$^2$ (N/m$^2$).
$q_s$ = Wind ~~velocity~~ stagnation pressure in lb/ft$^2$ (N/m$^2$). (Per Table 1609.6.2(1))

**1609.6.4.3 Determination of Net Pressure Coefficients, $C_{net}$.** For the design of the Main Wind-Force-Resisting-System and for Components and Cladding, the sum of the internal and external net pressure shall be based on the net pressure coefficient $C_{net}$.

1. The pressure coefficient, $C_{net,}$ for walls and roofs shall be determined from Table 1609.6.2(2).
2. Where $C_{net}$ may have more than one value, the more severe wind load ~~combination~~ condition shall be used for design.

**TABLE 1609.6.2(2)**
**NET PRESSURE COEFFICIENTS, $C_{NET}$[a,b,c]**

| STRUCTURE OR PART THEREOF | DESCRIPTION | | $C_{net}$ FACTOR | | |
|---|---|---|---|---|---|
| 1.Main Wind Force Resisting Frames and Systems | WALLS: | | Enclosed | Part Enclosed | |
| | Windward Wall | | 0.43 | 0.11 | |
| | Leeward Wall | | -0.51 | -0.83 | |
| | Side Wall | | -0.66 | -0.97 | |
| | Parapet Wall | Windward | 1.28 | 1.28 | |
| | | Leeward | -0.85 | -0.85 | |
| | | | | | |
| | ROOFS: | | Enclosed | Part Enclosed | |
| | Wind perpendicular to ridge | | | | |
| | Leeward roof or flat roof | | -0.66 | -0.97 | |
| | Windward roof slopes: | | | | |
| | Slope < 2:12 ( 10°) | ~~Case~~ Condition 1 | -1.09 | -1.41 | |
| | | ~~Case~~ Condition 2 | -0.28 | -0.60 | |
| | Slope = 4:12 ( 18°) | ~~Case~~ Condition 1 | -0.73 | -1.04 | |
| | | ~~Case~~ Condition 2 | -0.05 | -0.37 | |
| | Slope = 5:12 ( 23°) | ~~Case~~ Condition 1 | -0.58 | -0.90 | |
| | | ~~Case~~ Condition 2 | 0.03 | -0.29 | |
| | Slope = 6:12 ( 27°) | ~~Case~~ Condition 1 | -0.47 | -0.78 | |
| | | ~~Case~~ Condition 2 | 0.06 | -0.25 | |
| | Slope = 7:12 ( 30°) | ~~Case~~ Condition 1 | -0.37 | -0.68 | |
| | | ~~Case~~ Condition 2 | 0.07 | -0.25 | |
| | Slope 9:12 ( 37°) | ~~Case~~ Condition 1 | -0.27 | -0.58 | |
| | | ~~Case~~ Condition 2 | 0.14 | -0.18 | |
| | Slope 12:12 ( 45°) | | 0.14 | -0.18 | |
| | Wind parallel to ridge and flat roofs | | -1.09 | -1.41 | |
| | - | | | | |
| | Non Building Structures: Chimneys, Tanks and Similar Structures: | | | | |
| | | | h/D | | |
| | | | 1 | 7 | 25 |
| | Square (Wind normal to face) | | 0.99 | 1.07 | 1.53 |
| | Square (Wind on diagonal) | | 0.77 | 0.84 | 1.15 |
| | Hexagonal or Octagonal | | 0.81 | 0.97 | 1.13 |
| | Round | | 0.65 | 0.81 | 0.97 |
| | | | | | |
| | Open Signs and Lattice Frameworks | | Ratio of solid to gross area | | |
| | | | < 0.1 | 0.1 to 0.29 | 0.3 to 0.7 |
| | Flat | | 1.45 | 1.30 | 1.16 |
| | Round | | 0.87 | 0.94 | 1.08 |
| | | | | | |
| | | | | | |
| | | | | | |
| 2. Components and Cladding not in areas of discontinuity – Roofs and overhangs | Roof Elements and slopes | | Enclosed | Partially Enc. | |
| | Gable or Hipped Configurations (Zone 1) | | | | |
| | Flat < Slope < 6:12 ( 27°) | | | | |

| | | | | |
|---|---|---|---|---|
| | Positive | 10 SF or less | 0.58 | 0.89 |
| | | 100 SF or more | 0.41 | 0.72 |
| | Negative | 10 SF or less | -1.00 | -1.32 |
| | | 100 SF or more | -0.92 | -1.23 |
| | Overhang: Flat < Slope < 6:12 ( 27°) | | | |
| | Negative | 10 SF or less | -1.45 | |
| | | 100 SF or more | -1.36 | |
| | | 500 SF or more | -0.94 | |
| | 6:12 (27°) < Slope < 12:12 ( 45°) | | | |
| | Positive | 10 SF or less | 0.92 | 1.23 |
| | | 100 SF or more | 0.83 | 1.15 |
| | Negative | 10 SF or less | -1.00 | -1.32 |
| | | 100 SF or more | -0.83 | -1.15 |
| | Monosloped Configurations (Zone 1) | | Enclosed | Partially Enc. |
| | Flat < Slope < 7:12 ( 30°) | | | |
| | Positive | 10 SF or less | 0.49 | 0.81 |
| | | 100 SF or more | 0.41 | 0.72 |
| | Negative | 10 SF or less | -1.26 | -1.57 |
| | | 100 SF or more | -1.09 | -1.40 |
| | Tall flat topped roofs h> 60' | | Enclosed | Partially Enc. |
| | Flat <slope < 2:12 (10°) (Zone 1) | | | |
| | Negative | 10 SF or less | -1.34 | -1.66 |
| | | 500 SF or more | -1.00 | -1.32 |
| 3.Components and Cladding in areas of discontinuities – Roofs and overhangs | Roof Elements and slopes | | Enclosed | Partially Enc. |
| | Gable or Hipped Configurations at Ridges, Eaves and Rakes (Zone 2) | | | |
| | Flat < Slope < 6:12 ( 27°) | | | |
| | Positive | 10 SF or less | 0.58 | 0.89 |

| | | | | |
|---|---|---|---|---|
| | | 100 SF or more | 0.41 | 0.72 |
| | Negative | 10 SF or less | -1.68 | -2.00 |
| | | 100 SF or more | -1.17 | -1.49 |
| | Overhang for Slope Flat < Slope < 6:12 ( 27°) | | | |
| | Negative | 10 SF or less | -1.87 | |
| | | 100 SF or more | -1.87 | |
| | 6:12 (27°) < Slope < 12:12 ( 45°) | | Enclosed | Partially Enc. |
| | Positive | 10 SF or less | 0.92 | 1.23 |
| | | 100 SF or more | 0.83 | 1.15 |
| | Negative | 10 SF or less | -1.17 | -1.49 |
| | | 100 SF or more | -1.00 | -1.32 |
| | Overhang for 6:12 ( 27°) < Slope < 12:12 ( 45°) | | | |
| | Negative | 10 SF or less | -1.70 | |
| | | 100 SF or more | -1.53 | |
| | Monosloped Configurations at Ridges, Eaves and Rakes (Zone 2) | | | |
| | Flat < Slope < 7:12 ( 30°) | | | |
| | Positive | 10 SF or less | 0.49 | 0.81 |
| | | 100 SF or more | 0.41 | 0.72 |
| | Negative | 10 SF or less | -1.51 | -1.83 |
| | | 100 SF or more | -1.43 | -1.74 |
| | Tall flat topped roofs h> 60' | | Enclosed | Partially Enc. |
| | Flat <slope < 2:12 (10°) (Zone 2) | | | |
| | Negative | 10 SF or less | -2.11 | -2.42 |
| | | 500 SF or more | -1.51 | -1.83 |
| | Gable or Hipped Configurations at Corners (Zone 3) | | | |
| | Flat < Slope < 6:12 ( 27°) | | Enclosed | Partially Enc. |
| | Positive | 10 SF or less | 0.58 | 0.89 |
| | | 100 SF or more | 0.41 | 0.72 |
| | Negative | 10 SF or less | -2.53 | -2.85 |

| | | | | |
|---|---|---|---|---|
| | | 100 SF or more | -1.85 | -2.17 |
| | Overhang for Slope Flat < Slope < 6:12 ( 27°) | | | |
| | Negative | 10 SF or less | -3.15 | |
| | | 100 SF or more | -2.13 | |
| | 6:12 (27°) < Slope < 12:12 ( 45°) | | | |
| | Positive | 10 SF or less | 0.92 | 1.23 |
| | | 100 SF or more | 0.83 | 1.15 |
| | Negative | 10 SF or less | -1.17 | -1.49 |
| | | 100 SF or more | -1.00 | -1.32 |
| | Overhang for 6:12 ( 27°) < Slope < 12:12 ( 45°) | | Enclosed | Partially Enc. |
| | Negative | 10 SF or less | -1.70 | |
| | | 100 SF or more | -1.53 | |
| | Monosloped Configurations at corners (Zone 3) | | | |
| | Flat < Slope < 7:12 ( 30°) | | | |
| | Positive | 10 SF or less | 0.49 | 0.81 |
| | | 100 SF or more | 0.41 | 0.72 |
| | Negative | 10 SF or less | -2.62 | -2.93 |
| | | 100 SF or more | -1.85 | -2.17 |
| | | | | |
| | Tall flat topped roofs h> 60' | | Enclosed | Partially Enc. |
| | Flat <slope < 2:12 (10°) (Zone 3) | | | |
| | Negative | 10 SF or less | -2.87 | -3.19 |
| | | 500 SF or more | -2.11 | -2.42 |
| 4. Components and Cladding not in areas of discontinuity – Walls and parapets | Wall Elements: h ≤ 60' (Zone 4) | | Enclosed | Partially Enc. |
| | Positive | 10 SF or less | 1.00 | 1.32 |
| | | 500 SF or more | 0.75 | 1.06 |
| | Negative | 10 SF or less | -1.09 | -1.40 |
| | | 500 SF or more | -0.83 | -1.15 |
| | Wall Elements: h > 60' (Zone 4) | | | |

| | | | | |
|---|---|---|---|---|
| | Positive | 20 SF or less | 0.92 | 1.23 |
| | | 500 SF or more | 0.66 | 0.98 |
| | Negative | 20 SF or less | -0.92 | -1.23 |
| | | 500 SF or more | -0.75 | -1.06 |
| | Parapet Walls | | | |
| | Positive | | 2.87 | 3.19 |
| | Negative | | -1.68 | -2.00 |
| 5.Components and Cladding in areas of discontinuity - Walls and parapets | Wall Elements: h ≤ 60' (Zone 5) | | Enclosed | Partially Enc. |
| | Positive | 10 SF or less | 1.00 | 1.32 |
| | | 500 SF or more | 0.75 | 1.06 |
| | Negative | 10 SF or less | -1.34 | -1.66 |
| | | 500 SF or more | -0.83 | -1.~~05~~15 |
| | Wall Elements: h > 60' (Zone 5) | | | |
| | Positive | 20 SF or less | 0.92 | 1.23 |
| | | 500 SF or more | 0.66 | 0.98 |
| | Negative | 20 SF or less | -1.68 | -2.00 |
| | | 500 SF or more | -1.00 | -1.32 |
| | Parapet Walls | | | |
| | Positive | | 3.64 | 3.95 |
| | Negative | | -2.45 | -2.76 |

a. Linear interpolation between values in the table is acceptable.
~~b. For open buildings, multispan gable roofs, stepped roofs, sawtooth roofs, domed roofs, solid free standing walls and solid signs apply ASCE 7.~~
~~c.~~ b. Some $C_{net}$ values have been grouped together. Less conservative results may be obtained by applying ASCE 7.

(Portions of proposal not shown remain unchanged)

**Commenter's Reason:** At meetings in March and June with the ASCE/SEI 7 Wind Load Subcommittee, several improvements to Code Change Proposal S84 were discussed. These include:

- Adding the words regularly shaped to other structures in 1609.6.1 to avoid misapplication
- Adding the words where wind loads are only transmitted to the Main-Wind-Force-Resisting System at the diaphragms. To Item 4 in 1609.6.1 to avoid misapplication. This change is under consideration by the ASCE/SEI 7 Wind Load Subcommittee and will appear in ASCE/SEI 7-10.
- Moving and expanding Footnote 2 to Table 1609.6.2(2) to Item 5 in 1609.6.1 to make it more visable.
- Changing the definition of $q_s$ in 1609.6.2 from to Wind velocity pressure to Wind stagnation pressure to avoid confusion with ASCE/SEI 7.
- Changing the words wind load combination in Item 2 of 1609.6.4.3 to wind load condition to avoid confusion with the load combinations of Section 1605. In Table 1609.6.2(2) the Case 1 and Case 2 categories for wind perpendicular to a ridge were also changed from ~~case~~ to condition for the same reason.
- In Table 1609.6.2(2) Item 5. Components and Cladding in areas of discontinuity - Walls and parapets, the value for Wall Elements: h ≤ 60' (Zone 5), negative pressure for 500 SF or more for a Partially Enclosed condition was changed from 1.05 to 1.15 to correct a minor typographical mistake.

*Public Comment 4:*

**James A. Rossberg, SEI of ASCE, requests Disapproval**

**Commenter's Reason:** The Structural Engineering Institute of the American Society of Civil Engineers (SEI/ASCE) requests that the ICC membership disapprove S 84-07/08 for the following reasons:

Technical errors: During the hearings in Palm Springs in March of 2008, members of the ICC Structural Committee identified several technical errors with S84 which were subsequently transmitted to the proponent. In early June, the ASCE 7 Wind Load Subcommittee evaluated proposal S84 in detail and identified additional technical issues with S84, which, if not addressed, would result in unconservative wind pressures on the Main Wind Force Resisting System. There are also types of buildings and structures where the S84 procedure would yield inappropriate results, and thus should not be included.

SEI/ASCE has brought the technical issues of the ICC Structural Committee and the ASCE 7 Wind Load Subcommittee to the attention of representatives of NCSEA. One of the errors is principally editorial in nature relating to referencing of the appropriate figures in ASCE/SEI 7 to be used in conjunction with S84; without this modification, S84 would be incomplete.

A second, and much more substantial error, requires that further limitations be introduced to the proposed provisions to exclude types of buildings and structures that cannot be correctly calculated using this procedure. The assumptions underlying the proposed provisions necessitates that they be limited in use to simple diaphragm buildings; the same as the simplified method already referenced by the IBC. For the proposed alternative simplified provisions to be properly used it is crucial that the external wind loads be transferred into the MWFRS solely via the diaphragms. (The ASCE 7 Wind Load Subcommittee is clarifying this requirement in ASCE 7).

The third, and most substantial technical error with S84, requires the addition of a new series of pressure coefficients to address the case of negative internal pressurization of the building. This loading case, which may often be the controlling load case for windward walls, for windward roofs of slope greater than 25º and for moment-resisting frames, was ignored in S84. We note that ASCE/SEI 7 includes this case, and its commentary includes specific warning of the importance of this load case.

SEI/ASCE considers the correction of these last two errors to be essential to the technical validity of the proposed provisions so that they produce loads that are approximately equivalent or conservative relative to the ASCE/SEI 7 wind loads that are presently referenced in the International Building Code. Conversely, if these two changes are not made, S84 can produce inadequately low calculated pressures that do not comply with the ASCE/SEI 7 Standard for Main Wind Force Resisting members in certain buildings and structures.

Accordingly, SEI/ASCE strongly urges disapproval of proposal S84 unless these two critical modifications are included. Otherwise, structural engineers will be making significant errors in wind load calculations.

The ASCE 7 standards committee is in the process of completing several more comprehensive improvements in ease of use and simplification of the wind load provisions to be included the next edition of the ASCE/SEI 7 Standard and which will be offered for adoption into the IBC.

*Public Comment 5:*

**Can Xiao, City of Phoenix, AZ, representing herself, requests Disapproval**

**Commenter's Reason:** This proposed method requires a lot of reference back to ASCE 7-05 and make designers jump back and forth between IBC code and ASCE 7-05, making it not that simple. This proposal was also opposed by ASCE 7-05 and lacks of evidence to prove that it is going to be equivalent or more conservative to method 2. Furthermore, ASCE 7-05 stated that they are working on a simplified wind analysis method and soon will be adopted a part of ASCE7.

*Public Comment 6:*

**John Woestman, The Kellen Company, Window and Door Manufacturers Association (WDMA), requests Disapproval**

**Commenter's Reason:** The Window and Door Manufacturers Association (WDMA) recommends Disapproval for code proposal S84-07/08.

WDMA is not debating the technical merits of the proposed Alternative All-heights Method. However, WDMA strongly cautions that technical details of this magnitude should remain in the reference standard and should not be incorporated into code language. Approving this code proposal, before these details are resolved in the ASCE 7 technical arena and are ready to be adopted in the IBC by reference, will likely bring to the ICC code development arena technical debate that is much better addressed in ASCE 7 standard development

Final Action: AS AM AMPC____ D

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# S88-07/08

## 1612.3.2 (New)

*Proposed Change as Submitted:*

**Proponent:** Rebecca C. Quinn, R.C. Quinn Consulting, Inc., representing US Department of Homeland Security, Federal Emergency Management Agency

**Revise as follows:**

**1612.3.1 (Supp) Design flood elevations.** Where design flood elevations are not included in the flood hazard areas established in Section 1612.3, or where floodways are not designated, the building official is authorized to require the applicant to:

1. Obtain and reasonably utilize any design flood elevation and floodway data available from a federal, state or other source; or
2. Determine the design flood elevation and/or floodway in accordance with accepted hydrologic and hydraulic engineering practices used to define special flood hazard areas. Determinations shall be undertaken by a registered design professional who shall document that the technical methods used reflect currently accepted engineering practice.

**1612.3.2 Determination of impacts.** In riverine flood hazard areas where design flood elevations are specified but floodways have not been designated, the applicant shall demonstrate that the cumulative effect of the proposed buildings and structures, when combined with all other existing and anticipated flood hazard area encroachments, will not increase the design flood elevation more than 1 foot (305 mm) at any point within the jurisdiction of the applicable governing authority.

**Reason:** The purpose of this code change is to improve consistency with the requirements of the National Flood Insurance Program (NFIP) regarding development in flood hazard areas where base (or design) flood elevations are shown on the Flood Insurance Rate Map, but analyses to delineate the floodway were not performed. Development in riverine floodplains can increase flood levels and loads on other properties, especially if it occurs in areas known as floodways that must be reserved to convey flood flows. The floodway, as defined in 1612.2, is the area along riverine waterways that "must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height." For the situation addressed by this code change, the designed height is one foot.

Similar language appears in four locations in the I-Codes: (1) IBC 1803.4(4) to address proposed grading and filling; (2) IBC Appendix G103.4; (3) IBC Appendix J101.2; and IRC R324.1.3.2. The requirement to determine cumulative impacts has been part of the NFIP for more than 20 years and has been administered by more than 20,000 local jurisdictions that participate in the NFIP.

References:
Title 44 Code of Federal Regulations Parts 59 and 60, Regulations for Floodplain Management and Flood Hazard Identification." Online at http://www.fema.gov/business/nfip/laws1.shtm.

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

**Committee Action:** **Disapproved**

**Committee Reason:** While this may already be a part of the federal regulations, the committee felt it includes ill-defined terminology. Particularly the phrase "anticipated flood hazard areas" leaves too much to interpretation.

**Assembly Action:** **None**

## *Individual Consideration Agenda*

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

*Public Comment:*

**Rebecca C. Quinn, RCQuinn Consulting, Inc, representing Department of Homeland Security/Federal Emergency Management Agency, requests Approval as Modified by this Public Comment.**

**Replace proposal as follows:**

**1612.3.2 Determination of impacts**. In riverine flood hazard areas where design flood elevations are specified but floodways have not been designated, the applicant shall provide a floodway analysis that demonstrates that the proposed work will not increase the design flood elevation more than 1 foot (305 mm) at any point within the jurisdiction of the applicable governing authority.

**Commenter's Reason:** This public comment modifies the code change by replacing the phrase considered to leave too much to interpretation with a statement that a floodway analysis is required to determine impacts. Development in riverine floodplains can increase flood levels and loads on other properties, especially if it occurs in areas known as floodways that must be reserved to convey flood flows. Floodways are the channels and those portions of riverine floodplains that convey most of the volume of floodwaters. Floodway analyses have been performed for decades. Commercial software packages for these analyses are readily available and FEMA provides software and technical guidance at www.fema.gov/plan/prevent/fhm/frm_soft.shtm#1.

The intent of the proposal is to improve consistency with the National Flood Insurance Program which requires applicants to demonstrate whether their proposed work will increase flood levels in floodplains where the NFIP's Flood Insurance Rate Map shows Base Flood Elevations, but floodways are not shown. A small percentage of floodplains where FEMA has specified Base Flood Elevations do not have designated floodways.

Final Action: AS AM AMPC____ D

# S95-07/08
## 1613.7 (New), 1613.7.1 (New), 1613.7.2 (New)

*Proposed Change as Submitted:*

**Proponent:** Jim Messersmith, Jr., PE, Portland Cement Association

**Add new text as follows:**

**1613.7 General**. The text of ASCE 7 shall be modified as indicated in Sections 1613.7.1 through 1613.7.2.

**1613.7.1 ASCE 7, Section 12.11.2**. Modify ASCE 7, Section 12.11.2 to read as follows:

**12.11.2 Anchorage of Concrete and Masonry Structural Walls.** The anchorage of concrete or masonry structural walls to supporting construction shall provide a direct connection capable of resisting the force set forth in Section 12.11.1.

Structural walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 ft (1,219 mm).

**1613.7.2 ASCE 7, Section 12.14.7.5.** Modify ASCE 7, Section 12.14.7.5 to read as follows:

**12.14.7.5 Anchorage of Concrete and Masonry Structural Walls**. Concrete or masonry structural walls shall be anchored to all floors, roofs and members that provide out-of-plane lateral support for the wall or that are supported by the wall. The anchorage shall provide a positive direct connection between the wall and floor, roof or supporting member with the strength to resist horizontal forces specified in this section for structures with flexible diaphragms or of Section 13.3.1 (using $a_p$ *and* $R_p$ equal to 2.5) for structures with diaphragms that are not flexible.

Anchorage of structural walls to flexible diaphragms shall have the strength to develop the out-of-plane force given by Eq. 12.14.10:

$$F_p = 0.8S_{DS}W_p \qquad (12.14\text{-}10)$$

Where:

$F_p$ = the design force in the individual anchor
$S_{DS}$ = the design spectral response acceleration at short periods per Section 12.14.8.1
$W_p$ = the weight of the wall tributary to the anchor

> **Exception:** For Seismic Design Category B, the coefficient shall be 0.4, with a minimum force of 10 percent of the tributary weight of the wall.

**Reason:** Since ASCE 7-05 will be the loading standard referenced in the 2009 IBC, the only recourse to getting changes made to ASCE 7-05 is through modifications to the standard within the IBC.

The requirement that anchors attaching concrete and masonry walls to supporting construction be designed for a lower bound value of 280 plf or $400S_{DS}I$ plf, whichever is greater, is excessive and discriminatory considering that anchorage of walls of other materials, regardless of their mass, is required to be designed for a horizontal force of $0.40S_{DS}I$ or 10%, whichever is greater, times the weight of the wall tributary to the anchor.

To illustrate the punitive nature of the provision, consider two 10-foot high walls that are representative of walls used in single family dwellings and small commercial buildings; one a 5.5-inch thick concrete wall, the other a light-framed wall with 4-inch nominal masonry veneer anchored to the wall framing. For this example we'll assume that $S_{DS}$ equals 0.32 (SDC B). The weight of the light-framed wall, including veneer is estimated to be 45 psf; therefore, the weight tributary to an anchor at the top of the wall is 225 plf (45 * (10/2) = 225). Therefore, the required design anchorage force for the light-framed wall is 29 plf (0.40 * 0.32 * 1 * 225 = 29). For a 5.5-inch concrete wall, which weighs approximately 69 psf, the weight tributary to an anchor at the top of the wall is 345 plf (69 * (10/2) = 345). Based on the requirement that applies to walls of other than concrete or masonry, the required anchorage design force for the concrete wall should be 44 plf (0.40 * 0.32 * 1 * 345 = 44); however, 280 plf must be used to design the anchorage.

Let's examine how the 280 plf requirement compares to wind design. ASCE 7, Section 6.1.4.2 requires that components and cladding be designed for a minimum service level design wind pressure of 10 psf. For our example walls using this minimum design wind pressure, the factored force at the top of the wall due to wind is 80 plf (1.6 * 10 * (10/2) = 80). Now let's determine what basic wind speed is required to produce a design force at the top of the wall of 280 plf (strength level). The service level (unfactored) force equal to 280 plf is 175 plf (280/1.6 = 175). Since 175 plf is based on a tributary wall height of 5 feet, the unfactored design wind pressure is 35 psf (175/5 = 35). From ASCE 7, Figure 6-3, for a building in exposure B, height of 30 feet, $K_{zt}$ of 1.0, and effective wind area of less than or equal to 10 square feet, for wall area 4 the negative design pressure for a basic wind speed of 140 mph is 38.2 psf. Therefore, the requirement that the 5.5-inch, 10-foot high concrete wall be anchored against a force of 280 plf is the same as requiring that the connection be designed for a basic wind speed of approximately 135 mph in exposure B.

Next we'll examine the second criterion that the design anchorage force be not less than $400S_{DS}I$ plf. By setting the two criteria equal to each other, it can be determined that the 280 plf criterion controls at values of $S_{DS}$ less than 0.70 ($S_{DS}$ = 280/400 = 0.70) for I equal to 1.0. For values of $S_{DS}$ greater than 0.70, $400S_{DS}I$ governs. If we're designing a connection between the concrete wall and a rigid diaphragm in a building where $S_{DS}$ equals 1.0, the design force will be 400 plf; whereas, without the special criteria for concrete and masonry walls, the design force would be 138 plf (0.40 * 1 * 1 * 345 = 138). In other words, the design force for a concrete or masonry wall is 2.9 times greater than it is for another type of wall with the same mass.

If the concrete wall is being connected to a flexible diaphragm in a building where $S_{DS}$ equals 1.0, the special provisions of Section 12.11.2.1 apply. This requires that the design force for the anchor be $0.80S_{DS}I$ times the weight of the wall tributary to the anchor. In the case of the concrete wall, this will require the anchor design force to be 276 plf (0.80 * 1 * 1 * 345 = 276); however, this too is less than 280, which is less than 400. Therefore, the design force for connections of the concrete wall to a flexible or rigid diaphragm where $S_{DS}$ equals 1.0 is the same (i.e., 400 plf).

Another aspect of these high connection design forces that should be considered is the force for which the wall itself must be designed. ASCE 7, Section 12.11.1 requires that all structural walls in buildings assigned to SDC B or higher, regardless of materials of construction, be designed for a force normal to surface equal to $0.40S_{DS}I$ or 10%, whichever is greater, times the weight of the wall. Again considering the concrete wall cited above in a building where $S_{DS}$ equals 0.32, the wall will be designed for a lateral force of 9 psf (0.40 * 0.32 * 1 * 69 = 9). On the other hand, the connection between the top of the wall and the diaphragm or other laterally supporting element must be designed for 280 plf. Since the height of the wall tributary to the connection is 5 feet, this suggests that the design force on the wall is 56 psf (280/5 = 56), or over 6 (56/9 = 6.22) times the force for which the wall is actually designed. One has to question the logic of requiring the connection to be designed for a force that is so much greater than the design force for the wall, given that the load factor on E is 1.0.

There are three other factors that need to be considered. First, many anchors that resist wall out-of-plane forces also must be designed to resist other forces at the same time. In fact, an anchor may be resisting vertical and horizontal shear forces in addition to the out-of-plane tensile force. Second, Appendix D of ACI 318 (Anchoring to Concrete) requires the anchor design strength be reduced 25% in structures assigned to Seismic Design Category C and higher. This is the same as requiring that the anchor design force be increased by 33-1/3%. This increase is in addition to the higher forces imposed by the requirements of ASCE 7 that apply only to concrete and masonry walls. Third, Appendix D of ACI 318 requires that in structures assigned to Seismic Design Category C and higher, the anchor design strength must be based on the failure of a ductile steel element. This means that the design strength based on all the concrete failure modes must be greater than the design strength based on the steel anchor. The only way to get around this is to increase the required anchor design force by 2.5 times per the IBC modification to ACI 318 Section D.3.3.5 in IBC Section 1908.1.16. All of these requirements are compounded by these minimum forces of ASCE 7 that apply only to concrete and masonry walls.

It is obvious that in a world that is rapidly embracing performance-based design, the requirement that anchorages for concrete and masonry walls be designed for a force of 280 or $400S_{DS}I$ pounds per linear foot of wall, whichever is greater, without considering the mass of the wall tributary to the anchor is not necessary and discriminates against these materials. In addition, by singling our walls of concrete and masonry, walls of other materials that could have and equal or greater mass per unit area are exempt from the requirement. Based on the foregoing, the anchorage force requirement for concrete and masonry walls in items "b" and "c" of Section 12.11.2 should be deleted, as should the provision in the exception to Section 12.14.7.5.

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

**Committee Action: Disapproved**

**Committee Reason:** The affected ASCE 7 provision is a structural integrity issue and the committee prefers to keep the current ASCE 7 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:*

**Joseph J. Messersmith, Jr., P.E., Portland Cement Association, requests Approval as Submitted.**

**Commenter's Reason:** The reason cited for disapproving this change indicates that "the affected ASCE 7 provision is a structural integrity issue and the committee prefers to keep the current ASCE 7 provision intact." This is not a structural integrity issue, but rather a question of how much design force is enough. The current requirements of ASCE 7 specify design forces for taller, heavier walls of concrete or masonry that are derived from a technically correct equation that considers ground acceleration and the mass of the wall. On the other hand, as illustrated in the reason for the proposal, short, lighter walls of concrete or masonry are required to be designed for arbitrary lower bound forces that in some cases are several times greater than the forces derived from the same technically correct equation.

Although we are in general agreement that consensus standards should be used to the fullest extent without modification in the I-codes, there are situations where modifications should be and have been made to standards. Two examples from the Palm Springs hearings stand out; S57-07/08 and S96-07/08 make modifications to provisions of ASCE 7 regarding anchorage of concrete and masonry walls for buildings assigned to Seismic Design Category A. In the reason given for the committee approving S57, it is indicated that "the minimum horizontal force of 280 pounds per foot that has been carried over from legacy codes is arbitrary and overly conservative." In these case, the committee felt that modifications to ASCE 7 were justified. They are just as justified for buildings assigned to Seismic Design Category B and higher, which this proposal addresses.

Since the Palm Springs hearings, an ad hoc committee within the ASCE 7 Seismic Subcommittee has been reviewing this subject. While the committee is in agreement that the existing lower bound values of ASCE 7 are not justified, they have not been able to come to agreement on an alternative approach.

Based on the foregoing, I urge you to vote against the floor motion to disapprove the change, and vote for a subsequent floor motion to approve the change as submitted.

Final Action: AS AM AMPC____ D

# S96-07/08

## 1613.7 (New), 1613.7.1 (New)

*Proposed Change as Submitted:*

**Proponent:** Jim Messersmith, Jr. PE, Portland Cement Association

**Add new text as follows:**

**1613.7 General.** The text of ASCE 7 shall be modified as indicated in Section 1613.7.1.

**1613.7.1 ASCE 7, Section 11.7.5.** Modify ASCE 7, Section 11.7.5 to read as follows:

**11.7.5 Anchorage of walls.** Walls shall be anchored to the roof and all floors and members that provide lateral support for the wall or that are supported by the wall. The anchorage shall provide a direct connection between the walls and the roof or floor construction. The connections shall be capable of resisting the forces specified in Section 11.7.3 applied horizontally, substituted for *E* in load combinations of Section 2.3 or 2.4.

**Reason:** Since ASCE 7-05 will be the loading standard referenced in the 2009 IBC, the only recourse to getting changes made to ASCE 7-05 is through modifications to the standard within the IBC.

The requirement that anchors attaching concrete and masonry walls to supporting construction be designed for a lower bound value of 280 pounds per linear foot is excessive and discriminatory considering that anchorage of walls of other materials, regardless of their mass, is required to be designed for a horizontal force of 5% of the weight of the wall tributary to the anchor.

To illustrate the punitive nature of the provision, consider two 10-foot high walls that are representative of walls used in single family dwellings and small commercial buildings; one a 5.5-inch thick concrete wall, the other a light-framed wall with 4-inch nominal masonry veneer anchored to the wall framing. The weight of the light-framed wall, including veneer, is estimated to be 45 psf; therefore, the weight tributary to an anchor at the top of the wall is 225 plf (45 * (10/2) = 225). The required design anchorage force for this wall is 11 plf (225 * 0.05 = 11). On the other hand, for the 5.5-inch concrete wall, which weighs 69 psf, the weight tributary to an anchor at the top of the wall is 345 plf (69 * (10/2) = 345). Based on the requirement that applies to walls of other than concrete or masonry, the required anchorage design force for the concrete wall should be 17 plf (345 * 0.05 = 17); however, 280 plf must be used to design the anchorage.

Let's examine how the 280 plf requirement compares to wind design. ASCE 7, Section 6.1.4.2 requires that components and cladding be designed for a minimum service level design wind pressure of 10 psf. For our example walls using this minimum design wind pressure, the strength level (factored) force at the top of the wall due to wind is 80 plf (10 * (10/2) * 1.6 = 80). Now let's determine what basic wind speed is required to produce a factored design force at the top of the wall of 280 plf (strength level). The service level (unfactored) force comparable to 280 plf is 175 plf (280/1.6 = 175). Since 175 plf is based on a tributary wall height of 5 feet, the design wind pressure is 35 psf (175/5 = 35). From ASCE 7, Figure 6-3, for a building in exposure B, height of 30 feet, $K_{zt}$ of 1.0, and effective wind area of less than or equal to 10 square feet for wall area 4, the negative design pressure for a basic wind speed of 140 mph is 38.2 psf. Therefore, the requirement that the 5.5-inch, 10-foot high concrete wall be anchored against a force of 280 plf is the same as requiring that the connection be designed for a basic wind speed of approximately 135 mph.

It is obvious that in a world that is rapidly embracing performance-based design, the requirement that anchorages for concrete and masonry walls be designed for a force of 280 plf is not necessary and discriminates against these products. In addition, by singling out walls of concrete and masonry, walls of other materials that could have comparable mass per unit area are exempt from the requirement. Based on the foregoing, the requirement that anchorages for concrete and masonry walls be designed for 280 plf should be deleted.

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

**Committee Action:** **Approved as Submitted**

**Committee Reason:** The committee believes this is a reasonable change to the ASCE 7 requirement for anchorage of walls and it is consistent with the committee's action on S57-07/08.

**Assembly Action:** **None**

*Individual Consideration Agenda*

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

*Public Comment:*

**Can Xiao, City of Phoenix, AZ, representing herself, requests Disapproval.**

**Commenter's Reason:** The 280plf minimum horizontal force design was intent to improve the connections between masonry/concrete walls and floor/roof diaphragm. This is a structural detailing requirement that was intent to achieve "strong connection weak members" seismic design concept. Because masonry/concrete walls have larger self weight, they tend to attract more seismic forces during earthquake. It is important to make sure that the connections between horizontal diaphragm and masonry/concrete walls are strong enough to withstand the seismic forces attracted by masonry/concrete walls, so the building will not collapse and cause human being casualties. This requirement was actually based on several post-earthquake investigations where most damages were found to occur. This requirement is a very important seismic detailing requirement that is for protecting life safety. It shall not be removed from the building code solely because of economic reason. The No.1 priority of building code is to protect public life safety. This priority can not be compromised by economic reason at any time.

The similar proposal to ASCE 7 has been disapproved. ASCE 7 is one of the national standards that are referred by IBC. By approving this code proposal, it makes IBC to be conflicting with ASCE 7. This will definitely create difficulties and problems in code enforcement.

Final Action: AS AM AMPC____ D

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# S98-07/08

## 1613.7 (New), 1613.7.1 (New)

*Proposed Change as Submitted:*

**Proponent:** Jim Messersmith, Jr., PE, Portland Cement Association

**Add new text as follows:**

**1613.7 General.** The text of ASCE 7 shall be modified as indicated in Section 1613.7.1.

**1613.7.1 ASCE 7, Section 12.14.7.5.** Modify ASCE 7, Sections 12.14.7.5.1 through 12.14.7.5.4 to read as follows:

**12.14.7.5.1 Transfer of anchorage forces into diaphragm.** In buildings assigned to Seismic Design Category C, D or E, diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorage forces into the diaphragm. 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 the structural subdiaphragm 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.

**12.14.7.5.2 Wood diaphragms.** In buildings assigned to Seismic Design Category C, D or E, in wood diaphragms, the continuous ties shall be in addition to the diaphragm sheathing. Anchorage shall not be accomplished by use of toenails or nails subject to withdrawal nor shall wood ledgers or framing be used in cross-gain bending or cross-grain tension. The diaphragm sheathing shall not be considered effective as providing the ties or struts required by this section.

**12.14.7.5.3 Metal deck diaphragms.** In buildings assigned to Seismic Design Category C, D or E, in metal deck diaphragms, the metal deck shall not be used as the continuous ties required by this section in the direction perpendicular to the deck span.

**12.14.7.5.4 Embedded straps.** In buildings assigned to Seismic Design Category C, D or E, diaphragm to wall anchorage using embedded straps shall be attached to or hooked around the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

**Reason:** Since ASCE 7-05 will be the loading standard referenced in the 2009 IBC, the only recourse to getting changes made to ASCE 7-05 is through modifications to the standard within the IBC.

The provisions in Sections 12.14.7.5.1, 12.14.7.5.2, 12.14.7.5.3 and 12.14.7.5.4 (Simplified Alternate Structural Design Criteria …) were extracted from Sections 12.11.2.2.1, 12.11.2.2.3, 12.11.2.2.4 and 12.11.2.2.5, respectively. In the latter sections the provisions apply to buildings assigned to Seismic Design Category C and higher; whereas, the identical provisions in Section 12.14.7.5 apply to buildings of SDC B and higher. While conservatism is warranted in some cases where simplified provisions are used, applying the provisions in question to buildings assigned to SDC B is not justified. Use of simplified provisions should be encouraged by eliminating unnecessary requirements, rather than discouraging their use by adding requirements that do not apply where regular procedures are used.

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

**Committee Action:** **Disapproved**

**Committee Reason:** The committee prefers that this proposal for diaphragm anchorage forces be addressed by the ASCE 7 committee.

**Assembly Action:** **None**

*Individual Consideration Agenda*

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

*Public Comment:*

**Joseph J. Messersmith, Jr. PE., Portland Cement Association, requests Approval as Submitted.**

**Commenter's Reason: T**he provisions in Sections 12.14.7.5.1, 12.14.7.5.2, 12.14.7.5.3 and 12.14.7.5.4 (Simplified Alternate Structural Design Criteria …) of ASCE 7-05 were extracted from Sections 12.11.2.2.1, 12.11.2.2.3, 12.11.2.2.4 and 12.11.2.2.5, respectively, of ASCE 7-05. The latter sections apply to buildings assigned to Seismic Design Category (SDC) C and higher; whereas, the identical provisions in Section 12.14.7.5 apply to buildings assigned to SDC B and higher.

The provisions of Section 12.14.7.5.1 are especially onerous since continuous ties or struts are required to extend across the building length and width at each wall anchor. In the direction of the main framing members (i.e., joists, trusses, etc.), these members can be used as the continuous ties; however, in the direction perpendicular to the main framing members, steel straps continuous across the building length are generally required to comply with this requirement since wood structural sheathing is not permitted to serve as continuous ties (see Section 12.14.7.5.2).

The requirement for continuous ties originated in the UBC and eventfully found its way into the NEHRP Provisions and ASCE 7. In the UBC, these provisions applied to Seismic Zones 2, 3 and 4 (areas of moderate and high seismic risk). When incorporated into the NEHRP Provisions and ASCE 7, they applied to buildings assigned to SDC C, D, E or F (building at moderate or high seismic risk). When the simplified design procedure (SDP) of the 2003 NEHRP Provisions was incorporated into ASCE 7-05, the provisions were extended to apply to buildings assigned to SDC B (buildings at low seismic risk).

"Based on default site class D soil, buildings of Occupancy Categories I, II and III in a significant portion of the eastern US and Rocky Mountain area are assigned to SDC B (low seismic risk)". These additional requirements, which are not justified for buildings assigned to SDC B, will discourage the use of the SDP. Use of simplified provisions should be encouraged rather than discouraged by adding requirements that do not apply where the regular design procedures of ASCE 7 are used.

Based on the foregoing, I urge you to vote against the floor motion to disapprove the change, and vote for a subsequent floor motion to approve the change as submitted.

Final Action: AS AM AMPC____ D

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# S101-07/08
## 1614 (New)

*Proposed Change as Submitted:*

**Proponent:** Ronald O. Hamburger, SE, Simpson Gumpertz & Heger, Inc, representing National Council of Structural Engineers Associations/Ad Hoc Joint Industry Committee on Structural Integrity

**Add new section as follows:**

**SECTION 1614**
**STRUCTURAL INTEGRITY**

**1614.1 General.** Buildings and other structures assigned to Occupancy Category II, III, or IV, exceeding three stories above grade plane shall comply with the requirements of this section. Frame structures shall comply with the requirements of Section 1614.3. Bearing wall structures shall comply with the requirements of Section 1614.4.

**Exception:** Structures other than buildings with structural systems that are not like building structures including, but not limited to, billboards, signs, silos, tanks, stacks, mechanical and electrical equipment.

**1614.2 Definitions.** The following words and terms shall, for the purposes of Section 1614, have the meanings shown herein.

**BEARING WALL STRUCTURE**. A building or other structure in which vertical loads from floors and roofs are primarily supported by walls.

**FRAME STRUCTURE.** A building or other structure in which vertical loads from floors and roofs are primarily supported by columns.

**1614.3 Frame structures.** Frame structures shall comply with the requirements of this section.

**1614.3.1 Concrete frame structures.** Frame structures constructed primarily of reinforced or prestressed concrete, either cast-in-place or precast, or a combination of these, shall conform to the requirements of ACI 318 Sections 7.13, 13.3.8.5, 13.3.8.6, 16.5 and 18.12.6, b18.12.7 and 18.12.8 as applicable. Where ACI 318 requires that nonprestressed reinforcing or prestressing steel pass through the region bounded by the longitudinal column reinforcement, that reinforcing or prestressing steel shall have a minimum nominal tensile strength equal to 2/3 of the required one-way vertical strength of the connection of the floor or roof system to the column in each direction of beam or slab reinforcement passing through the column.

**Exception:** Where concrete slabs with continuous reinforcing having an area not less than 0.0015 times the concrete area in each of two orthogonal directions are present and are either monolithic with or equivalently bonded to beams, girders or columns, the longitudinal reinforcing or prestressing steel passing through the column reinforcement shall have a nominal tensile strength of 1/3 of the required one-way vertical strength of the connection of the floor or roof system to the column in each direction of beam or slab reinforcement passing through the column.

**1614.3.2 Structural steel, open web steel joist or joist girder, or composite steel and concrete frame structures.** Frame structures constructed with a structural steel frame or a frame composed of open web steel joists, joist girders with or without other structural steel elements or a frame composed of composite steel or composite steel joists and reinforced concrete elements shall conform to the requirements of this section.

**1614.3.2.1 Columns.** Each column splice shall have the minimum design strength in tension to transfer the design dead and live load tributary to the column between the splice and the splice or base immediately below.

**1614.3.2.2 Beams**. End connections of all beams and girders shall have a minimum nominal axial tensile strength equal to the required vertical shear strength for Allowable Strength Design (ASD) or 2/3 of the required shear strength for Load and Resistance Factor Design (LRFD) but not less than 10 kips (45 kN). For the purpose of this section, the shear force and the axial tensile force need not be considered to act simultaneously.

**Exception:** Where beams, girders, open web joist, and joist girders support a concrete slab or concrete slab on metal deck that is attached to the beam or girder with not less than 3/8 in. (9.5 mm) diameter headed shear studs, at a spacing of not more than 12 in. (305 mm) on center, averaged over the length of the member, or other attachment having equivalent shear strength, and the slab contains continuous distributed reinforcement in each of two orthogonal directions with an area not less than 0.0015 times the concrete area, the nominal axial tension strength of the end connection shall be permitted to be taken as half the required vertical shear strength for ASD or 1/3 of the required shear strength for LRFD, but not less than 10 kips (45 kN).

**1614.4 Bearing wall structures**. Bearing wall structures shall have vertical ties in all load bearing walls and longitudinal ties, transverse ties, and perimeter ties at each floor level in accordance with this section and as shown in Figure 1614.4.

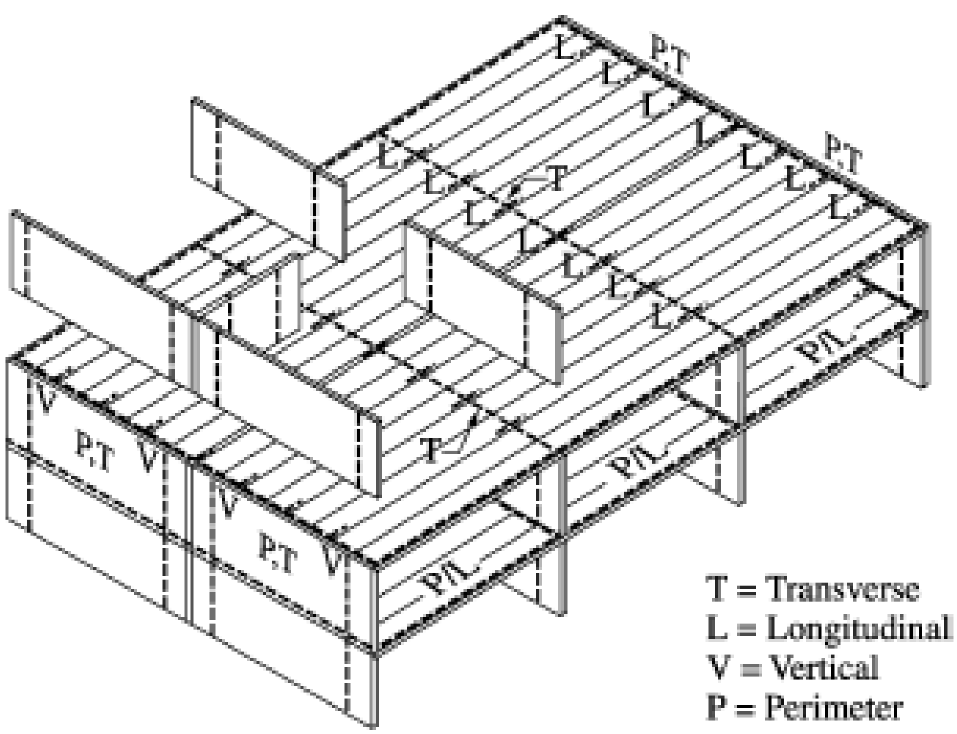


**FIGURE 1614.4**
**LONGITUDINAL, PERIMETER, TRANSVERSE AND VERTICAL TIES**

**1614.4.1 Concrete wall structures**. Precast bearing wall structures constructed solely of reinforced or prestressed concrete, or combinations of these shall conform to the requirements of Sections 7.13, 13.3.8.5 and 16.5 of ACI 318.

**1614.4.2 Other bearing wall structures.** Ties in bearing wall structures other than those covered in Section 1614.4.1 shall conform to this section.

**1614.4.2.1 Longitudinal ties**. Longitudinal ties shall consist of continuous reinforcement in slabs; continuous or spliced decks or sheathing; continuous or spliced members framing to, within, or across walls; or, connections of continuous framing members to walls. Longitudinal ties shall extend across interior load bearing walls and shall connect to exterior load bearing walls and shall be spaced at not greater than 10 feet (3038 mm) on center. Ties shall have a minimum nominal tensile strength, $T_T$, given by Equation 16-45. For ASD the minimum nominal tensile strength may be taken as 1.5 times the allowable tensile stress times the area of the tie.

$$T_T = wLs \le \alpha_T s \qquad \textbf{(Equation 16-45)}$$

where:

$L$ = the span of the horizontal element in the direction of the tie, between bearing walls, ft, (m)

$w$ = the weight per unit area of the floor or roof in the span being tied to or across the wall, psf, (N/m$^2$)

$S$ = the spacing between ties, ft (m)

$\alpha_T$= a coefficient with a value of 1,500 lb/ft (2.25 kN/m) for masonry bearing wall structures and a value of 375 lb/ft (0.6 kN/m) for structures with bearing walls of light wood or cold formed steel frame construction.

**1614.4.2.2 Transverse ties.** Transverse ties shall consist of continuous reinforcement in slabs; continuous or spliced decks or sheathing; continuous or spliced members framing to, within, or across walls; or, connections of continuous framing members to walls. Transverse ties shall be placed no farther apart than the spacing of load bearing walls. Transverse ties shall have minimum nominal tensile strength $T_T$, given by Equation 16-45. For ASD the minimum nominal tensile strength may be taken as 1.5 times the allowable tensile stress times the area of the tie.

**1614.4.2.3 Perimeter ties.** Perimeter ties shall consist of continuous reinforcement in slabs; continuous or spliced decks or sheathing; continuous or spliced members framing to, within, or across walls; or, connections of continuous framing members to walls. Ties around the perimeter of each floor and roof shall be located within 4 feet (1219 mm) of the edge and shall provide a nominal strength in tension not less than $T_p$, given by Equation 16-46. For ASD the minimum nominal tensile strength may be taken as 1.5 times the allowable tensile stress times the area of the tie.

$$T_p = 200w \le \beta_T \qquad \textbf{(Equation 16-46)}$$

For SI:

$$T_p = 90.7w \le \beta_T$$

where

$w$ = as defined in Section 1614.4.2.1

$\beta_T$ = a coefficient with a value of 16,000 lbs (7.200 KN) for structures with masonry bearing walls and a value of 4,000 lbs (1,300 KN) for structures with bearing walls of light wood or cold formed steel frame construction.

**1614.4.3.4 Vertical ties.** Vertical ties shall consist of continuous or spliced reinforcing, continuous or spliced members, wall sheathing or other engineered systems. Vertical tension ties shall be provided in bearing walls and shall be continuous over the height of the building. The minimum nominal tensile strength for vertical ties within a bearing wall shall be equal to the weight of the wall within that story plus the weight of diaphragm tributary to the wall in the story below. No fewer than two ties shall be provided for each wall. The strength of each tie need not exceed 3,000 lb/ft (450 kN/m) of wall tributary to the tie for walls of masonry construction or 750 lb/ft (140 kN/m) of wall tributary to the tie for walls of light wood or steel frame construction.

**Reason:** This proposal was developed by a broad industry coalition that includes participation by the National Council of Structural Engineers Associations, the Structural Engineering Institute of the American Society of Civil Engineers, the American Institute of Architects, the American Concrete Institute, the American Forest & Paper Association, the American Iron and Steel Institute, the American Institute of Steel Construction, the Masonry Alliance for Codes and Standards, The Masonry Society, the Portland Cement Association, the Steel Joist

Institute, the Precast/Prestressed Concrete Institute. Corresponding members included the International Code Council and the National Fire Protection Association. In addition, there was nonvoting participation by the National Institute of Building Sciences and the National Institute of Standards and Technology.

It is the general consensus of NCSEA and the other members of the Ad Hoc Joint Industry Committee on Structural Integrity that the requirements already embodied in the building codes and standards together with the common structural design and construction practices prevalent in the United States today provide the overwhelming majority of structures with adequate levels of reliability and safety. The proposed provisions contained in this proposal are predicated upon requirements contained within the ACI 318 for many years. by adapting those requirements to structures of other construction types based on the differing conditions of weight and detailing. It is the opinion of the Ad Hoc Joint Industry Committee that these provisions will generally enhance the general structural integrity and resistance of structures by establishing minimum requirements for tying together the primary structural elements.

No cost impact on structures that are three stories or less in height. For some structures exceeding three stories in height, this proposal may result in minor increases in structural cost due to the additional strength of connections that are required. However, as the provisions contained in this proposal embody common design practices employed by many structural engineers, for many structures, the cost impact will be negligible.

**Cost Impact:** No cost impact on structures that are three stories or less in height. For some structures exceeding three stories in height, this proposal may result in minor increases in structural cost due to the additional strength of connections that are required. However, as the provisions contained in this proposal embody common design practices employed by many structural engineers, for many structures, the cost impact will be negligible.

## Committee Action: Disapproved

**Committee Reason:** There is a need for some structural integrity measures and some committee members feel this proposal would be a good step. However, it appears, as proposed, the current ACI 318 provisions for concrete have been extended to other materials without adequate explanation. The logic in doing so is lacking. These provisions would involve too many buildings that do not have integrity issues and there is no demonstrated need for enhancing these structures. There are also concerns about the consequences of requiring these provisions for buildings that are currently built all the time. There is some concern regarding how, or if, this analysis would relate to other required loading conditions – in particular, lateral loads.

## Assembly Action: None

# *Individual Consideration Agenda*

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

*Public Comment 1:*

### Paul K. Heilstedt, PE, FAIA, Chair, representing ICC Code Technology Committee and Gerry Jones/Herman Brice, representing NIBS/MMC Committee for Translating the NIST World Trade Center Investigation Recommendations into Building Codes, request Approval as Modified by this Public Comment.

**Modify proposal as follows:**

**1614.1 General**. Buildings <u>classified as high rise buildings in accordance with Section 404</u> and ~~other structures~~ assigned to Occupancy Category ~~II,~~ III, or IV, ~~exceeding three stories above grade plane~~ shall comply with the requirements of this section. Frame structures shall comply with the requirements of Section 1614.3. Bearing wall structures shall comply with the requirements of Section 1614.4.

> ~~**Exception:** Structures other than buildings with structural systems that are not like building structures including, but not limited to, billboards, signs, silos, tanks, stacks, mechanical and electrical equipment.~~

**1614.4.2.1 Longitudinal ties**. Longitudinal ties shall consist of continuous reinforcement in slabs; continuous or spliced decks or sheathing; continuous or spliced members framing to, within, or across walls; or, connections of continuous framing members to walls. Longitudinal ties shall extend across interior load bearing walls and shall connect to exterior load bearing walls and shall be spaced at not greater than 10 feet (3038 mm) on center. Ties shall have a minimum nominal tensile strength, $T_T$, given by Equation 16-45. For ASD the minimum nominal tensile strength ~~may~~ <u>shall be permitted to</u> be taken as 1.5 times the allowable tensile stress times the area of the tie.

$$T_T = wLs \leq \alpha_T s \qquad \textbf{(Equation 16-45)}$$

where:

L = the span of the horizontal element in the direction of the tie, between bearing walls, ft, (m)

w = the weight per unit area of the floor or roof in the span being tied to or across the wall, psf, (N/m$^2$)

S = the spacing between ties, ft (m)

$\alpha_T$= a coefficient with a value of 1,500 lb/ft (2.25 kN/m) for masonry bearing wall structures and a value of 375 lb/ft (0.6 kN/m) for structures with bearing walls of ~~light wood or~~ cold formed steel frame construction.

**1614.4.2.3 Perimeter ties**. Perimeter ties shall consist of continuous reinforcement in slabs; continuous or spliced decks or sheathing; continuous or spliced members framing to, within, or across walls; or, connections of continuous framing members to walls. Ties around the perimeter of each floor and roof shall be located within 4 feet (1219 mm) of the edge and shall provide a nominal strength in tension not less than $T_p$, given by Equation 16-46. For ASD the minimum nominal tensile strength ~~may~~ <u>shall be permitted to</u> be taken as 1.5 times the allowable tensile stress times the area of the tie.

$$T_p = 200w \leq \beta_T$$ **(Equation 16-46)**

For SI:

$$T_p = 90.7w \leq \beta_T$$

where
w = as defined in Section 1614.4.2.1
$\beta_T$ = a coefficient with a value of 16,000 lbs (7.200 KN) for structures with masonry bearing walls and a value of 4,000 lbs (1,300 KN) for structures with bearing walls of ~~light wood or~~ cold formed steel frame construction.

**1614.4.3.4 Vertical ties**. Vertical ties shall consist of continuous or spliced reinforcing, continuous or spliced members, wall sheathing or other engineered systems. Vertical tension ties shall be provided in bearing walls and shall be continuous over the height of the building. The minimum nominal tensile strength for vertical ties within a bearing wall shall be equal to the weight of the wall within that story plus the weight of diaphragm tributary to the wall in the story below. No fewer than two ties shall be provided for each wall. The strength of each tie need not exceed 3,000 lb/ft (450 kN/m) of wall tributary to the tie for walls of masonry construction or 750 lb/ft (140 kN/m) of wall tributary to the tie for walls of ~~light wood or~~ steel frame construction.

(Portions of proposal not shown remain unchanged)

**Commenter's Reason: Reason**: As noted in the reason for disapproval, there is a need for structural integrity provisions in the code and the reason further states that the provisions would involve too many building that do not have integrity issues. The CTC concurs with this philosophy - the need for such provisions should be a function of the relative risk. Low rise buildings which typically employ a less sophisticated structural system do not represent the same risk as taller buildings such as high rise buildings. Further, inclusion of Category II buildings also represents a large volume of buildings which would envelope a large population of buildings without detailing the risk. Category III buildings which are noted in Table 1604.5 of the code as "representing a substantial hazard to human life in the event of failure" such as high occupant load buildings, as well as Category IV buildings which are classified as "essential facilities" such as hospitals, warrant such provisions.

This public comment responds to these two fundamental issues by limiting the application to only Category III and IV buildings which are considered high rises.

Sections 164.4.2.1, 1614.4.2.3 and 1614.4.3.4 are correspondingly revised to remove the reference to wood construction as the application is now limited to high rises for which wood bearing walls would not be permitted based on type of construction.

Code issues are assigned to the CTC by the ICC Board as "areas of study". Information on the CTC, including: meeting agendas; minutes; reports; resource documents; presentations; and all other materials developed in conjunction with the CTC effort can be downloaded from the following website: http://www.iccsafe.org/cs/cc/ctc/index.html. Since its inception in Aprril/2005, the CTC has held fifteen meetings - all open to the public. This public comment is a result of the CTC's investigation of the area of study entitled "NIST World Trade Center Recommendations". The CTC web page for this area of study is: http://www.iccsafe.org/cs/cc/ctc/WTC.html

*Public Comment 2:*

## Joseph J. Messersmith, Portland Cement Association, requests Approval as Modified by this Public Comment.

**Modify proposal as follows:**

**1614.1 General.** Buildings ~~and other structures~~ assigned to Occupancy Category ~~II,~~ III~~,~~ or IV, ~~exceeding three stories above grade plane~~ <u>with an occupied floor located more than 75 feet (22 860 mm) above the lowest level of fire department vehicle access, and other structures assigned to Occupancy Category III or IV greater than 75 feet (22 860 mm) in height</u> shall comply with the requirements of this section. Frame structures shall comply with the requirements of 1614.3. Bearing wall structures shall comply with the requirements of Section 1614.4.

**Exception:** ~~Structures other than buildings with structural systems that are not like building structures including, but not limited to, billboards, signs, silos, tanks, stacks, mechanical and electrical equipment.~~ <u>Nonbuilding structures (see ASCE 7, Chapter 15).</u>

(Portions of proposal not shown remain unchanged)

**Commenter's Reason:** This code change was disapproved in part because the "provisions would involve too many buildings that do not have integrity issues." The modifications proposed will reduce the scope of application by deleting Occupancy Category II buildings and structures, and by increasing the height threshold from greater than 3 stories to high rise buildings as currently defined in Section 403.1, and greater than 75 feet in the case of other structures. The exception has been revised to refer to "nonbuilding structures" which is terminology used in Chapter 15 of ASCE 7, which seems to be consistent with the intent of the proposal.

Based on the forgoing, you are urged to overturn the motion for disapproval, and vote for a subsequent motion to approve the change as modified above.

Final Action: AS AM AMPC____ D

# S104-07/08
## 1704.1

### *Proposed Change as Submitted:*

**Proponent:** Gary J. Ehrlich, P.E., National Association of Home Builders, representing National Association of Home Builders

**Revise as follows:**

**1704.1 (Supp) General.** 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 special inspectors to provide inspections during construction on the types of work listed under Section 1704. These inspections are in addition to the inspections identified in Section 109.

The special inspector shall be a qualified person who shall demonstrate competence, to the satisfaction of the building official, for the inspection of the particular type of construction or operation requiring special inspection. The special inspector shall provide written documentation to the building official demonstrating their competence and relevant experience or training. 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.

**Exceptions:**

1. Special inspections are not required for work of a minor nature or as warranted by conditions in the jurisdiction as approved by the building official.
2. Special inspections are not required for building components unless the design involves the practice of professional engineering or architecture as defined by applicable state statutes and regulations governing the professional registration and certification of engineers or architects.
3. Unless otherwise required by the building official, special inspections are not required for occupancies in Group U that are accessory to a residential occupancy including, but not limited to, those listed in Section 312.1.
4. <u>Special inspections are not required for structures designed and constructed in accordance with the conventional construction provisions of Section 2308.</u>

**Reason:** A proposal in the previous cycle (RB31-06/07) struck the exemption for R-3 structures, subjecting those one- and two-family dwellings and townhouses which happen to fall under the IBC rather than the IRC to special inspections on top of the standard building department inspections. In addition, other structures falling under an R-3 occupancy (group homes, day care) will be subject to special inspections for all elements of their construction. Yet, contrary to the proponent's contentions, many of these structures will be simple structures and conform fully to the conventional construction provisions of Section 2308. They will not necessarily have the complex roof systems, steel framing, reinforced masonry and other elements the proponent offered as justification for removing the exemption.

Section 1704.1.1 states that the registered design professional is not required to prepare, and the permit applicant not required to submit, a statement of special inspections for structures designed and constructed per Section 2308. The implication is that these simple structures do not therefore require special inspections, which are really intended for structures that due to their complexity or use of unusual construction materials and methods require observation by an individual with the qualifications and experience of a special inspector. Thus it should be clarified that conventionally-framed structures are also exempt from the special inspections themselves, and not just from the documentation requirement.

NAHB asks for your support of this proposal.

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

**Committee Action:** **Disapproved**

**Committee Reason:** The proposed exception to special inspection for conventional light-frame construction may not be necessary, because exception 2 already covers this for the most part. It would be a good idea to resubmit this proposal with narrower limits on the proposed exception.

**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 proposal as follows:**

**1704.1 (Supp) General.** 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 special inspectors to provide inspections during construction on the types of work listed under Section 1704. These inspections are in addition to the inspections identified in Section 109.

The special inspector shall be a qualified person who shall demonstrate competence, to the satisfaction of the building official, for the inspection of the particular type of construction or operation requiring special inspection. The special inspector shall provide written documentation to the building official demonstrating their competence and relevant experience or training. 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.

**Exceptions:**

1. Special inspections are not required for work of a minor nature or as warranted by conditions in the jurisdiction as approved by the building official.
2. Special inspections are not required for building components unless the design involves the practice of professional engineering or architecture as defined by applicable state statutes and regulations governing the professional registration and certification of engineers or architects.
3. Unless otherwise required by the building official, special inspections are not required for occupancies in Group U that are accessory to a residential occupancy including, but not limited to, those listed in Section 312.1.
4. Special inspections are not required for portions of structures designed and constructed in accordance with the conventional construction provisions of Section 2308.

**Commenter's Reason:** While the IBC-S committee and testifiers supported this proposal in concept, there were concerns that the original language could lead to an exemption from special inspections for elements such as masonry walls or structural steel framing contained within an otherwise conventionally-framed building, or for the foundations of an otherwise conventionally-framed structure. This revision limits the exemption to just the wood-framed elements covered by Section 2308 and satisfies the committee's concerns. NAHB asks for your support in approving this proposal as modified and overturning the committee's action.

Final Action: AS AM AMPC____ D

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# S118-07/08

## 1704.5, Table 1704.5.1, Table 1704.5.3, 1708.1 through Table 1708.1.4

*Proposed Change as Submitted:*

**Proponent:** Jason Thompson, Concrete Masonry Association, representing Masonry Alliance for Codes and Standards

**1. Revise as follows:**

**1704.5 Masonry construction.** Masonry construction shall be inspected and ~~evaluated~~ verified in accordance with the requirements of Sections 1704.5.1 through 1704.5.3, depending on the classification of the building or structure or nature of the occupancy, as defined by this code.

**Exception:** Special inspections shall not be required for:

1. Empirically designed masonry, glass unit masonry or masonry veneer designed by Section 2109, 2110 or Chapter 14, respectively, or by Chapter 5, 7 or 6 of ACI 530/ASCE 5/TMS 402, respectively, when they are part of structures classified as Occupancy Category I, II or III in accordance with Section 1604.5.
2. Masonry foundation walls constructed in accordance with Table 1805.5(1), 1805.5(2), 1805.5(3) or 1805.5(4).
3. Masonry fireplaces, masonry heaters or masonry chimneys installed or constructed in accordance with Section 2111, 2112 or 2113, respectively.

**TABLE 1704.5.~~3~~2**
**~~LEVEL 2 SPECIAL INSPECTION~~**
**LEVEL 2 REQUIRED VERIFICATION AND INSPECTION OF MASONRY CONSTRUCTION**

| VERIFICATION AND INSPECTION ~~TASK~~ | ~~Continuous during task listed~~ CONTINUOUS | ~~Periodically during task listed~~ PERIODIC | REFERENCE FOR CRITERIA | | |
|---|---|---|---|---|---|
| | | | IBC ~~section~~ SECTION | TMS 402/ACI 530/ ASCE 5 [a] | TMS 602/ACI 530.1/ ASCE 6 [a] |
| 1. Verify compliance with the approved submittals | — | X | — | — | Art. 1.5 |
| 2. Verification of $f'_m$ and $f'_{AAC}$ prior to construction and for every 5000 sq.ft. during construction. | — | X | — | — | Art. 1.4B |
| 3. Verification of proportions of materials in premixed or preblended mortar and grout as delivered to the site. | — | X | — | — | Art. 1.5B |
| 4 Verification of slump flow and VSI as delivered to the site for self-consolidating grout | X | — | — | — | Art. 1.5B.1.b.3 |
| ~~1~~5 ~~From the beginning of masonry construction, the~~ The following shall be verified to ensure compliance: | | | | | |
| a. Proportions of site-prepared mortar, grout and prestressing grout for bonded tendons. | — | X | — | — | Art. 2.6A |
| b. Placement of masonry units and construction of mortar joints. | — | X | — | — | Art. 3.3B |
| c. Placement of reinforcement, connectors and prestressing tendons and anchorages. | X~~—~~ | —~~X~~ | — | Sec. 1.13 | Art. 3.4, 3.6A |
| d. Grout space prior to grouting. | X | — | — | — | Art. 3.2D |
| e. Placement of grout. | X | — | — | — | Art. 3.5 |
| f. Placement of prestressing grout. | X | — | — | — | Art. 3.6C |
| ~~2. The inspection program shall verify:~~ | | | | | |
| ~~a~~g. Size and location of structural elements | — | X | — | — | Art. 3.3G |
| ~~b~~h. Type, size and location of anchors, including other details of anchorage of masonry to structural members, frames or other construction. | X | — | — | Sec. 1.2.2(e), 2.1.4, 3.1.6 | — |
| ~~c~~i. Specified size, grade and type of reinforcement, anchor bolts, prestressing tendons and anchorages. | — | X | — | Sec. 1.13 | Art. 2.4, 3.4 |
| ~~d~~j. Welding of reinforcing bars. | X | — | — | Sec. 2.1.10.7.2, 3.3.3.4(b) | — |
| ~~e~~k. Preparation, construction and protection of masonry during cold weather (temperature below 40°F) or hot weather (temperature | — | X | Sec 2104.3, 2104.4 | — | Art. 1.8C, 1.8D |
| ~~f~~l. Ap | X | — | — | — | Art. 3.6B |

| VERIFICATION AND INSPECTION ~~TASK~~ | ~~Continuous during task listed~~ CONTINUOUS | ~~Periodically during task listed~~ PERIODIC | REFERENCE FOR CRITERIA | | |
|---|---|---|---|---|---|
| | | | IBC ~~section~~ SECTION | TMS 402/ACI 530/ ASCE 5[a] | TMS 602/ACI 530.1/ ASCE 6[a] |
| ~~3~~6 Preparation of any required grout specimens, mortar specimens and/or prisms shall be observed. | X | — | Sec. 2105.2.2, 2105.3 | — | Art. 1.4 |
| ~~4. Compliance with required inspection provisions of the construction documents and the approved submittals shall be verified~~ | — | ~~X~~ | — | — | ~~Art. 1.5~~ |

For SI: °C = (°F - 32)/1.8.
a. The specific standards referenced are those listed in Chapter 35.

**2. Delete without substitution as follows:**

~~**1708.1 Masonry.** Testing and verification of masonry materials and assemblies prior to construction shall comply with the requirements of Sections 1708.1.1 through 1708.1.4, depending on the classification of the building or structure or nature of the occupancy, as defined by this code.~~

~~**1708.1.1 Empirically designed masonry and glass unit masonry in Occupancy Category I, II or III.** For masonry designed by Section 2109 or 2110 or by Chapter 5 or 7 of ACI 530/ASCE 5/TMS 402 in structures classified as Occupancy Category I, II or III, in accordance with Section 1604.5, certificates of compliance used in masonry construction shall be verified prior to construction.~~

~~**1708.1.2 Empirically designed masonry and glass unit masonry in Occupancy Category IV.** The minimum testing and verification prior to construction for masonry designed by Section 2109 or 2110 or by Chapter 5 or 7 of ACI 530/ASCE 5/TMS 402 in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with the requirements of Table 1708.1.2.~~

~~**TABLE 1708.1.2**~~
~~**LEVEL 1 QUALITY ASSURANCE**~~

| ~~**MINIMUM TESTS AND SUBMITTALS**~~ |
|---|
| ~~Certificates of compliance used in masonry construction.~~<br><br>~~Verification of $f'_m$ and $f'_{AAC}$ prior to construction, except where specifically exempted by this code.~~ |

~~**1708.1.3 Engineered masonry in Occupancy Category I, II or III.** The minimum testing and verification prior to construction for masonry designed by Section 2107 or 2108 or by chapters other than Chapter 5, 6 or 7 of ACI 530/ASCE 5/TMS 402 in structures classified as Occupancy Category I, II or III, in accordance with Section 1604.5, shall comply with Table 1708.1.2.~~

~~**1708.1.4 Engineered masonry in Occupancy Category IV.** The minimum testing and verification prior to construction for masonry designed by Section 2107 or 2108 or by chapters other than Chapter 5, 6 or 7 of ACI 530/ASCE 5/TMS 402 in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with Table 1708.1.4.~~

~~**TABLE 1708.1.4**~~
~~**LEVEL 2 QUALITY ASSURANCE**~~

| ~~**MINIMUM TESTS AND SUBMITTALS**~~ |
|---|
| ~~Certificates of compliance used in masonry construction.~~<br><br>~~Verification of $f'_m$ and $f'_{AAC}$ prior to construction and every 5,000 square feet during construction.~~<br><br>~~Verification of proportions of materials in mortar and grout as delivered to the site.~~ |

(Renumber subsequent sections)

**Reason:** The revisions proposed in this code change reflect editorial and substantive revisions incorporated into the 2008 edition of the Building Code Requirements for Masonry Structures (TMS 402/ACI 530/ASCE 5), commonly referred to as the Masonry Standard Joint Committee (MSJC) Code. This code change proposal is one of several to harmonize the design and construction requirements for masonry within the IBC with those in the reference standard. A complete list of revisions incorporated into the reference standard is available for download at www.masonrystandards.org.

Specific revisions proposed above include:

IBC Section T1704.5.1 and 1704.5.3 have been revised to comply with the changes in the 2008 MSJC and to conform to the format of the tables for steel and concrete. In Section 1708 we are proposing to delete the Seismic Testing Provisions for masonry. 1708 doesn't indicate which SDC's require these tests. As such they are required in any SDC and are routinely preformed in all SDC's. This has caused significant confusion to many practitioners who don't know when to require this testing. They think it is only required in zones of moderate or high seismicity. Moving these requirements to T1704.5.1 and 1704.5.3 allows them to be eliminated from 1708 and follows the model of concrete.

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

## Committee Action: Approved as Modified

**Modify proposal as follows:**

**1704.5 Masonry construction.** Masonry construction shall be inspected and verified in accordance with the requirements of Sections 1704.5.1 through 1704.5.~~3~~ 2, depending on the classification of the building or structure or nature of the occupancy, as defined by this code.

**Exception:** Special inspections shall not be required for:

1. Empirically designed masonry, glass unit masonry or masonry veneer designed by Section 2109, 2110 or Chapter 14, respectively, or by Chapter 5, 7 or 6 of ACI 530/ASCE 5/TMS 402, respectively, when they are part of structures classified as Occupancy Category I, II or III in accordance with Section 1604.5.
2. Masonry foundation walls constructed in accordance with Table 1805.5(1), 1805.5(2), 1805.5(3) or 1805.5(4).
3. Masonry fireplaces, masonry heaters or masonry chimneys installed or constructed in accordance with Section 2111, 2112 or 2113, respectively.

**TABLE 1704.5.1**
**LEVEL 1 REQUIRED VERIFICATION AND INSPECTION OF MASONRY CONSTRUCTION**

| VERIFICATION AND INSPECTION | FREQUENCY OF INSPECTION | | REFERENCE FOR CRITERIA | | |
|---|---|---|---|---|---|
| | CONTINUOUS | PERIODIC | IBC SECTION | ACI 530/ASCE 5/TMS 402[a] | ACI 530.1/ASCE 6/TMS 602[a] |
| 1. Compliance with required inspection provisions of the construction documents and the approved submittals shall be verified. ~~Verify compliance with the approved submittals~~ | — | X | — | — | Art. 1.5 |

**TABLE 1704.5.2**
**LEVEL 2 REQUIRED VERIFICATION AND INSPECTION OF MASONRY CONSTRUCTION**

| VERIFICATION AND INSPECTION | CONTINUOUS | PERIODIC | REFERENCE FOR CRITERIA | | |
|---|---|---|---|---|---|
| | | | IBC SECTION | TMS 402/ACI 530/ ASCE 5[a] | TMS 602/ACI 530.1/ ASCE 6[a] |
| 1. Compliance with required inspection provisions of the construction documents and the approved submittals ~~Verify compliance with the approved submittals~~ | — | X | — | — | Art. 1.5 |
| 5 The following shall be verified to ensure compliance: | | | | | |
| a. Proportions of site-prepared mortar, grout and prestressing grout for bonded tendons. | — | X | — | — | Art. 2.6A |
| b. Placement of masonry units and construction of mortar joints. | — | X | — | — | Art. 3.3B |
| c. Placement of reinforcement, connectors and prestressing tendons and anchorages. | ~~X~~ | X | — | Sec. 1.13 | Art. 3.4, 3.6A |

(Portions of proposal not shown do not change)

**Committee Reason:** This code change brings the IBC requirements for masonry inspections into better alignment with the referenced standards for masonry construction. The modification retains the current requirement for periodic inspection of placement of reinforcement, connectors and prestressing tendons and anchorages. It also makes appropriate wording changes for the purpose of consistency.

**Assembly Action:** **None**

*Individual Consideration Agenda*

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

*Public Comment:*

**Gary R. Searer, S.E., Wiss, Janney, Elstner Associates, Inc., requests Approval as Modified by this Public Comment.**

**Further modify proposal as follows:**

**TABLE 1704.5.2**
**LEVEL 2 REQUIRED VERIFICATION AND INSPECTION OF MASONRY CONSTRUCTION**

| VERIFICATION AND INSPECTION | CONTINUOUS | PERIODIC | REFERENCE FOR CRITERIA | | |
|---|---|---|---|---|---|
| | | | IBC SECTION | TMS 402/ACI 530/ ASCE 5 [a] | TMS 602/ACI 530.1/ ASCE 6 [a] |
| 1. Compliance with required inspection provisions of the construction documents and the approved submittals. | — | X | — | — | Art. 1.5 |
| 2. Verification of $f'_m$ and $f'_{AAC}$ prior to construction and for every 5000 sq.ft. during construction. | — | X | — | — | Art. 1.4B |
| 3. Verification of proportions of materials in premixed or preblended mortar and grout as delivered to the site. | — | X | — | — | Art. 1.5B |
| 4. Verification of slump flow and VSI as delivered to the site for self-consolidating grout | X | — | — | — | Art. 1.5B.1.b.3 |
| 5. The following shall be verified to ensure compliance: | | | | | |
| a. Proportions of site-prepared mortar, grout and prestressing grout for bonded tendons. | — | X | — | — | Art. 2.6A |
| b. Placement of masonry units and construction of mortar joints. | — | X | — | — | Art. 3.3B |
| c. Placement of reinforcement, connectors and prestressing tendons and anchorages. | X | ~~X~~ | — | Sec. 1.13 | Art. 3.4, 3.6A |
| d. Grout space prior to grouting. | X | — | — | — | Art. 3.2D |
| e. Placement of grout. | X | — | — | — | Art. 3.5 |
| f. Placement of prestressing grout. | X | — | — | — | Art. 3.6C |
| g. Size and location of structural elements | — | X | — | — | Art. 3.3G |
| h. Type, size and location of anchors, including other details of anchorage of masonry to structural members, frames or other construction. | X | — | — | Sec. 1.2.2(e), 2.1.4, 3.1.6 | — |
| i. Specified size, grade and type of reinforcement, anchor bolts, prestressing tendons and anchorages. | — | X | — | Sec. 1.13 | Art. 2.4, 3.4 |

| VERIFICATION AND INSPECTION | CONTINUOUS | PERIODIC | REFERENCE FOR CRITERIA | | |
|---|---|---|---|---|---|
| | | | IBC SECTION | TMS 402/ACI 530/ ASCE 5 [a] | TMS 602/ACI 530.1/ ASCE 6 [a] |
| j. Welding of reinforcing bars. | X | — | — | Sec. 2.1.10.7.2, 3.3.3.4(b) | — |
| k. Preparation, construction and protection of masonry during cold weather (temperature below 40°F) or hot weather (temperature above 90°F). | — | X | Sec 2104.3, 2104.4 | — | Art. 1.8C, 1.8D |
| l. Application and measurement of prestressing force. | X | — | — | — | Art. 3.6B |
| 6. Preparation of any required grout specimens, mortar specimens and/or prisms shall be observed. | X | — | Sec. 2105.2.2, 2105.3 | — | Art. 1.4 |

For SI: °C = (°F - 32)/1.8.
a. The specific standards referenced are those listed in Chapter 35.

**(Portions of proposal not shown remain unchanged)**

**Commenter's Reason:** In Table 1704.5.2, the original S118 proposal required continuous inspection of placement of reinforcement, connectors, and prestressing tendons and anchorages to be consistent with the 2008 edition of the Building Code Requirements for Masonry Structures (TMS 402/ ACI 530/ ASCE5), commonly referred to as the Masonry Standard Joint Committee (MSJC) Code. A floor modification was presented by the proponent that changed this inspection requirement to periodic, but then the Committee was requested to alter the floor modification back to continuous. Given the confusion on this issue and rather than just disapprove this large and overall well-put-together proposal, the Committee elected to accept the floor modification as presented to the Committee (requiring only periodic inspection) and sort out this single continuous-versus-periodic issue as part of the Public Comment process. Although I am not submitting this public comment on behalf of the Committee, I believe after further research, Level 2 inspection of masonry should include continuous inspection of placement of reinforcement, connectors, and prestressing tendons and anchorages to be consistent with the MSJC.

Therefore, in order to be consistent with the MSJC, the above proposed change should be accepted.

Final Action: AS AM AMPC____ D

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# S124-07/08

## 1704.15 (New), Chapter 25 (New)

**THIS CODE CHANGE WILL BE HEARD ON THE IBC FIRE SAFETY PORTION OF THE HEARING ORDER.**

*Proposed Change as Submitted:*

**Proponent:** Gilbert Gonzales, Murray City Corporation, representing Utah Chapter ICC

**1. Add new text as follows:**

**1704.15 Fire-resistant penetration and joints.** Special inspection for through penetrations, membrane penetrations, joints and perimeter fire barrier systems of the types specified in Sections 712.3.1.2,712.4.1.2,713.3 and 713.4 respectively shall be in accordance with Section 1704.15.1 or 1704.15.2. Special inspections shall be based on fire-resistance rated design or system as designated in the approved construction documents.

**1704.15.1 Fire-resistant penetrations.** Inspections of fire-resistant penetrations systems of the types specified in Sections 712.3.1.2 and 712.4.1.2 shall be conducted by an inspection agency in accordance with ASTM E 2174.

**Exceptions:**

1. Buildings less than 4 stories above grade plane or,
2. Installation by UL or FM certified contractors.

**1704.15.2 Fire-resistive joints.** Inspection of joints of the types specified in Sections 713.3 and 713.4 shall be conducted by an approved inspection agency in accordance with ASTM E 2393.

**Exceptions:**

1. Buildings less than 4 stories above grade plane, or
2. Installation by UL or FM certified contractors.

**2. Add standards to Chapter 35 as follows:**

**ASTM International**

| | |
|---|---|
| E 2174-04 | Standard Practice for On-Site Inspection of Installed Fire Stops. |
| E 2393-04 | Standard Practice for On-Site Inspection of Installed Fire Resistive Joint Systems and Perimeter Fire Barriers. |

**Reason:** Installation of firestop systems is often installed by trades and or contractors who do not have the extensive knowledge or training needed to ensure that these critical life safety systems are installed correctly. At the same time, firestop and joint systems designs and materials are increasing in number and sophistication. Adding ASTM standard E2174-04 & ASTM E2393-04 outlines the inspection procedures for firestop inspectors. The addition of these new standards and certified contractors and or special inspection would provide and identify a means for building departments to have effective tools to instruct either their own staff or third party inspection agencies on good methodologies for inspection of these important systems. Requiring special inspection or certified contractors to perform the work will result in a proper installation.

This may or may not increase the cost of construction, depending on weather a UL of FM certified contractor is hired or requiring special inspection.

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

**This code change was heard by the IBC Fire Safety Code Development Committee.**

**Committee Action:** **Disapproved**

**Committee Reason:** The committee agreed that there was a lack of technical justification with respect to the exceptions of proposed Section 1704.15.1 for buildings less than 4 stories above grade and for installations by UL or FM certified contractors. The committee was also concerned that the proposed referenced standards (ASTM E2174 and E2393) contained nonmandatory language.

**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, Firestop Contractors International Association, representing himself and Gilbert Gonzales, Utah Chapter ICC, representing himself, request Approval as Modified by this Public Comment.**

**Modify proposal as follows:**

**1704.15 Fire-resistant penetration and joints**. Special inspection for through penetrations, membrane penetrations, joints and perimeter fire barrier systems of the types specified in Sections 712.3.1.2,712.4.1.2,713.3 and 713.4 respectively shall be in accordance with Section 1704.15.1 or 1704.15.2. Special inspections shall be based on fire-resistance rated design or system as designated in the approved construction documents.

**1704.15.1 Fire-resistant penetrations**. Inspections of fire-resistant penetrations systems of the types specified in Sections 712.3.1.2 and 712.4.1.2 shall be conducted by an inspection agency in accordance with ASTM E 2174.

**Exceptions:**

1. Buildings less than 4 stories above grade plane or,
2. Installation by UL Qualified or FM 4991 Approved ~~certified~~ contractors.

**1704.15.2 Fire-resistive joints**. Inspection of joints of the types specified in Sections 713.3 and 713.4 shall be conducted by an approved inspection agency in accordance with ASTM E 2393.

**Exceptions:**

1. Buildings less than 4 stories above grade plane, or
2. Installation by UL Qualified or FM 4991 Approved ~~certified~~ contractors.

**ASTM International**

E 2174-04 Standard Practice for On-Site Inspection of Installed Fire Stops.

E 2393-04 Standard Practice for On-Site Inspection of Installed Fire Resistive Joint Systems and Perimeter Fire Barriers.

**Commenter's Reason:** Installation of firestop systems is not required to be performed by contractors with any experience whatsoever in firestop systems. The firestopping trade is highly complex and detailed installation is imperative to assure performance when called upon by fire. Firestopping about fire science and zero-tolerance systems installation processes and many in the construction industry just don't understand the systems concept needed to get firestopping right.

Therefore, on projects where no certified contractor by an approved agency such as FM for FM 4991, Standard for the Approval of Firestop Contractors, or the UL Qualified Firestop Contractor are awarded work, there should be a final assurance that the important life safety protection is in fact installed to the classified firestop design before it's concealed. We are modifying the code change to specify the correct language so that those using and enforcing the code understand exactly what is meant by the certified firestop contractor concept.

In our opinion, the addition of these standards does not increase the real cost of construction. Improperly applied firestop systems with no classified UL, FM or Omega Point Laboratory designs used to install products do not become firestop systems, and may or may not work if called upon by fire…as well as protection of the structural fire resistant elements as well. Therefore, a special inspection of firestop systems is more than warranted and produces the desired result and may even reduce the cost as firestopping will have been installed properly the first time.

Although we heard the committee's call for greater detail in the scoping requirements for use of the standards, we respectfully disagree that the code should specify how small a building should be inspected per this requirement. There are many, many situations where size of the building, contents, occupancy, etc., may mean the building official would require use of the standards, regardless of how small or large the project is. Therefore, we believe the scoping language as written by the ICC Utah Chapter is appropriate.

Additionally, the non-mandatory language in the document is being addressed at ASTM. However, because it is a Standard Practice and not a finite product specification, there may always be some permissive language in the document.

Final Action: AS AM AMPC____ D

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# S125-07/08

## 1705.3. 1707.1. 1708.2

*Proposed Change as Submitted:*

**Proponent:** Bonnie Manley, American Iron and Steel Institute, representing American Institute of Steel Construction

**1. Revise as follows:**

**1705.3 (Supp) Seismic resistance.** The statement of special inspections shall include seismic requirements for the following cases:

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C, D, E or F, in accordance with Section 1613.
2. Designated seismic systems in structures assigned to Seismic Design Category D, E or F.
3. The following additional systems and components in structures assigned to Seismic Design Category C:
   3.1. Heating, ventilating and air-conditioning (HVAC) ductwork containing hazardous materials and anchorage of such ductwork.
   3.2. Piping systems and mechanical units containing flammable, combustible or highly toxic materials.
   3.3. Anchorage of electrical equipment used for emergency or standby power systems.
4. The following additional systems and components in structures assigned to Seismic Design Category D:
   4.1. Systems required for Seismic Design Category C.
   4.2. Exterior wall panels and their anchorage.
   4.3. Suspended ceiling systems and their anchorage.
   4.4. Access floors and their anchorage.
   4.5. Steel storage racks and their anchorage, where the importance factor is equal to 1.5 in accordance with Section 15.5.3 of ASCE 7.
5. The following additional systems and components in structures assigned to Seismic Design Category E or F:
   5.1. Systems required for Seismic Design Categories C and D.
   5.2. Electrical equipment.

   **Exception:** Seismic requirements are permitted to be excluded from the statement of special inspections for structures designed and constructed in accordance with the following:

   1. The structure consists of light-frame construction; the design spectral response acceleration at short periods, $S_{DS}$, is determined in Section 1613.5.4, does not exceed 0.5g; and the height of the structure does not exceed 35 feet (10 668 mm) above grade plane; or

2. The structure is constructed using a reinforced masonry structural system or reinforced concrete structural system; the design spectral response acceleration at short periods, $S_{DS}$, as determined in Section 1613.5.4, does not exceed 0.5g, and the height of the structure does not exceed 25 feet (7620 mm) above grade plane; or
3. Detached one- or two-family dwellings not exceeding two stories above grade plane, provided the structure does not have any of the following plan or vertical irregularities in accordance with Section 12.3.2 of ASCE 7:
   3.1. Torsional irregularity.
   3.2. Nonparallel systems.
   3.3. Stiffness irregularity extreme soft story and soft story.
   3.4. Discontinuity in capacity weak story.
4. Steel systems in structures that are assigned to Seismic Design Category C that are not specifically detailed for seismic resistance, with a response modification coefficient, R, of 3 or less, excluding cantilever column systems.

**1707.1 Special inspections for seismic resistance.** Special inspections itemized in Sections 1707.2 through 1707.10, unless exempted by the exceptions of Section 1704.1 or 1705.3, are required for the following:

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C, D, E or F, as determined in Section 1613.
2. Designated seismic systems in structures assigned to Seismic Design Category D, E or F.
3. Architectural, mechanical and electrical components in structures assigned to Seismic Design Category C, D, E or F that are required in Sections 1707.7 and 1707.8.

**1708.2 (Supp) Testing and qualification for seismic resistance.** The testing and qualification specified in Sections 1708.3 through 1708.6, unless exempted from special inspections by the exceptions of Section 1704.1 and 1705.3, are required as follows:

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C, D, E or F, as determined in Section 1613 shall meet the requirements of Sections 1708.3 and 1708.4, as applicable.
2. Designated seismic systems in structures assigned to Seismic Design Category D, E or F in Section 13.2.2 of ASCE 7 shall meet the requirements of Section 1708.5.
3. Architectural, mechanical and electrical components in structures assigned to Seismic Design Category C, D, E or F with an $I_p$ = 1.0 shall be permitted to be seismically qualified by meeting the requirements of Section 1708.5.
4. The seismic isolation system in seismically isolated structures shall meet the testing requirements of Section 1708.6.

**Reason**: In Sections 1705.3, 1707.1 and 1708.2, a general reference to SDC C is included for seismic force resisting systems in order to recognize that many structural systems require special detailing because of their seismic response characteristics. However, this general requirement does not reflect the unique response characteristics of some steel buildings. ASCE 7-05, Table 12.2-1 assigns steel building structures a response modification coefficient of R = 3, if they are built in SDC A, B or C as a "steel system not specifically detailed for seismic resistance, excluding cantilever column systems." For these building systems, the assigned seismic response coefficient reflects their inherent ductility. As a consequence, these structures are permitted to be constructed using only AISC 360 (that is, not detailed in accordance with the additional provisions of AISC 341). As these construction details and connections are the same as would be used in typical steel buildings following AISC 360, no additional inspection or testing should be required beyond that applied to typical steel buildings. The modifications to IBC Sections 1705.3, 1707.1 and 1708.2 reflect this concept.

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

**Committee Action:** **Approved as Modified**

**Modify proposal as follows:**

**1705.3 (Supp) Seismic resistance.** The statement of special inspections shall include seismic requirements for the following cases:

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C, D, E or F, in accordance with Section 1613.

   **Exception:** Requirements for the seismic-force resisting system are permitted to be excluded from the statement of special inspections for steel systems in structures assigned to Seismic Design Category C that are not specifically detailed for seismic resistance, with a response modification coefficient, R, of 3 or less, excluding cantilever column systems.

2. Designated seismic systems in structures assigned to Seismic Design Category D, E or F.
3. The following additional systems and components in structures assigned to Seismic Design Category C:
   3.1. Heating, ventilating and air-conditioning (HVAC) ductwork containing hazardous materials and anchorage of such ductwork.

3.2. Piping systems and mechanical units containing flammable, combustible or highly toxic materials.
3.3. Anchorage of electrical equipment used for emergency or standby power systems.

4. The following additional systems and components in structures assigned to Seismic Design Category D:
   4.1. Systems required for Seismic Design Category C.
   4.2. Exterior wall panels and their anchorage.
   4.3. Suspended ceiling systems and their anchorage.
   4.4. Access floors and their anchorage.
   4.5. Steel storage racks and their anchorage, where the importance factor is equal to 1.5 in accordance with Section 15.5.3 of ASCE 7.
5. The following additional systems and components in structures assigned to Seismic Design Category E or F:
   5.1. Systems required for Seismic Design Categories C and D.
   5.2. Electrical equipment.

**Exception:** Seismic requirements are permitted to be excluded from the statement of special inspections for structures designed and constructed in accordance with the following:

1. The structure consists of light-frame construction; the design spectral response acceleration at short periods, $S_DS$, is determined in Section 1613.5.4, does not exceed 0.5g; and the height of the structure does not exceed 35 feet (10 668 mm) above grade plane; or
2. The structure is constructed using a reinforced masonry structural system or reinforced concrete structural system; the design spectral response acceleration at short periods, *SDS*, as determined in Section 1613.5.4, does not exceed 0.5g, and the height of the structure does not exceed 25 feet (7620 mm) above grade plane; or
3. Detached one- or two-family dwellings not exceeding two stories above grade plane, provided the structure does not have any of the following plan or vertical irregularities in accordance with Section 12.3.2 of ASCE 7:
   3.1. Torsional irregularity.
   3.2. Nonparallel systems.
   3.3. Stiffness irregularity extreme soft story and soft story.
   3.4. Discontinuity in capacity weak story.
4. ~~Steel systems in structures that are assigned to Seismic Design Category C that are not specifically detailed for seismic resistance, with a response modification coefficient, R, of 3 or less, excluding cantilever column systems.~~

(Portions of proposal not shown remain unchanged)

**Committee Reason:** The committee agrees that the new exception is consistent with the intent of the current code and differentiates between designs under AISC 360 versus AISC 341. The modification puts the exception in a more appropriate location.

## Assembly Action: None

# *Individual Consideration Agenda*

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

*Public Comment:*

## David P. Tyree, American Forest and Paper Association, requests Approval as Modified by this Public Comment.

**Further modify proposal as follows:**

**1705.3 Seismic resistance.** The statement of special inspections shall include seismic requirements for ~~the following cases:~~ cases covered in Sections 1705.3.1 through 1705.3.5.

**Exception:** Seismic requirements are permitted to be excluded from the statement of special inspections for structures designed and constructed in accordance with the following:

1. The structure consists of light-frame construction; the design spectral response acceleration at short periods, $S_{DS}$, as determined in Section 1613.5.4, does not exceed 0.5g; and the height of the structure does not exceed 35 feet (10 668 mm) above grade plane; or
2. The structure is constructed using a reinforced masonry structural system or reinforced concrete structural system; the design spectral response acceleration at short periods, $S_{DS}$, as determined in Section 1613.5.4, does not exceed 0.5g, and the height of the structure does not exceed 25 feet (7620 mm) above grade plane; or
3. Detached one- or two-family dwellings not exceeding two stories above grade plane, provided the structure does not have any of the following plan or vertical irregularities in accordance with Section 12.3.2 of ASCE 7:
   3.1. Torsional irregularity.
   3.2. Nonparallel systems.
   3.3. Stiffness irregularity-extreme soft story and soft story.
   3.4. Discontinuity in capacity-weak story.

**1705.3.1** ~~1.~~ The seismic-force-resisting systems in structures assigned to Seismic Design Category C, D, E or F, in accordance with Section 1613.

**Exception:** Requirements for the seismic-force resisting system are permitted to be excluded from the statement of special inspections for steel systems in structures assigned to Seismic Design Category C that are not specifically detailed for seismic resistance, with a response modification coefficient, R, of 3 or less, excluding cantilever column systems.

**1705.3.2** ~~2.~~ Designated seismic systems in structures assigned to Seismic Design Category D, E or F.

**1705.3.3** ~~3.~~ The following additional systems and components in structures assigned to Seismic Design Category C:

~~3.~~1. Heating, ventilating and air-conditioning (HVAC) ductwork containing hazardous materials and anchorage of such ductwork.
~~3.~~2. Piping systems and mechanical units containing flammable, combustible or highly toxic materials.
~~3.~~3. Anchorage of electrical equipment used for emergency or standby power systems.

**1705.3.4** ~~4.~~ The following additional systems and components in structures assigned to Seismic Design Category D:

~~4.~~1. Systems required for Seismic Design Category C.
~~4.~~2. Exterior wall panels and their anchorage.
~~4.~~3. Suspended ceiling systems and their anchorage.
~~4.~~4. Access floors and their anchorage.
~~4.~~5. Steel storage racks and their anchorage, where the importance factor is equal to 1.5 in accordance with Section 15.5.3 of ASCE 7.

**1705.3.5** ~~5.~~ The following additional systems and components in structures assigned to Seismic Design Category E or F:
~~5.~~1. Systems required for Seismic Design Categories C and D.
~~5.~~2. Electrical equipment.

~~**Exception:** Seismic requirements are permitted to be excluded from the statement of special inspections for structures designed and constructed in accordance with the following:~~

1. ~~The structure consists of light-frame construction; the design spectral response acceleration at short periods, *SDS*, is determined in Section 1613.5.4, does not exceed 0.5g; and the height of the structure does not exceed 35 feet (10 668 mm) above grade plane; or~~
2. ~~The structure is constructed using a reinforced masonry structural system or reinforced concrete structural system; the design spectral response acceleration at short periods, *SDS*, as determined in Section 1613.5.4, does not exceed 0.5g, and the height of the structure does not exceed 25 feet (7620 mm) above grade plane; or~~
3. ~~Detached one- or two-family dwellings not exceeding two stories above grade plane, provided the structure does not have any of the following plan or vertical irregularities in accordance with Section 12.3.2 of ASCE 7:~~
   ~~3.1. Torsional irregularity.~~
   ~~3.2. Nonparallel systems.~~
   ~~3.3. Stiffness irregularity-extreme soft story and soft story.~~
   ~~3.4. Discontinuity in capacity-weak story.~~

(Portions of proposal not shown remain unchanged)

**Commenter's Reason:** This public comment is editorial in nature and renumbers this section and the original proposal into a more user friendly format. Further, it relocates the existing exception which is placed under the current 5.1 (which obviously does not apply to electrical equipment and will cause confusion) to apply to the appropriate section, Section 1705.3.

Final Action: AS AM AMPC____ D

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# S126-07/08
## 1705.4.2, 1708.4

*Proposed Change as Submitted:*

**Proponent:** Michael D. Fischer, The Kellen Company, representing ARMA: The Asphalt Roofing Manufacturer's Association

**Revise as follows:**

**1705.4.2 Detailed requirements.** The statement of special inspections shall include at least the following systems and components:

1. Roof deck connections. ~~cladding and roof framing connections.~~
2. Roof framing connections.
~~2~~3. Wall connections to roof and floor diaphragms and framing.
~~3~~4. Roof and floor diaphragm systems, including collectors, drag struts and boundary elements.
~~4~~5. Vertical windforce-resisting systems, including braced frames, moment frames and shear walls.
~~5~~6. Windforce-resisting system connections to the foundation.
~~6~~7. Fabrication and installation of systems or components required to meet the impact-resistance requirements of Section 1609.1.2.

   **Exception:** Fabrication of manufactured systems or components that have a label indicating compliance with the wind-load and impact-resistance requirements of this code.

**1708.4 (Supp) Wind-resisting components.** Periodic special inspection is required for the following systems and components:

1. Roof deck connections. ~~cladding.~~
2. Roof framing connections.
~~2~~ 3. Wall cladding.

**Exception:** Fabrication of manufactured systems or components that have a label indicating compliance with the wind-load and impact-resistance requirements of this code.

**Reason:** The addition of the requirement for special inspections for roof cladding in the last cycle is inconsistent with the existing "statement of special inspections" provisions found in 1705.4.2, most notably the exception for labeled components. This proposal is necessary to clarify that the connections between the roof covering and the roof framing are subject to special inspections, not the roof covering itself. By removing "roof cladding" from the IBC text and substituting it with "roof deck connections", and adding the requirement for "roof framing", the proposal is more easily interpreted. The exception for labeled products precludes the need for a special inspector to perform plant visits for roof coverings. This proposal solves the definition problems between ASCE-7 and the code.

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

**Committee Action: Disapproved**

**Committee Reason:** The committee did not agree with the removal of roof cladding. Special inspection of cladding is important and should be retained.

**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 proposal as follows:**

**1705.4.2 Detailed requirements**. The statement of special inspections shall include at least the following systems and components:

1. Roof ~~deck~~ cladding and roof cladding connections to roof deck, diaphragms, and framing.
2. Roof deck and roof framing connections.
3. Wall cladding and wall cladding connections to roof and floor deck, diaphragms and framing.
4. Roof and floor diaphragm systems, including collectors, drag struts and boundary elements.
5. Vertical windforce-resisting systems, including braced frames, moment frames and shear walls.
6. Windforce-resisting system connections to the foundation.
7. Fabrication and installation of systems or components required to meet the impact-resistance requirements of Section 1609.1.2.

Exception: Fabrication of manufactured systems or components that have a label indicating compliance with the wind-load and impact-resistance requirements of this code.

**1708.4 (Supp) Wind-resisting components**. Periodic special inspection is required for the following systems and components:

1. Roof ~~deck~~ cladding and roof cladding connections to roof deck, diaphragms, and framing.
2. Roof deck and roof framing connections.
3. Wall cladding and wall cladding connections to roof and floor deck, diaphragms and framing.

**Exception**: Fabrication of manufactured systems or components that have a label indicating compliance with the wind-load and impact-resistance requirements of this code.

**Commenter's Reason:** Proposal S45-06/07 last cycle added a very generic special inspection requirement for "wall cladding" and "roof cladding". This public comment attempts to clarify what needs to be inspected. The actual connections of the cladding elements to the building structure are of as much importance, if not more important, than the wind-resistance of the element itself. This modification allows the exception to work as intended to exempt a tested, labeled system or component from a special inspection, but retains a requirement to inspect the connections of the system or component to the building structure. The requirement is further coordinated with the statement of special inspections. This will achieve the proponent's desired action while addressing the committee's desire to retain some level of special inspection for roof and wall cladding. NAHB asks for your support in approving this proposal as modified and overturning the committee's decision.

Final Action: AS AM AMPC____ D

# S127-07/08
## 1706.1

*Proposed Change as Submitted:*

**Proponent:** D. Kirk Harman, PE, SE, The Harman Group, Inc., representing The National Council of Structural Engineers Associations (NCSEA) Code Advisory Committee, Quality Assurance and Special Inspection Subcommittee

**Delete without substitution:**

**~~SECTION 1706~~**
**~~CONTRACTOR RESPONSIBILITY~~**

~~**1706.1 Contractor responsibility.** Each contractor responsible for the construction of a main wind- or seismic-force-resisting system, designated seismic system or a wind- or seismic-resisting component listed in the statement of special inspections shall submit a written statement of responsibility to the building official and the owner prior to the commencement of work on the system or component. The contractor's statement of responsibility shall contain the following: 1. Acknowledgment of awareness of the special requirements contained in the statement of special inspections; 2. Acknowledgment that control will be exercised to obtain conformance with the construction documents approved by the building official; 3. Procedures for exercising control within the contractor's organization, the method and frequency of reporting and the distribution of the reports; and 4. Identification and qualifications of the person(s) exercising such control and their position(s) in the organization.~~

(Renumber subsequent sections)

**Reason:** This requirement was originally to go along with the Quality Assurance Plan, which has now been deleted from the code. The requirement is unenforceable, is not followed typically by contractors and is often ignored by jurisdictions.

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

**Committee Action:** **Approved as Submitted**

**Committee Reason:** There is no need for the section on contractor responsibility. Contractor responsibility is already implicit.

**Assembly Action:** **None**

*Individual Consideration Agenda*

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

*Public Comment 1:*

**Stephan Kiefer, City of Livermore, representing Peninsula, East Bay and Monterey Bay Chapters (Tri-Chapters), requests Approval as Modified by this Public Comment.**

**Replace proposal as follows:**

**1706.1 Contractor responsibility**. Each contractor responsible for the construction of a main wind- or seismic-force-resisting system, designated seismic system or a wind- or seismic-resisting component listed in the statement of special inspections shall submit a written statement of responsibility to the building official and the owner prior to the commencement of work on the system or component. The contractor's statement of responsibility shall contain ~~the following:~~ <u>acknowledgement of awareness of the special requirements contained in the statement of special inspection.</u>

1. ~~Acknowledgement of awareness of the special requirements contained in the statement of special inspections;~~
2. ~~Acknowledgement that control will be exercised to obtain conformance with the construction documents approved by the building official;~~
3. ~~Procedures for exercising control within the contractor's organization, the method and frequency of reporting and the distribution of the reports; and~~
4. ~~Identification and qualifications of the person(s) exercising such control and their position(s) in the organization.~~

**Commenter's Reason:** "Section 1704.1.1 requires that the registered design professional in responsible charge prepare and submits a "statement of special inspections" identifying the special inspections and tests required by Sections 1704, 1707, and 1708. Section 1706, as currently written, requires the Contractor to submit a written statement to the building official and to the owner acknowledging awareness of the items in the statement of special inspection, and describing how the contractor's quality control effort will be used to ensure that the work conforms with the construction documents. This section came into IBC at the beginning, directly from the Quality Assurance provisions of the NEHRP "Recommended Provisions for the Development of Seismic Regulations for New Buildings" (FEMA 95 from 1986, FEMA 222A from 1995.) These documents were the source for the "Quality Assurance Plan" of earlier editions of the IBC that is now called the statement of special inspection.

The contractor's statement of responsibility is a central part of this nationally-recognized long-standing effort to improve construction quality for seismic resistance by stressing the Contractor's Quality Control effort as an important element of the overall construction quality assurance effort, and thus should not be completely removed. This proposed modification simplifies the existing language by eliminating items 2, 3 and 4."

*Public Comment 2:*

**Can Xiao, City of Phoenix, AZ, representing herself, requests Disapproval.**

**Commenter's Reason:** It is important to have contractors' responsibilities being clarified and documented for important structural elements/systems. Proper construction of those systems will help protecting public safety and prevent casualties during natural disasters. Documentation of those responsibilities will help to clarify liability issue. I don't see why jurisdictions can not enforce this requirement.

Final Action: AS AM AMPC____ D

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# S128-07/08

## 1707.2

*Proposed Change as Submitted:*

**Proponent:** Bonnie Manley, American Iron and Steel Institute, representing American Institute of Steel Construction

**Revise as follows:**

**1707.2 Structural steel.** ~~Continuous special inspection is required for structural welding in accordance with AISC 341.~~Special inspection for structural steel members shall be in accordance with the quality assurance plan requirements of AISC 341.

**~~Exceptions:~~**

1. ~~Single-pass fillet welds not exceeding 5/16 inch (7.9 mm) in size.~~
2. ~~Floor and roof deck welding.~~

**Reason:** Between the 2003 and 2006 editions of the IBC, the terminology in Section 1705 was changed from "quality assurance plan" to "statement of special inspection". Unfortunately, the change in terminology was not picked up in time for the 2005 edition of AISC 341, *Seismic Provisions for Structural Steel Buildings*. In order to ensure that there is no confusion, a direct reference to the quality assurance plan requirements in AISC 341 is recommended for structural steel members. Part 1, Appendix Q of the 2005 AISC Seismic Provisions provides a comprehensive Quality Assurance Plan including Tables of QC and QA inspection requirements. For structures designed according to AISC 341, it is required that QC and QA be provided as specified in that section.

Earlier versions of AISC 341 did not specifically address the frequency of welding inspection. However, the Quality Assurance Plan in Appendix Q of AISC 341-05 now addresses frequency of inspection. The first exception for single pass fillet welds is recommended for deletion. Fillet welds are now covered in Appendix Q of AISC 341-05, so the exception is no longer necessary. Also, the second exception for floor and roof deck welding is recommended for deletion. This section requires adequate special inspections for seismic resistance of structural steel only. Section 1704.3 Exception 2.2 already sufficiently addresses the welding of the floor and roof deck in a general manner.

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

**Committee Action:** **Approved as Modified**

**Modify proposal as follows:**

**1707.2 Structural steel.** Special inspection for structural steel ~~members~~ shall be in accordance with the quality assurance plan requirements of AISC 341.

**Exception:** Special inspections of structural steel in structures assigned to Seismic Design Category C that are not specifically detailed for seismic resistance, with a response modification coefficient, R, of 3 or less, excluding cantilever column systems.

**Committee Reason:** This proposal helps clarify special inspections for seismic resistance. The modification adds as exception that is consistent with the intent of the current code as well as the action taken on S125-07/08.

**Assembly Action:** **None**

*Individual Consideration Agenda*

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

*Public Comment:*

**W. Lee Shoemaker, P.E., Ph.D., Thomas Associates Inc, representing Metal Building Manufacturers Association, requests Approval as Modified by this Public Comment.**

**Further modify proposal as follows:**

**1707.2 Structural steel.** Special inspection for structural steel shall be in accordance with the quality assurance plan requirements of AISC 341.

**Exceptions:**

1. Special inspections of structural steel in structures assigned to Seismic Design Category C that are not specifically detailed for seismic resistance, with a response modification coefficient, R, of 3 or less, excluding cantilever column systems.
2. For ordinary moment frames, ultrasonic and magnetic particle testing of complete joint penetration groove welds is only required for demand critical welds.

**Commenter's Reason:** This proposal is consistent with ongoing discussions at AISC to modify AISC 341 in recognition that ordinary moment frames have minimal inelastic straining. Therefore, only demand critical welds are of concern with regard to potential flaws in complete joint penetration groove welds. The first exception was accepted by the ICC Structural Committee as a modification by the proponent. The added exception is the modification requested by this public comment to make IBC 2009 consistent with what has been proposed to AISC 341 update process. But, because IBC-09 will reference the older edition of AISC 341 (2005), this exception needs to be added here as well. This is intended to be a temporary solution until AISC 341-10 is completed and adopted by the IBC.

Final Action: AS AM AMPC____ D

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# S132-07/08
## 1708.2

*Proposed Change as Submitted:*

**Proponent:** Gary J. Ehrlich, PE, National Association of Home Builders, representing National Association of Home Builders

**Revise as follows:**

**1708.2 (Supp) Structural wood.** ~~Continuous special inspection is required during field gluing operations of elements of the main wind-force-resisting system.~~ Periodic special inspection is required for nailing, bolting, anchoring and other fastening of components within the main wind-force-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 wind-force-resisting system, where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center.

**Reason:** Proposal S44-06/07 in the last code cycle introduced a new section on Special Inspections for Wind Requirements. The inspection requirements for wood framing were copied in their entirety from the existing requirements in Section 1707.3 for seismic resistance. However, the requirement for continuous inspection of field gluing is not warranted for the main wind-force resisting system. Adhesives present a known problem for seismic resistance as they affect the stiffness and energy dissipation of the system under seismic loading. We are not aware of any similar performance problems of adhesives under wind loads, where the load cycles are much longer and the energy level of a high-seismic event is not being imparted to the structure. Thus the continuous inspection requirement for adhesives should be deleted from the wind section.

The primary purpose for adhesives in structural wood framing is to improve the serviceability performance in floor systems and reduce vibration and deflection under standard floor live loads. An additional benefit is that adhesives can used to bond sheathing to supporting members in a roof assembly, providing substantial added resistance to hurricane winds. In fact, several adhesive products (FoamSeal, for example) are being touted as a cost-effective retrofit measure as well as in new construction, and insurance companies in hurricane-prone areas are offering discounts and incentives for use of the products. These mitigation efforts should not be penalized by imposing a costly and onerous special inspection requirement on new construction or substantial remodeling and retrofit work which would offset the incentives for using the adhesives.

NAHB asks for your support of this proposal.

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

**Committee Action:** **Disapproved**

**Committee Reason:** The committee did not agree with removing special inspections during field gluing operations, since this is something that can't be inspected after the fact.

**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, National Association of Home Builders (NAHB), requests Approval as Modified by this Public Comment.**

**Modify proposal as follows:**

**1708.2 (Supp) Structural wood**. Periodic special inspection is required for nailing, bolting, anchoring, field gluing of elements when specified for additional strength, and other fastening of components within the main wind-force-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 wind-force-resisting system, where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center.

**Commenter's Reason:** The IBC-S committee, in disapproving the original proposal, felt that field-gluing of elements in the MWFRS should be inspected. We agree where adhesives are specifically used to provide additional strength, for example attaching wall sheathing to studs for additional holding strength or shear strength. We disagree that inspection is required if the adhesives are used purely for aesthetic or serviceability reasons, for example to minimize deflection/vibration in an I-joist floor. However, given the provisions above, a wood-sheathed floor in a shear wall/diaphragm building is part of the MWFRS, and thus would be subject to the continuous inspection requirement regardless of the reason for using an adhesive. This proposed modification limits the requirement for inspection of adhesives to applications when they are used to improve the strength of an element, and moves the requirement to a periodic inspection. This modification meets the committee's desire to preserve a requirement for an appropriate level of inspection but does not require the constant presence of the inspector. We believe that, coupled with the standard building inspections, this modified requirement will provide an appropriate level of quality control. NAHB asks for your support in approving this proposal as modified, overturning the committee's action.

Final Action: AS AM AMPC____ D

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# S134-07/08
## 1708.4

## *Proposed Change as Submitted:*

**Proponent:** Michael D. Fischer, The Kellen Company, representing ARMA: The Asphalt Roofing Manufacturer's Association

**Revise as follows:**

**1708.4 (Supp) Wind-resisting components.** Periodic special inspection is required for the following systems and components:

1. Roof cladding and roof framing connections.
2. Wall cladding.

> **Exception:** Fabrication of manufactured systems or components that have a label indicating compliance with the wind-load and impact-resistance requirements of this code.

**Reason:** The addition of the requirement for special inspections for roof cladding in the last ICC cycle is inconsistent with the existing "statement of special inspections" provisions found in 1705.4.2, most notably the exception for labeled components. This proposal is necessary to clarify that the connections between the roof covering and the roof framing are subject to special inspections, not the roof cladding itself. The ambiguity between roof covering and roof cladding- defined in ASCE-7 but not in the IBC- is resolved in a separate code proposal. The exception for labeled products precludes the need for a special inspector to perform plant visits for roof coverings.

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

**Committee Action:** **Disapproved**

**Committee Reason:** It is not clear what type of labeling would be compliant were the proposed exception 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:*

**Michael D. Fischer, The Kellen Company, representing the Asphalt Roofing Manufacturers Association, requests Approval as Submitted.**

**Commenter's Reason:** The IBC Structural committee considered this proposal and S 126 together. While both were disapproved, the proponent asks for approval as submitted for S134 in order to clarify that it is the connections of both the roof cladding as well the roof framing that should be inspected. The exception for labeled products is necessary. The code intends that labeled products be manufactured under third party quality assurance programs; without the exception for labeled products such as roll-formed metal roofing, asphalt shingles or other listed roofing materials, the code could be interpreted to require plant visits by special inspectors. We do not believe that is the intent of this requirement.

The issue of correlation between inspection requirements in 1708.4 and reporting requirements 1705.4.2 raised in S 126 will be reconsidered in a future cycle. This clarification to define the inspection requirements will set the table for that effort**.**

Final Action: AS AM AMPC____ D

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# S136-07/08
## 1708.3

## *Proposed Change as Submitted:*

**Proponent:** Philip Brazil, PE, SE, Reid Middleton, Inc., representing himself

**Revise as follows:**

**1708.3 Reinforcing and prestressing steel**. Certified mill test reports shall be provided for each shipment of ~~reinforcing steel~~ reinforcement used to resist flexural~~, shear~~ and axial forces in reinforced concrete intermediate ~~frames,~~ and special moment frames and boundary elements of special reinforced concrete ~~or reinforced~~ and masonry shear walls. Where reinforcement complying with ASTM A 615 ~~reinforcing steel~~ is used to resist earthquake-induced flexural and axial forces in special moment frames and ~~in wall boundary elements of~~ special reinforced concrete shear walls in structures assigned to Seismic Design Category C, D, E or F, as determined in Section 1613, the testing requirements of Section 21.2.5 of ACI 318 shall be met. Where reinforcement complying with ASTM A 615 ~~reinforcing steel~~ is to be welded, chemical tests shall be performed to determine weldability in accordance with Section 3.5.2 of ACI 318.

> **Exception**: Certified mill test reports are not required to be provided for reinforcement complying with ASTM A 706.

**Reason:** The purpose of this proposal is to align the provisions of IBC Section 1708.3 with related provisions in ASCE 7 and ACI 318. The reference to axial forces is deleted for consistency with Section 21.2.5 of ACI 318-05. The references to types of seismic-force resisting systems are revised for consistency with Table 12.1-1 of ASCE 7-05. The exception for reinforcement complying with ASTM A 706 is proposed in recognition of the exemption from any special requirements for the use of such bars by Sections 3.5.2 and 21.2.5 of ACI 318-05. Note that Section 16 of ASTM A 706 specifies requirements for the marking of individual reinforcing bars complying with the standard for ready identification during construction.

The references in the proposal to sections of ACI 318, current and proposed, are to the 2005 edition. I assume the 2008 edition of ACI 318 will be the edition that is referenced in the 2009 IBC. The sections in the public draft of ACI 318-08 corresponding to the sections in the proposal are 21.1.5.2 for Section 21.2.5 and 3.5.2 for Section 3.5.2.

The reference to the testing requirements in Section 21.2.5 of ACI 318-05 ought to specify all frame members and structural wall boundary elements, which could conceivably include intermediate and special reinforced concrete moment frames and shear walls. Section 21.1.5.2 of the public draft of ACI 318-08, however, revises the requirement so that it applies to special moment frames, special structural walls and coupling beams. Section 21.1.1.4 of the public draft on structures assigned to Seismic Design Category C specifies compliance with the applicable provisions of Sections 21.1.3 through 21.1.7 for structures using special moment frames or special structural walls. The proposed revisions incorporate these upcoming changes in ACI 318.

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

**Committee Action:** **Approved as Modified**

**Modify proposal as follows:**

**1708.3 ~~Reinforcing and prestressing steel~~ Concrete reinforcement**. ~~Certified mill test reports shall be provided for each shipment of reinforcement used to resist flexural and axial forces in reinforced concrete intermediate and special moment frames and boundary elements of special reinforced concrete and masonry shear walls.~~ Where reinforcement complying with ASTM A 615 is used to resist earthquake-induced flexural and axial forces in special moment frames, ~~and~~ special ~~reinforced concrete shear~~ structural walls and coupling beams connecting special structural walls, in structures assigned to Seismic Design Category B, C, D, E or F~~,~~ as determined in Section 1613, the ~~testing requirements of~~ reinforcement shall comply with Section ~~21.2.5~~ 21.1.5.2 of ACI 318 ~~shall be met~~. 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.

~~**Exception**: Certified mill test reports are not required to be provided for reinforcement complying with ASTM A 706.~~

**Committee Reason:** This code change to seismic testing of concrete reinforcement provides consistency with the ACI 318 standard. The modification makes further adjustments in the wording and eliminates the proposed exception to certified mill test reports.

**Assembly Action:** **None**

## *Individual Consideration Agenda*

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

*Public Comment:*

**Can Xiao, City of Phoenix, AZ, representing herself, requests Approval as Modified by this Public comment.**

**Further modify proposal as follows:**

1708.3. Concrete reinforcement. Where reinforcement complying with ASTM A 615 is used to resist earthquake-induced flexural and axial forces in special moment frames, special shear walls and coupling beams connecting special structural walls, in structures assigned to Seismic Design Category B, C, D, E or F~~,~~ as determined in Section 1613, 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.

**Commenter's Reason:** The proponent didn't state any reason why shear reinforcement should not be required for special inspection. Good construction quality of Shear reinforcement is important for structural elements to prevent from shear failure, which is a non-ductile failure without any warning and can easily cause human casualties.

Final Action: AS AM AMPC____ D

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# S137-07/08
## 1708.4

## *Proposed Change as Submitted:*

**Proponent:** Bonnie Manley, American Iron and Steel Institute, representing American Institute of Steel Construction

**1. Revise as follows:**

**1708.4 Structural steel.** Testing for structural steel shall be in accordance with the quality assurance plan requirements of AISC 341.~~The testing contained in the quality assurance plan shall be as required by AISC 341 and the additional requirements herein. The acceptance criteria for nondestructive testing shall be as required in AWS D1.1 as specified by the registered design professional. Base metal thicker than 1.5 inches (38 mm), where subject to through-thickness weld shrinkage strains, shall be ultrasonically tested for discontinuities behind and adjacent to such welds after joint completion. Any material discontinuities shall be accepted or rejected on the basis of ASTM A 435 or ASTM A 898 (Level 1 criteria) and criteria as established by the registered design professional(s) in responsible charge and the construction documents~~

**2. Revise Chapter 35 as follows:**

**AISC**
341-05 Seismic Provisions for Structural Steel Buildings, including Supplement No. 1 dated 2005~~6~~. . . .. .
1613.6.2, 1707.2, 1709~~8~~.4, 2205.2.1, 2205.2.2, 2205.3, 2205.3.1

**ASTM INTERNATIONAL**

~~A 435/A 435M—90 (2001) Specification for Straight-beam Ultrasonic Examination of Steel Plates~~
~~A 898/A 898M—91 (2001) Specification for Straight Beam Ultrasonic Examination of Rolled Steel Shapes~~

**AWS**

D1.1—04 Structural Welding Code—Steel

**Reason:** *Section 1709.4, 1st paragraph (Numbering based upon IBC-06 w/2007 Supplement):* Between the 2003 and 2006 editions of the IBC, the terminology in Section 1705 was changed from "quality assurance plan" to "statement of special inspection". Unfortunately, the change in terminology was not picked up in time for the 2005 edition of AISC 341, *Seismic Provisions for Structural Steel Buildings*. In order to ensure that there is no confusion, a direct reference to the quality assurance plan requirements in AISC 341 is recommended for structural steel. In fact, AISC 341-05 Appendix Q provides the user with the minimum acceptable requirements for a quality assurance plan that applies to the construction of welded joints, bolted joints, and other details in the seismic load resisting system. The requirements of AISC 341, Appendix Q are recommended for implementation without revision. Where appropriate, AISC 341-05 Appendix Q references AWS D1.1 for specific acceptance criteria. Thus, the second sentence of the first paragraph is unnecessary and redundant with language that currently exists in AISC 341-05.

*Section 1709.4, 2nd paragraph:* The requirements of this paragraph are recommended for deletion. This paragraph focuses on the ultrasonic testing of base metal that may be subject to lamellar tearing or have laminations present. However, AISC 341-05, Section 5.2(2)(c) currently addresses this specific topic by stating when non-destructive testing (NDT) is needed, where it is needed and the appropriate acceptance criteria as follows:

**Q5.2(2)(c) Base Metal NDT for Lamellar Tearing and Laminations.** After joint completion, base metal thicker than 1-1/2 in. (38 mm) loaded in tension in the through thickness direction in tee and corner joints, where the connected material is greater than 3/4 in. (19 mm) and contains CJP groove welds, shall be ultrasonically tested for discontinuities behind and adjacent to the fusion line of such welds. Any base metal discontinuities found within t/4 of the steel surface shall be accepted or rejected on the basis of criteria of AWS D1.1 Table 6.2, where t is the thickness of the part subjected to the through thickness strain.

Referenced in AISC-341, Section Q5.2(2)(c), AWS D1.1 Table 6.2 provides the acceptance criteria for ultrasonically tested joints when statically loaded. The criteria is similar to that used prior to adoption of the current language in IBC 2000, which had used the term of "larger reflector criteria" in the UBC, and left it to the engineer in the NBC. The "larger reflector criteria", a termed used in the 1970s, is now identified as a "Class A" discontinuity in Table 6.2. By referencing only Table 6.2, and not referencing Class A, the additional considerations of flaw length and reflector height is made.

Finally, the direct references to ASTM A 435 and ASTM A898 are no longer needed because the AISC 341 criteria has been made more restrictive regarding permitted flaws, and more properly reflects the angle-beam ultrasonic methodology used for post-welding examinations. The prior reference to ASTM A 435 and A 898 were straight-beam ultrasonic tests to detect laminations in base metal prior to welding, and have been deemed inadequate for post-welding lamellar tearing checks.

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

## Committee Action: Approved as Modified

**Modify proposal as follows:**

**1708.4 Structural steel.** Testing for structural steel shall be in accordance with the quality assurance plan requirements of AISC 341.

<u>**Exception:** Testing for structural steel in structures assigned to Seismic Design Category C that are not specifically detailed for seismic resistance, with a response modification coefficient, R, of 3 or less, excluding cantilever column systems.</u>

(Portions of proposal not shown remain unchanged)

**Committee Reason:** This proposal helps clarify structural steel testing requirements for seismic resistance. The modification adds as exception that is consistent with the intent of the current code as well as S125-07/08.

## Assembly Action: None

# *Individual Consideration Agenda*

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

*Public Comment:*

## W. Lee Shoemaker, P.E., Ph.D, Thomas Associates, Inc, representing Metal Building Manufacturers Association requests Approval as Modified by this Public Comment.

**Further modify proposal as follows:**

**1708.4 Structural steel.** Testing for structural steel shall be in accordance with the quality assurance plan requirements of AISC 341.

**Exception<u>s</u>**

<u>1.</u> Testing for structural steel in structures assigned to Seismic Design Category C that are not specifically detailed for seismic resistance, with a response modification coefficient, R, of 3 or less, excluding cantilever column systems.

<u>2.</u> <u>For ordinary moment frames, ultrasonic and magnetic particle testing of complete joint penetration groove welds is only required for demand critical welds.</u>

(Portions of proposal not shown remain unchanged)

**Commenter's Reason:** This proposal is consistent with ongoing discussions at AISC to modify AISC 341 in recognition that ordinary moment frames have minimal inelastic straining. Therefore, only demand critical welds are of concern with regard to potential flaws in complete joint penetration groove welds. The first exception was accepted by the ICC Structural Committee as a modification by the proponent. The added exception is the modification requested by this public comment to make IBC 2009 consistent with what has been proposed to AISC 341 update process. But, because IBC-09 will reference the older edition of AISC 341 (2005), this exception needs to be added here as well. This is intended to be a temporary solution until AISC 341-10 is completed and adopted by the IBC.

Final Action: AS AM AMPC____ D

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# S141-07/08, Part I
## 1714.5

*Proposed Change as Submitted:*

**Proponent:** John Woestman, The Kellen Company, representing Window and Door Manufacturers Association

**PART I – IBC STRUCTURAL**

**Revise as follows:**

**1714.5 Exterior window and door assemblies.** The ~~design pressure~~ performance rating of exterior windows and doors in buildings shall be determined in accordance with Section 1714.5.1 or 1714.5.2.

> **Exception:** Structural wind load design pressures for exterior window and door units smaller than the size tested in accordance with Section 1714.5.1 or 1714.5.2 shall be permitted to be higher than the design value of the tested unit provided such higher pressures are determined by accepted engineering analysis. All components of the small unit shall be the same as the tested unit. Where such calculated design pressures are used, they shall be validated by an additional test of the window unit having the highest allowable design pressure.

**1714.5.1 Exterior windows and doors.** Exterior windows, ~~and~~ sliding doors, and side-hinged 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 1714.5.2.~~ Products tested and labeled as conforming to AAMA/WDMA/CSA 101/I.S.2/A440 shall not be subject to the requirements of Sections 2403.2 and 2403.3.

**1714.5.2 (Supp) Exterior windows and door assemblies not provided for in Section 1714.5.1**. Exterior window and door assemblies shall be tested in accordance with ASTM E 330. Structural performance of garage doors shall be determined in accordance with either ASTM E 330 or ANSI/DASMA 108, and shall meet the acceptance criteria of ANSI/DASMA 108. Exterior window and door assemblies containing glass shall comply with Section 2403. The design pressure for testing shall be calculated in accordance with Chapter 16. Each assembly shall be tested for a minimum of 10 seconds at a load equal to 1.5 times the design pressure.

(IBC) This proposal adds testing and labeling requirements for side-hinged door assemblies that are included within the scope of AAMA/WDMA/CSA 101/I.S.2/A440. Starting with the 2000 IBC (and IRC), exterior windows and exterior sliding doors have been required to be tested and labeled as conforming to AAMA/WDMA/CSA 101/I.S.2/A440 (or to the 1997 or 2002 versions of this standard) requiring window and sliding door assemblies to meet air infiltration, water infiltration, structural, operational, and forced entry performance requirements. This proposal adds side-hinged doors to the list of exterior fenestration products which will be required to meet air infiltration, water infiltration, operational, and forced entry performance requirements in addition to structural performance requirements currently required by the IBC and IRC.

It is important to note that the following products are not within the scope of AAMA/WDMA/CSA 101/I.S.2/A440, as listed in this industry consensus standard, and would not be affected by this proposal: curtain wall and storefront, storm doors, commercial entrance systems, revolving doors, site-built door systems, and commercial steel doors.

This proposal clarifies in the IBC that exterior window and door performance is not just design pressure. This clarification is important as exterior fenestration assemblies tested and labeled as conforming to AAMA/WDMA/CSA 101/I.S.2/A440 are required by this industry consensus standard to meet numerous performance requirements in addition to design pressure performance.

There are several editorial changes which clarify the code.

This proposal will increase complexity and cost of manufacturing side-hinged door assemblies because it requires side-hinged door assemblies to be tested, and labeled, to performance requirements previously not required.

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

**PART I – IBC STRUCTURAL**
**Committee Action: Disapproved**

**Committee Reason:** The committee's disapproval was based on the proponent's request.

**Assembly Action: None**

## *Individual Consideration Agenda*

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

*Public Comment 1:*

**Michael D. Fischer, The Kellen Company, representing The Window and Door Manufacturers Association (WDMA), requests Approval as Submitted for Part I.**

**Commenter's Reason:** The Window and Door Manufacturers Association (WDMA) recommends "Approval as Submitted."
All exterior components in a structure face the same exposure to weather. This proposal is a logical extension to apply the same requirements to side-hinged doors (within the scope of 101/I.S.2/A440) that are currently applied to windows and sliding doors. It is inconsistent for the code to require side-hinged door installation and flashing to prevent water infiltration while the door itself is exempted from any water testing requirements. Expanding the requirements for testing and labeling to 101/I.S.2/A440 to side-hinged doors addresses this inconsistency in the code.

*Public Comment 2:*

**The following list of individuals request Disapproval for Part I.**

**Commenters' Reason:** Standard does not reflect industry consensus on proper protocols for testing of exterior side hinged doors. According to ICC code development guidelines a need must exist, in this case no factual data or analysis has been presented to support S141's intentions.

**Commenter's Reason: (Blosser)** As a pre-hanger of side hinged door units this S141 would be cost prohibitive as it is currently proposed. The monumental testing costs plus capital and man power resources would render an operation such as ours as a non-feasible entity. Also the end consumer would have to ultimately pay for the additional costs which in my mind do nothing for the added value or performance of the end product. Simply put, S141 as written would create unnecessary regulation for pre-hangers, a virtual windfall for the testing agency, added cost to the consumer with little or no value added.
A more reasonable approach would be to have component manufacturers test their products, which many already do, and issue an umbrella authorization for their customers such as Bailey Millwork and establish a governing audit body by a testing lab to verify that tested products supplied by the manufacturer are being used in accordance with the manufacturer's specification. This approach allows testing costs to be spread over millions of tested components by manufacturers thus lessening the impact on pre-hangers and giving the end user an added value product at an affordable price. It should be noted that several vinyl window extruders are allowed a similar format in meeting or exceeding all national and international thermal, water, structural impact testing requirements.

**Commenter's Reason: (Lamont)** This is not needed and is harmful to trade and places undue burden on smaller businesses.

**Commenter's Reason: (Pixley)** The majority of pre-hung doors used in our market are built by pre-hangers similar to us. Like most pre-hangers, we source our components from multiple providers. Testing and certification of the thousands of combinations of door types (wood, steel, fiberglass), frame components (sills, weatherstripping, mulls, astragals), swing direction (outswing, inswing), door heights, door widths, configurations (single, side-hinged double, center-hinged doubles, transoms, with sidelights one or both sides), and glass inserts types will be astronomical and unaffordable.
The housing industry already is dealing with highly depressed market opportunities and the cost to comply with the proposed testing requirements simply will make things worse for those of us who create jobs and contribute positively to our nation's economy.

**Commenter's Reason: (Georgia Smith/Carolyn Stewart)** As stated by the committee in Palm Springs.

**William Bauman, Merrill Millwork, Inc., Merrill, WI**
**Joseph Bayer, President & CEO, Bayer Built Woodwork, Inc.**
**Tarry Beasley, Welsh Forest Products**
**Don Bell, Southwest Moulding Company**
**Jay M. Bjorndahl, Capitol Hardware, Inc.**
**Dennis H. Blosser, Bailey Millwork & Specialty Products**
**Chris Bougie, Scherer Bros. Lumber Company Millwork Solutions**
**James W. Bounds, Delmarva Millwork Corporation**
**Michael Brannon, Stock Building Supply**
**Ray Breedlove, Dealers Choice Millwork**
**Nathan Brown, Scherer Bros. Lumber Company Millwork Solutions**
**Kevin R. Bulow, Taylor Building Products, Inc.**

**J. Terry Bumgarner, King Sash & Door**
**Matt Carl, Badger Corrugating**
**Ed Chase, Smittys Millwork & Supply**
**Marshall D. Christie, Christie's Wood and Glass**
**Mark DeVol, Andersonville, TN, representing himself**
**Kolby Dickover, Hass Wholesale Westfield LLC, representing Hass Wholesale Millwork, Inc.**
**Glen Doke, McFarland Door Manufacturing Company, Inc.**
**Bryan Duncan, Alpine Millwork Company**
**Arthur M. Felder, Jr., Free Millwork Company, Inc.**
**Mark Fortun, Endura Products, Inc.**
**John D. Francis, Francis Schulze Company**
**Barry V. French, Acadian Millwork & Supply Company, Inc.**
**Don Hall, Don Hall & Associates, representing AMD**
**Jack Harbaugh, Foremost Industries, Inc.**
**James Hoxsie, Millwork Products, Inc.**
**Jeff Johnson, Western Pacific**
**Kevin P. Kavanagh, RSI, Inc.**
**Chris Kelly, Tague Lumber of Phoenixville**
**John Kerr, Kerr Millwork**
**Brad King, Pacific Pine Products**
**Noah Lamont, Lamont Glass, Window & Door**
**Kirby Letham, McFarland Door Manufacturing Company, Inc.**
**Richard Lowenthal, Lowenthal Sales Company**
**Angelo M. Marasco, ODL, Inc.**
**Lorn Marcellini, A Better Door,**
**Greg McAllister, Polaris Technologies**
**Peter McIlwee, J.J. McIlwee DBA McIlwee Millwork Company**
**Anthony Mehlbauer, Young Manufacturing Company, Inc.**
**Keith M. Milliken, Milliken Millwork, Inc.**
**Allen M. Nash, Sellersville, PA, representing himself**
**Bruce Norlie, Norfield Industries**
**Paul Pixley, Pixley Lumber Company, representing himself**
**Michael Pontremole, Fort Worth Sash and Door, representing AMD**
**Frank Racanelli, Medieval Glass Industries Iowa, Inc.**
**Tom Ringelberg, Quantum Sales and Marketing, representing Association of Millwork Distributors**
**Chris Roberson, Mid-Atlantic Millwork Sales, representing ODC**
**Daniel C. Rotchadl, Neubert Millwork Company**
**William Sarbaugh, River City Millwork**
**Randy Scarborough, Georgia Window Company**
**Judith M. Schell, Kimal Lumber Company, Window & Door Division**
**William Schmidt, Healdsburg Door & Sash**
**Jay Schrock, Woodwork Manufacturing Supply**
**Gary Smith, Coastal Door and Window**
**Georgia Smith, American Pre-Hung Door Association**
**Mark Sorenson, representing Scherer Brothers Lumber Company**
**Dave Stammen, Stock Building Supply**
**Sean Stevens, M&M Lumber Company**
**William G. Stevens, Trimpac, Inc., representing Millwork Distribution Organizations**
**Byron Stevenson, Dunn Lumber Company, Custom Door Shop**
**Carolyn Stewart, Tree Court Builders Supply, Inc.**
**Jim Stokes, Scherer Brothers Lumber Company Millwork Solutions**
**O. Frank Storch, Summit Construction, Inc.**
**Julie A. Sutton, Kimal Lumber Company, Window & Door Division**
**Vincent J. Tague, Jr., Tague Lumber, Inc.**
**Jeff VanWinkle, Dunagan Door Factory**
**Steve Wagman, J.B. Wagman Sales, Inc.**
**Lanie Wall, Jobbers, Inc.**
**Paul Weller, Bayerbuilt Woodworks, representing Association of Millwork Distributors**
**Kent C. Williams, Arrow Industries, Inc.**

Final Action: AS AM AMPC____ D

# S141-07/08, Part II
## IRC R613.4

### *Proposed Change as Submitted:*

**Proponent:** John Woestman, The Kellen Company, representing Window and Door Manufacturers Association

**PART II – IRC B/E**

**Revise as follows:**

**R613.4 Testing and labeling.** Exterior windows, ~~and~~ sliding doors, and side-hinged doors shall be tested ~~by an approved independent laboratory, and bear a label identifying manufacturer, performance characteristics and approved inspection agency to indicate compliance with AAMA/WDMA/CSA 101/I.S.2/A440. Exterior side-hinged doors shall be tested~~ and labeled as conforming to AAMA/ WDMA/CSA 101/I.S.2/A440 ~~or comply with Section R613.6.~~ The label shall state the name of the manufacturer, the approved labeling agency and the product designation as specified in AAMA/WDMA/CSA 101/I.S.2/A440.

**Exception:** Decorative glazed openings.

Reason: **(IRC) This proposal adds testing and labeling requirements for side-hinged door assemblies that are included within the scope of AAMA/WDMA/CSA 101/I.S.2/A440. Starting with the 2000 IRC (and IBC), exterior windows and exterior sliding doors have been required to be tested and labeled as conforming to AAMA/WDMA/CSA 101/I.S.2/A440 (or to the 1997 or 2002 versions of this standard) requiring window and sliding door assemblies to meet air infiltration, water infiltration, structural, operational, and forced entry performance requirements. This proposal adds side-hinged doors to the list of exterior fenestration products which will be required to meet air infiltration, water infiltration, operational, and forced entry performance requirements in addition to structural performance requirements currently required by the IBC and IRC.**

**This proposal also revises the language describing labeling requirements in IRC Section R613.4 to be consistent with the IBC.**

**It is important to note that storm doors and site-built door systems are not within the scope of AAMA/WDMA/CSA 101/I.S.2/A440, as listed in this industry consensus standard, and would not be affected by this proposal.**

**This proposal will increase complexity and cost of manufacturing side-hinged door assemblies because it requires side-hinged door assemblies to be tested, and labeled, to performance requirements previously not required.**

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

**PART II – IRC B/E**
**Committee Action: Disapproved**

**Committee Reason:** This proposal would create a hardship on the side-hinge door industry. This is an onerous requirement for an unproven need. There is no evidence that this is a big water intrusion problem. This proposal goes too far in trying to solve the inconsistency between flashing and water intrusion. The proponent should work with the door industry and bring this back.

**Assembly Action: None**

### *Individual Consideration Agenda*

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

*Public Comment 1:*

**Michael D. Fischer, The Kellen Company, representing The Window and Door Manufacturers Association (WDMA), requests Approval as Submitted for Part II.**

**Commenter's Reason:** The Window and Door Manufacturers Association (WDMA) recommends "Approval as Submitted."

All exterior components in a structure face the same exposure to weather. This proposal is a logical extension to apply the same requirements to side-hinged doors (within the scope of 101/I.S.2/A440) that are currently applied to windows and sliding doors. It is inconsistent for the code to require side-hinged door installation and flashing to prevent water infiltration while the door itself is exempted from any water testing requirements. Expanding the requirements for testing and labeling to 101/I.S.2/A440 to side-hinged doors addresses this inconsistency in the code.

*Public Comment 2:*

**The following list of individuals request Disapproval for Part II.**

**Commenters' Reason:** Standard does not reflect industry consensus on proper protocols for testing of exterior side hinged doors. According to ICC code development guidelines a need must exist, in this case no factual data or analysis has been presented to support S141's intentions.

**Commenter's Reason:** (Blosser) As a pre-hanger of side hinged door units this S141 would be cost prohibitive as it is currently proposed. The monumental testing costs plus capital and man power resources would render an operation such as ours as a non-feasible entity. Also the end consumer would have to ultimately pay for the additional costs which in my mind do nothing for the added value or performance of the end product. Simply put, S141 as written would create unnecessary regulation for pre-hangers, a virtual windfall for the testing agency, added cost to the consumer with little or no value added.

A more reasonable approach would be to have component manufacturers test their products, which many already do, and issue an umbrella authorization for their customers such as Bailey Millwork and establish a governing audit body by a testing lab to verify that tested products supplied by the manufacturer are being used in accordance with the manufacturer's specification. This approach allows testing costs to be spread over millions of tested components by manufacturers thus lessening the impact on pre-hangers and giving the end user an added value product at an affordable price. It should be noted that several vinyl window extruders are allowed a similar format in meeting or exceeding all national and international thermal, water, structural impact testing requirements.

**Commenter's Reason: (Lamont)** This is not needed and is harmful to trade and places undue burden on smaller businesses.

**Commenter's Reason: (Pixley)** The majority of pre-hung doors used in our market are built by pre-hangers similar to us. Like most pre-hangers, we source our components from multiple providers. Testing and certification of the thousands of combinations of door types (wood, steel, fiberglass), frame components (sills, weatherstripping, mulls, astragals), swing direction (outswing, inswing), door heights, door widths, configurations (single, side-hinged double, center-hinged doubles, transoms, with sidelights one or both sides), and glass inserts types will be astronomical and unaffordable.

The housing industry already is dealing with highly depressed market opportunities and the cost to comply with the proposed testing requirements simply will make things worse for those of us who create jobs and contribute positively to our nation's economy.

**Commenter's Reason: (Georgia Smith/Carolyn Stewart)** As stated by the committee in Palm Springs.

**William Bauman, Merrill Millwork, Inc., Merrill, WI**
**Joseph Bayer, President & CEO, Bayer Built Woodwork, Inc.**
**Tarry Beasley, Welsh Forest Products**
**Don Bell, Southwest Moulding Company**
**Jay M. Bjorndahl, Capitol Hardware, Inc.**
**Dennis H. Blosser, Bailey Millwork & Specialty Products**
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**James W. Bounds, Delmarva Millwork Corporation**
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**J. Terry Bumgarner, King Sash & Door**
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**Jeff Johnson, Western Pacific**
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**Chris Kelly, Tague Lumber of Phoenixville**
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**Brad King, Pacific Pine Products**
**Noah Lamont, Lamont Glass, Window & Door**
**Kirby Letham, McFarland Door Manufacturing Company, Inc.**
**Richard Lowenthal, Lowenthal Sales Company**
**Angelo M. Marasco, ODL, Inc.**
**Lorn Marcellini, A Better Door,**

**Greg McAllister, Polaris Technologies**
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**Anthony Mehlbauer, Young Manufacturing Company, Inc.**
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**Daniel C. Rotchadl, Neubert Millwork Company**
**William Sarbaugh, River City Millwork**
**Randy Scarborough, Georgia Window Company**
**Judith M. Schell, Kimal Lumber Company, Window & Door Division**
**William Schmidt, Healdsburg Door & Sash**
**Jay Schrock, Woodwork Manufacturing Supply**
**Gary Smith, Coastal Door and Window**
**Georgia Smith, American Pre-Hung Door Association**
**Mark Sorenson, representing Scherer Brothers Lumber Company**
**Dave Stammen, Stock Building Supply**
**Sean Stevens, M&M Lumber Company**
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**Vincent J. Tague, Jr., Tague Lumber, Inc.**
**Jeff VanWinkle, Dunagan Door Factory**
**Steve Wagman, J.B. Wagman Sales, Inc.**
**Lanie Wall, Jobbers, Inc.**
**Paul Weller, Bayerbuilt Woodworks, representing Association of Millwork Distributors**
**Kent C. Williams, Arrow Industries, Inc.**

Final Action: AS AM AMPC____ D

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# S142-07/08

## 1714.5.1

*Proposed Change as Submitted:*

**Proponent:** William E. Koffel, PE, Koffel Associates, Inc., representing Glazing Industry Code Committee

**Revise as follows:**

**1714.5.1 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 1714.5.2. ~~Products tested and labeled as conforming to AAMA/WDMA/CSA 101/I.S.2/A440 shall not be subject to the requirements of Sections 2403.2 and 2403.3.~~

**Reason:** The purpose of this proposal is to remove the exemption that fenestration products labeled to AAMA/WDMA/CSA 101/I.S.2/A440 do not have to meet the requirements of sections 2403.2 and 2403.3, which ensure safe performance through proper support of glass. Specifically, section 2403.3 requires that the deflection of framing members supporting glass may not exceed 1/175 of the glass edge length (or ¾ inch, whichever is less) when subjected to the design load. Chapter 24 of the IBC relies on glass design curves that are contained in ASTM E 1300. This ASTM standard recognizes the importance of limiting edge deflection of the glass and also recommends a limitation of 1/175 of the glass edge length. Prior to the IBC, the legacy codes required deflection limitations of 1/175 of the span for glass holding members. It was not until the IBC was published that this exemption was allowed.