2007/2008 PROPOSED CHANGES TO THE INTERNATIONAL BUILDING CODE — STRUCTURAL

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TENTATIVE ORDER OF DISCUSSION

2007/2008 PROPOSED CHANGES TO THE INTERNATIONAL BUILDING CODE

STRUCTURAL

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

Proposed change numbers that are indented are those which are being heard out of numerical order. Indentation does **not** necessarily indicate that one change is related to another. Proposed changes may be grouped for purposes of discussion at the hearing at the discretion of the chair.

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S1–07/08
1502.1

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.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S2–07/08
1502.1

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.

BALLAST (Supp). ~~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.

Reason: This code change will remove unnecessary language in the definition of ballast

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S3–07/08
1502.1 (New)

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:** 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.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S4–07/08
1503.1

Proponent: Michael D. Fischer, The Kellen Company, representing the Asphalt Roofing Manufacturer's Association (ARMA)

Revise as follows:

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

Reason: This proposal is an editorial change that removes a maintenance requirement that falls outside of the scope of the IBC.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S5–07/08
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; IRC R905.2.5 (New), R 905.4.6 (New), R905.6.6 (New), R905.7.6 (New), R905.8.7 (New), R905.10.5 (New), R907.3

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

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STUCTURAL AND IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

1. Add new text as follows:

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

<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.</u>

<u>**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.</u>

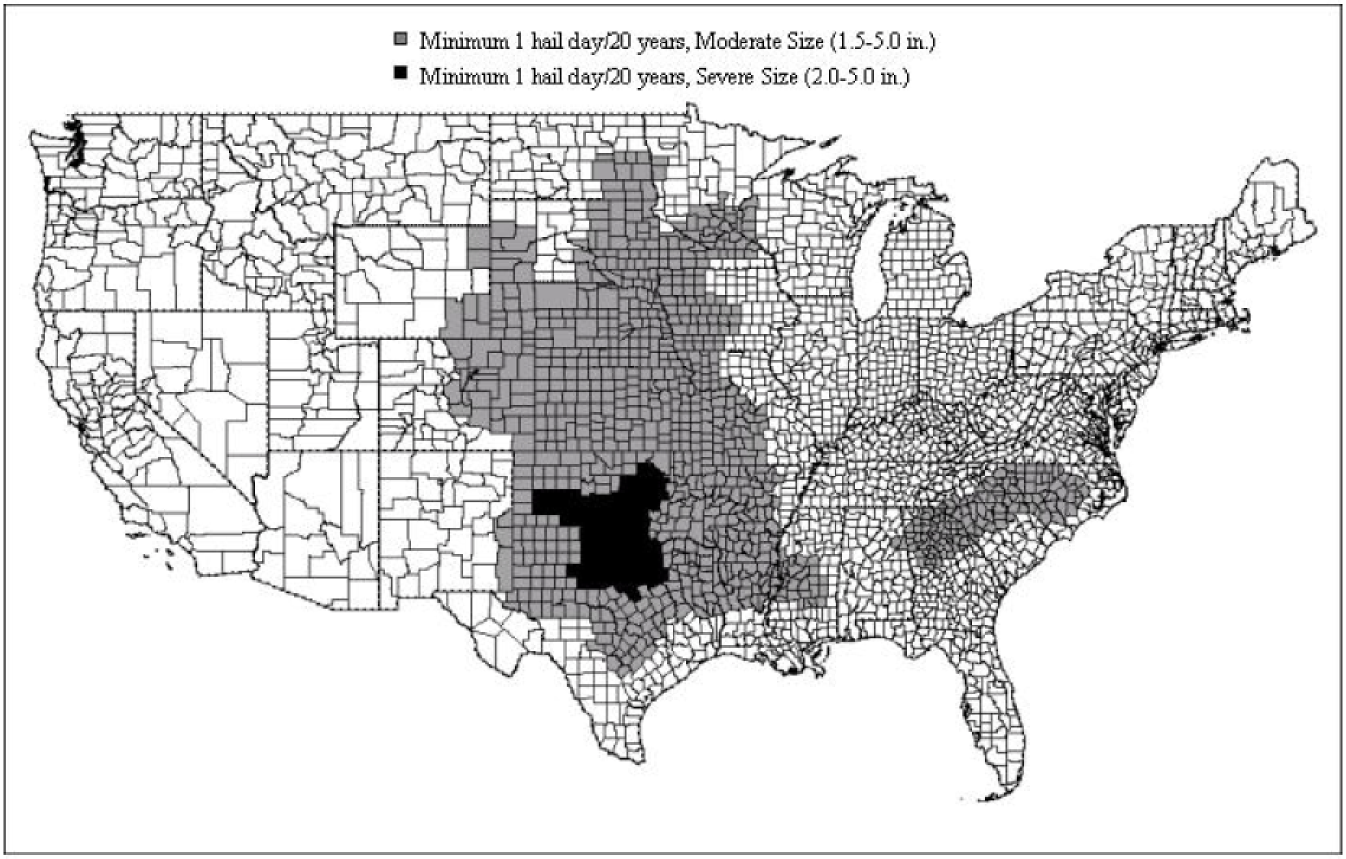

FIGURE 1503.6
HAIL EXPOSURE

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.

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.

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: (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.

(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 I – IBC STRUCTURAL

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IRC BUILDING/ENERGY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S6–07/08

1504.1.1, 1507.2.7.1, Table 1507.2.7, (New), 1609.5.2; IRC R905.2.4.1, Table R905.2.4.1 (New), Table R905.2.4.2 (New)

Proponent: Michael D. Fischer, The Kellen Company, representing the Asphalt Roofing Manufacturer's Association (ARMA)

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STUCTURAL AND IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

1. Revise as follows:

1504.1.1 (Supp) Wind resistance of asphalt shingles. Asphalt shingles shall ~~be installed in accordance~~ comply with Section 1507.2.7.

1507.2.7.1 (Supp) Wind resistance. Asphalt shingles shall be tested in accordance with ~~either ASTM D 3161 or~~ ASTM D 7158 ~~for wind resistance~~. Asphalt shingles shall meet the classification requirements of Table 1507.2.7.1(1) for the appropriate maximum basic wind speed. Asphalt shingle packaging shall bear a label to indicate compliance with ASTM D 7158 and the required classification in Table 1507.2.7.1(1).

> **Exception:** Asphalt shingles not included in the scope of ASTM D 7158 shall be tested and labeled to indicate compliance with ASTM D 3161 and the required classification in Table 1507.2.7.1(2).

2. Delete and substitute as follows:

~~**TABLE 1507.2.7**~~
~~**CLASSIFICATION OF ASPHALT ROOF SHINGLES**~~[a]

~~**MAXIMUM BASIC WIND SPEED FROM FIGURE 1609**~~	~~**ASTM D 3161**~~	~~**ASTM D 7158**~~ [b]
~~85~~	~~A,D, or F~~	~~D,G or H~~
~~90~~	~~A,D, or F~~	~~D,G or H~~
~~100~~	~~A,D, or F~~	~~G or H~~
~~110~~	~~F~~	~~G or H~~
~~120~~	~~F~~	~~G or H~~
~~130~~	~~F~~	~~H~~
~~140~~	~~F~~	~~H~~
~~150~~	~~F~~	~~H~~

~~For SI: 1 foot = 304.8 mm.~~

~~a. Asphalt Shingles shall be tested in accordance with ASTM D 3161 or ASTM D 7158. Refer to this table for selection of the appropriate product classification(s).~~

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

TABLE 1507.2.7.1(1)
CLASSIFICATION OF ASPHALT SHINGLES PER ASTM D 7158

MAXIMUM BASIC WIND SPEED FROM FIGURE 1609	CLASSIFICATION REQUIREMENT
85	D, G or H
90	D, G or H
100	G or H
110	G or H
120	G or H
130	H
140	H
150	H

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

TABLE 1507.2.7.1(2)
CLASSIFICATION OF ASPLALT SHINGLES PER ASTM D 3161

MAXIMUM BASIC WIND SPEED FROM FIGURE R301.2(4)	CLASSIFICATION REQUIREMENT
85	A, D or F
90	A, D or F
100	A, D or F
110	F
120	F
130	F
140	F
150	F

1609.5.2 (Supp) Roof coverings. Roof coverings shall comply with Section 1609.5.1.

Exception: Rigid tile roof coverings that are air permeable and installed over a roof deck complying with Section 1609.5.1 are permitted to be designed in accordance with Section 1609.5.3.

Asphalt shingles installed over a roof deck complying with 1609.5.1 shall ~~be permitted to be designed using ASTM D 7158 to determine~~ comply with the wind resistance requirements of Section 1507.2.7.1.

PART II – IRC BUILDING/ENERGY

1. Delete and substitute as follows:

~~**R905.2.4.1 Wind resistance of asphalt shingles.** Asphalt shingles shall be installed in accordance with Section R905.2.6. Asphalt shingles shall be tested for wind resistance in accordance with one of the following test standards:~~

~~1. ASTM D 3161~~
~~2. ASTM D 7158~~

~~Asphalt shingles shall meet the classification requirement of Table 905.2.4.1 for the applicable maximum basic wind speed. Asphalt shingle packaging shall bear a label to indicate compliance with one of the above ASTM test standards and the appropriate classification from Table 905.2.4.1.~~

R905.2.4.1 Wind resistance of asphalt shingles. Asphalt shingles shall be tested in accordance with ASTM D 7158. Asphalt shingles shall meet the classification requirements of Table R905.2.4.1(1) for the appropriate maximum basic wind speed. Asphalt shingle packaging shall bear a label to indicate compliance with ASTM D7158 and the required classification in Table R905.2.4.1(1).

Exception: Asphalt shingles not included in the scope of ASTM D 7158 shall be tested and labeled to indicate compliance with ASTM D 3161 and the required classification in Table R905.2.4.1(2).

~~TABLE R905.2.4.1~~
~~CLASSIFICATION OF ASPHALT ROOF SHINGLES~~[a]

~~MAXIMUM BASIC WIND SPEED FROM FIGURE 1609~~	~~ASTM D 3161~~	~~ASTM D 7158~~ [b]
~~85~~	~~A,D, or F~~	~~D,G or H~~
~~90~~	~~A,D, or F~~	~~D,G or H~~
~~100~~	~~A,D, or F~~	~~G or H~~
~~110~~	~~F~~	~~G or H~~
~~120~~	~~F~~	~~G or H~~
~~130~~	~~F~~	~~H~~
~~140~~	~~F~~	~~H~~
~~150~~	~~F~~	~~H~~

~~For SI: 1 foot = 304.8 mm~~

~~a. Asphalt shingles shall be tested in accordance with ASTM D 3161 or ASTM D 7158. Refer to this table for selection of the appropriate product classification(s).~~

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

TABLE R905.2.4.1(1)
CLASSIFICATION OF ASPLALT SHINGLES PER ASTM D 7158

MAXIMUM BASIC WIND SPEED FROM FIGURE R301.2(4)	CLASSIFICATION REQUIREMENT
85	D, G or H
90	D, G or H
100	G or H
110	G or H
120	G or H
130	H
140	H
150	H

TABLE R905.2.4.1(2)
CLASSIFICATION OF ASPLALT SHINGLES PER ASTM D 3161

MAXIMUM BASIC WIND SPEED FROM FIGURE R301.2(4)	CLASSIFICATION REQUIREMENT
85	A, D or F
90	A, D or F
100	A, D or F
110	F
120	F
130	F
140	F
150	F

Reason: This proposal completes the introduction of ASTM D7158 into the ICC, providing clear scoping for the applicable test standard for wind resistance of asphalt shingles. The reference to shingle compliance in 1504.1.1 provides clear compliance direction and requires the product comply with the appropriate product standard. The new reference in 1507.2.7.1 provides a “default” to ASTM D7158 with an option for products outside the scope of ASTM D3161. This requirement also removes a “dual path” to compliance by testing to either standard. In 1609.5.2, the pointer from the design section to the roofing requirement is complete.

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

PART I – IBC STRUCTURAL

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IRC BUILDING/ENERGY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S7–07/08
1504.2.1

Proponent: Michael D. Fischer, The Kellen Company, representing The Asphalt Roofing Manufacturer's Association (ARMA)

1. Delete without substitution as follows:

~~**1504.2.1 Alternative test method.** Testing the acceptability of special fastening methods using the methodology in this section is permitted. The wind-induced uplift force on the shingle shall be determined using the method in UL 2390. The resistance of the shingle to the uplift force shall be determined using ASTM D 6381. Shingles passing this test shall be considered suitable for roofs located where the basic wind speed per Figure 1609 is as given in Table 1504.2.1.~~

~~Classification requires that the resistance of the shingle to wind uplift, measured using the method in ASTM D 6381, exceed the calculated load imposed by wind in the applicable zone as determined using UL 2390.~~

~~Classification by this method applies to buildings less than 60 feet (18 288 mm) high and with Wind Exposures B and Conly in an Occupancy Category of I or II. Wrappers of shingle bundles that have been qualified using this alternative method shall be labeled with the tested wind classification and reference UL 2390/ASTM D 6381.~~

Reason: This section is incorrectly located; asphalt shingle requirements should not be located within the clay and concrete tile section. Moreover, this text should have been removed in the last cycle as part of FS 191 that added a reference to ASTM D7158, and also struck the Chapter 16 references to UL 2390 and ASTM D6381. Due to the incorrect location of this text in the 2006 IBC, this section was not included in the proposal. This proposal solves both issues. A companion proposal further clarifies the scoping of the wind standards for asphalt shingles.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S8–07/08
1504.3, Chapter 35 (New)

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</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: A review of the standard proposed for inclusion in the code, ANSI/SPRI WD-1, for compliance with ICC criteria for referenced standards given in Section 3.6 of Council Policy #CP 28 will be posted on the ICC website on or before January 15, 2008.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S9–07/08
1504.3.1, Chapter 35 (New)

Proponent: Mark S. Graham, National Roofing Contractors Association, representing Technical Operations Committee of the National Roofing Contractors Association; Phillip J. Smith, FM Approvals

1. Revise as follows:

1504.3.1 Other roof systems. Roof systems with built-up, modified bitumen, fully adhered or mechanically attached single-ply through fastened metal panel roof systems, and other types of membrane roof coverings shall also be tested in accordance with ~~FM 4450, FM 4470~~ <u>FM 4474</u>, UL 580 or UL 1897.

2. Add standard to Chapter 35 as follows:

FM

<u>4474-04</u> <u>Evaluating the Simulated Wind Uplift Resistance of Roof Assemblies Using Static Positive and/or Negative Differential Pressures</u>

Reason: (Graham) This code change proposal removes from the Code reference standards that do not comply with ICC's guidelines for reference standards and replaces them with a similar reference standard that appears to comply with ICC's guidelines.

FM 4450, "Approval Standard for Class 1 Insulated Steel Roof Decks—with Supplements through July 1992," and FM 4470, "Approval Standard for Class 1 Roof Covers," have been identified as not comply with ICC's guidelines for reference standards. Specifically, these standards have not been promulgated according by a consensus procedure and they prescribe a proprietary agency for quality control and testing. ICC acknowledged these shortcomings during testimony on FS193-06/07 and FS194-06/07 regarding this same issue; however, the code development committee was reluctant to withdraw these standards without substitution.

FM 4474, "Evaluating the Simulated Wind Uplift Resistance of Roof Assemblies Using Static Positive and/or Negative Differential Pressures," includes a similar wind test procedure to FM 4450 and FM 4470, but FM 4474 is promulgated through an ANSI process and it does not a proprietary agency for quality control and testing.

The addition of FM 4474 satisfies the code development committee's desire to provide an acceptable substitute for FM 4450 and FM 4470.

(Smith) The purpose of the change is to substitute an equivalent consensus standard for non consensus standards. FM 4450 and 4470 are not consensus documents. The wind uplift criteria of FM 4450 and FM 4470 is identical to that in ANSI/FM Approvals 4474. ANSI/FM Approvals 4474 is a consensus document meeting the ICC criteria for reference standards. A PDF copy of 4474 is attached.

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

Analysis: A review of the standard proposed for inclusion in the code, FM4474, for compliance with ICC criteria for referenced standards given in Section 3.6 of Council Policy #CP 28 will be posted on the ICC website on or before January 15, 2008.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S10–07/08
1504.4, 1504.4.1 (New), 1504.4.2 (New), 1504.4.3 (New), 1504.4.4 (New)

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:

<u>**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.</u>

<u>**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.</u>

<u>**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.</u>

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.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S11–07/08
1504.5 (New)

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

Add new text as follows:

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.

(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.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S12–07/08
1504.8

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.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S13–07/08
1504.8, Table 1504.8

Proponent: Mike Ennis, SPRI, Inc.

1. Revise as follows:

1504.8 (Supp) Aggregate. Aggregate shall not be used on the roof of a building located in a ~~hurricane-prone~~ <u>wind-borne debris</u> 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~~.

2. Delete table without substitution:

~~TABLE 1504.8~~
~~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]	~~BASIC WIND SPEED FROMFIGURE1609 (mph)~~[b]	~~MAXIMUM MEAN ROOF HEIGHT (ft)~~[a,c]
	~~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.~~

Reason: This code change addresses the use of aggregate in the wind-borne debris regions as defined in Section 1609.2: **WIND-BORNE DEBRIS REGION.** Table 1504.8 which extends the limitation of aggregate on roofs beyond the hurricane prone region is deleted because the major concern with aggregate blow-off, if any, is in the wind borne debris region.

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 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 aggregate blow-off in high wind conditions, actual experience shows that the concern about roof aggregate blow-off, if any, should only be in the wind borne debris regions.

The proposed restriction 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.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S14–07/08

1502.1, 1504.8, Chapter 35 (New)

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<u>, as defined in ASTM D 1863</u>.

(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.~~

<u>**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.</u>

~~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: A review of the standard proposed for inclusion in the code, ASTM D 1863, for compliance with ICC criteria for referenced standards given in Section 3.6 of Council Policy #CP 28 will be posted on the ICC website on or before January 15, 2008.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S15–07/08

Table 1504.8, 1602.1 (New); IRC R202

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

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STUCTURAL AND IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

1. Revise table as follows:

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.

2. Add new definition as follows:

SECTION 1602
DEFINITIONS AND NOTATIONS

MEAN ROOF HEIGHT. The vertical distance from grade plane to the average of the roof eave height and the height of the highest point on the roof surface, except that, for roof angles no greater than 10 degrees (0.174 rad), the mean roof height shall be the vertical distance from grade plane to the roof eave height.

PART II – IRC BUILDING/ENERGY

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 I – IBC STRUCTURAL

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IRC BUILDING/ENERGY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S16–07/08
1504.9 (New), Chapter 35 (New)

Proponent: Mike Ennis, SPRI, Inc.

1. Add new text as follows:

1504.9 Green roofs. Green roofs shall be installed in accordance with the requirements of Section 1507 and ANSI/SPRI RP-14.

2. Add standard to Chapter 35 as follows:

Single-Ply Roofing Institute
ANSI/SPRI RP-14 Wind Design Standard for Vegetative Roofing Systems

Reason: This code proposal addresses the wind uplift performance of green roof systems. **Section 1507.16 Roof gardens and landscaped roofs** requires that Green Roofs meet the requirements of Chapter 15, Section 1607.11.2.2 and Section 1607.11.2.3. This Code proposal is intended to provide requirements in Chapter 15 for green roofs to resist design wind uplift forces. The referenced standard ANSI/SPRI RP-14 Wind Design Standard For Vegetative Roofing Systems is currently in the ANSI canvassing process and will be completed in time for the initial code development hearing.

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

Analysis: A review of the standard proposed for inclusion in the code, ANSI/SPRI RP-14, for compliance with ICC criteria for referenced standards given in Section 3.6 of Council Policy #CP 28 will be posted on the ICC website on or before January 15, 2008.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S17–07/08
1505.1

Proponent: Jesse J. Beitel, Hughes Associates, Inc., representing Contractors Association and National Roofing Contractors Association

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

Revise as follows:

1505.1 General. Roof assemblies shall be divided into the classes defined below. Class A, B and C roof assemblies and roof coverings required to be listed by this section shall be tested in accordance with ASTM E 108 or UL 790. Testing shall be conducted such that for each slope tested, the test apparatus' airflow is uniform across the width of the test deck to within ± 4percent of the average. In addition, fire-retardant-treated wood roof coverings shall be tested in accordance with ASTM D 2898. The minimum roof coverings installed on buildings shall comply with Table 1505.1 based on the type of construction of the building.

Exception: Skylights and sloped glazing that comply with Chapter 24 or Section 2610.

Reason: Currently, both ASTM E 108 and UL 790 require that the airflow over the test sample be calibrated. This calibration is conducted with a calibration deck set at a slope of 5:12 and the airflow is measured in the center of the board as well as 3 inches in from each edge. The air flow is adjusted until the airflow provides a 1 minute timed average velocity of 1056 ± 44 ft./min (± 4%) at each of the three locations. While this calibration allows the test lab and other test laboratories to provide a fairly constant air flow, this calibration only applies to the 5:12 slope.

When roofing systems such as commercial roof decks with insulation and membrane roof coverings are tested, the test slopes are reduced from 5:12 to as low as ½:12. When the slope of the test deck is lowered, the air flow changes dramatically and it varies significantly across the test sample. Also, as the slope is lowered, the airflow is not the same from lab to lab.

The Midwest Roofing Contractors Association and the National Roofing Contractors Association contracted with Hughes Associates, Inc. to conduct a series of roofing tests to address several issues concerning the reproducibility and the repeatability of the ASTM E 108 roofing test. The results of this work are reported in a Hughes Associates, Inc. Final Report, “Fire Performance of Mechanically-Attached Single-Ply Membrane Roof Assemblies”, Project No. 5259, dated September 28, 2005.

The testing indicated that the repeatability (within lab) results were good however, the reproducibility (lab to lab) was very poor. One of the primary issues was the problem with differences in airflows when the sample is at slopes significantly less than the 5:12 calibration slope.

All test laboratories were capable of calibrating the airflow and flame temperatures at a 5:12 slope in accordance with ASTM E 108. Calibration at slopes other than 5:12 is not required by ASTM E 108, however, as shown in the Table below, airflow measurements at slopes less than 5:12 (average velocity to be 1056 ft/min) showed significantly lower air flow velocities combined with large fluctuations between centerline and the two outboard measurement locations.

Air Flow Calibration Results (Slopes other than 5:12)

Lab Designation	Calibration Slope	Air Flow Calibration Results (ft/min)			
		Left Average Air Flow	Center Average Air Flow	Right Average Air Flow	Average Air Flow
1 (Cal 3)	¼:12	394	1001	557	651
1 (Cal 4)	¼:12	404	991	513	636
2 (Cal 1)	¼:12	950	839	197	662
2 (Cal 6)	¼:12	376	565	170	370
1 (Cal 1)	½:12	498	976	712	729
1 (Cal 2)	½:12	567	991	660	739
2 (Cal 2)	½:12	419	800	256	492
2 (Cal 5)	½:12	440	968	277	562
2 (Cal 3)	1:12	637	926	415	659
2 (Cal 4)	1:12	585	853	402	613

At the three slopes less than 5:12 where airflow calibrations were measured (1:12, ½:12, and ¼:12), the variations in airflow were significant especially across the test deck. The variation in the measured airflow velocity ranged between 30 and 65 % of the average (i.e., standard deviation). These variations in airflow indicate that their variability may influence the flame spread noted on the test samples.

The measured average airflow values at the ¼:12 and the ½:12 slopes also show the significant differences between the two laboratories at the specified slopes.

Since air flow across the deck is a potentially significant variable, it would be prudent to require that the airflow be measured at the actual slope being tested and that the three measurements should agree within ± 4% in a similar manner as that currently done for the 5:12 calibration slope. While no value at each slope is currently being recommended, the airflow across the deck should at least be uniform and by taking and reporting this information, an appropriate air flow for each slope can be developed in the future.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S18–07/08

1505.2; IRC R902.1

Proponent: John C. Dean, National Association of State Fire Marshals (NASFM)

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC FIRE SAFTEY AND IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC FIRE SAFETY

Revise as follows:

1505.2 (Supp) Class A roof assemblies. Class A roof assemblies are those that are effective against severe fire test exposure. Class A roof assemblies and roof coverings shall be listed and identified as Class A by any approved testing agency. Class A roof assemblies shall be permitted for use in buildings or structures of all types of construction.

Exceptions:

1. Class A roof assemblies include those with coverings of brick, masonry, ~~slate, clay or concrete roof tile,~~ or an exposed concrete roof deck.
2. Class A roof assemblies also include ferrous or copper shingles or sheets, metal sheets and shingles, clay and concrete roof tile, and slate installed on non-combustible decks.

PART II – IRC BUILDING/ENERGY

R902.1 Roofing covering materials. Roofs shall be covered with materials as set forth in Sections R904 and R905. Class A, B or C roofing shall be installed in areas designated by law as requiring their use or when the edge of the roof is less than 3 feet (914 mm) from a property line. Classes A, B and C roofing required to be listed by this section shall be tested in accordance with UL 790 or ASTM E 108.

Exceptions:

1. Class A roof assemblies include those with coverings of brick, masonry~~, slate, clay or concrete roof tile,~~ and exposed concrete roof deck.
2. Class A roof assemblies also include ferrous or copper shingles or sheets, metal sheets and shingles, clay and concrete tile, installed on noncombustible decks.

Reason: (IBC/IRC) This is a follow-up proposal to code change FS199-06/07 that was submitted by the National Association of State Fire Marshals and approved as modified by their public comment at the ICC Final Action Hearings in Rochester, NY. The public comment cited some ASTM E108 (UL790) tests in UL Report file SV16680, Project 07CA03538 "Fact-Finding Investigation of Metal and Slate Prepared Roof Coverings" dated Jan 17, 2007.

It should be noted that during the prior code cycle the automatic Class A designation was removed for steel, copper, and slate. This proposal addresses the same problem and treats clay and concrete tile in the same manner.

This proposal only seeks to remove language and exceptions that automatically confer Class A status to certain materials. It does not eliminate the use of any product. The increasingly wide range of materials and configurations used as clay roofing now available in the market can no longer support a general exemption from fire testing these assemblies. All roof assemblies with clay or concrete tile roof covering should be tested in accordance with ASTM E108 or UL 790. The modification recognizes that clay and concrete tile roof coverings on non-combustible decks do not constitute a hazard.

Under brush fire conditions in the field, roof coverings can be exposed to burning brands that may break slate or clay or concrete roof tile, since they are brittle materials, and expose the roof deck to the fire; or the high winds caused by the brush fire can lift the butt ends of slate or concrete or clay roof tiles, allowing the entry of embers under the roof covering and igniting the combustible deck.

At the Rochester Final Action Hearing, the membership vote made it clear that fire resistance testing is favored over the exceptions based on field experience in past editions of the IBC. This change would also provide a level playing field for all of the roofing materials that would be included in exception 2.

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

PART I – IBC FIRE SAFETY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IRC BUILDING/ENERGY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S19–07/08
1505.2; IRC R902.1

Proponent: Kate Dargan, State Fire Marshal, CAL FIRE, representing California Office of the State Fire Marshal

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC FIRE SAFTEY AND IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC FIRE SAFETY

Revise as follows:

1505.2 Class A roof assemblies. Class A roof assemblies are those that are effective against severe fire test exposure. Class A roof assemblies and roof coverings shall be listed and identified as Class A by any approved testing agency. Class A roof assemblies shall be permitted for use in buildings or structures of all types of construction.

Exceptions:

1. Class A roof assemblies include those with coverings of brick, masonry, slate, clay or concrete roof tile, or an exposed concrete roof deck.
2. Class A roof assemblies also include ferrous or copper shingles or sheets, <u>metal sheets and shingles,</u> installed on non-combustible decks <u>or ferrous, copper or metal sheets installed without a roof deck</u>.

PART II – IRC BUILDING/ENERGY

Revise as follows:

R902.1 Roofing covering materials. Roofs shall be covered with materials as set forth in Sections R904 and R905. Class A, B or C roofing shall be installed in areas designated by law as requiring their use or when the edge of the roof is less than 3 feet (914 mm) from a property line. Classes A, B and C roofing required to be listed by this section shall be tested in accordance with UL 790 or ASTM E 108.

Exceptions:

1. Class A roof assemblies include those with coverings of brick, masonry, slate, clay or concrete roof tile, and exposed concrete roof deck.
2. Class A roof assemblies also include ferrous or copper shingles or sheets, metal sheets and shingles, installed on noncombustible decks <u>or ferrous, copper or metal sheets installed without a roof deck</u>.

Reason: (IBC) The purpose of this proposal is to add "metal sheets and shingles" to Exception 2 for compatibility with the current text in Section R902.1 of the IRC and to address the condition where roof decking is not needed for support of the roof covering.

The phrase "metal sheets and shingles" is needed to include aluminum or other noncombustible metal roofing.

During the discussion of code change FS199-06/07 at the ICC Public Hearings in Rochester, NY, it was clear that the concern was the flaming of the combustible deck that could cause premature failure of the roofing assembly. Many ferrous, copper and metal roof panels can be installed without the need for a supporting roof deck.

(IRC) The purpose of this proposal is to address the condition where roof decking is not needed for support of the roof covering.

During the discussion of code change FS199-06/07 at the ICC Public Hearings in Rochester, NY, it was clear that the concern was the flaming of the combustible deck that could cause premature failure of the roofing assembly. Many ferrous, copper and metal roof panels can be installed without the need for a supporting roof deck.

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

PART I – IBC FIRE SAFETY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IRC BUILDING/ENERGY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S20–07/08

1505.2, IRC R902.1

Proponent: Kate Dargan, State Fire Marshal, CAL FIRE, representing California Office of the State Fire Marshal

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC FIRE SAFTEY AND IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC FIRE SAFETY

Revise as follows:

1505.2 (Supp) Class A roof assemblies. Class A roof assemblies are those that are effective against severe fire test exposure. Class A roof assemblies and roof coverings shall be listed and identified as Class A by any approved testing agency. Class A roof assemblies shall be permitted for use in buildings or structures of all types of construction.

Exceptions:

1. Class A roof assemblies include those with coverings of brick, masonry, ~~slate, clay or concrete roof tile,~~ or an exposed concrete roof deck.
2. Class A roof assemblies also include ferrous or copper shingles or sheets, clay or concrete roof tile, or slate installed on non-combustible decks.

PART II – IRC BUILDING/ENERGY

Revise as follows:

R902.1 (Supp) Roofing covering materials. Roofs shall be covered with materials as set forth in Sections R904 and R905. Class A, B or C roofing shall be installed in areas designated by law as requiring their use or when the edge of the roof is less than 3 feet (914 mm) from a property line. Classes A, B and C roofing required to be listed by this section shall be tested in accordance with UL 790 or ASTM E 108.

1. Class A roof assemblies include those with coverings of brick, masonry, ~~slate, clay or concrete roof tile,~~ and exposed concrete roof deck.
2. Class A roof assemblies also include ferrous or copper shingles or sheets, metal sheets and shingles, clay or concrete roof tile, or slate installed on noncombustible decks.

Reason: (IBC/IRC) This is a follow-up proposal to code change FS199-06/07 that was submitted by the National Association of State Fire Marshals and approved as modified by their public comment at the ICC Final Action Hearings in Rochester, NY. The public comment cited some ASTM E108 (UL790) tests in UL Report file SV16680, Project 07CA03538 "Fact-Finding Investigation of Metal and Slate Prepared Roof Coverings" dated January 17, 2007.

Slate failed the test in less than 15 minutes but is currently included in the exception for materials that are to be considered without testing. The purpose of this proposed change is to require that decks for slate roofing or clay or concrete roof tile be noncombustible or that the roofing assembly pass the ASTM E108 (UL790) external fire and burning brand test.

Under brush fire conditions in the field, roof coverings can be exposed to burning brands that may break slate or clay or concrete roof tile, since they are brittle materials, and expose the roof deck to the fire; or the high winds caused by the brush fire can lift the butt ends of slate or concrete or clay roof tiles, allowing the entry of embers under the roof covering and igniting the combustible deck.

At the Rochester Final Action Hearing, the membership vote made it clear that fire resistance testing is favored over the exceptions based on field experience in past editions of the IBC. This change would also provide a level playing field for all of the roofing materials that would be included in exception 2.

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

PART I – IBC FIRE SAFETY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IRC BUILDING/ENERGY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S21–07/08
1505.2

Proponent: W. Lee Shoemaker, PhD, PE, Thomas Associates, Inc., representing Metal Building Manufacturers Association

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

Revise as follows:

1505.2 (Supp) Class A roof assemblies. Class A roof assemblies are those that are effective against severe fire test exposure. Class A roof assemblies and roof coverings shall be listed and identified as Class A by any approved testing agency. Class A roof assemblies shall be permitted for use in buildings or structures of all types of construction.

Exceptions:

1. Class A roof assemblies include those with coverings of brick, masonry, slate, clay or concrete roof tile, or an exposed concrete roof deck.
2. Class A roof assemblies also include ferrous or copper shingles or sheets, installed on non-combustible decks <u>or on noncombustible open framing</u>.

Reason: This proposal is intended to clarify exception 2 because in many cases the metal roof also acts as the roof deck. Such is the case in metal building systems with structural metal roofs. In metal buildings the roof panels are capable of spanning five feet between open framing that consists of either open web steel joists or cold-formed steel purlins. These structural metal roof panels do not need an additional roof deck for support. In either case the framing is non-combustible and should be included in the exception.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S22–07/08
1505.7 (New)

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

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

Add new text as follows:

<u>**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.</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 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.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S23–07/08
1505.8 (New)

Proponent: Mike Ennis, SPRI, Inc.

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

1. Add new text as follows:

<u>**1505.8 Green roofs.** The green roof system shall be installed to comply with ANSI/SPRI VF-1.</u>

2. Add new standard to Chapter 35 as follows:

Single-Ply Roofing Institute
<u>ANSI/SPRI VF-1</u> <u>Fire Design Standard Guidelines for Vegetative Roofs</u>

Reason: This code proposal addresses the fire performance of green roof systems. **Section 1507.16 Roof gardens and landscaped roofs** requires that Green Roofs meet the requirements of Chapter 15, Section 1607.11.2.2 and Section 1607.11.2.3. This Code proposal is intended to provide requirements in Chapter 15 for green roofs to resist the spread of fire on the rooftop. The referenced standard, ANSI/SPRI VF-1 Fire Design Standard Guidelines for Vegetative Roofs is currently in ANSI routing and will be completed in time for the initial Code Development Hearing.

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

Analysis: A review of the standard proposed for inclusion in the code, ANSI/SPRI VF-1, for compliance with ICC criteria for referenced standards given in Section 3.6 of Council Policy #CP 28 will be posted on the ICC website on or before January 15, 2008.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S24–07/08

1505.8 (New), 1505.8.1 (New) through 1505.8.3.2 (New)

Proponent: Mike Ennis, SPRI, Inc.

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

Add new text as follows:

1505.8 Green roofs. The green roof system shall be designed to comply with Sections 1505.8.1 through 1505.8.3.

1505.8.1 Exposed roof areas. Exposed roof areas shall meet the requirements of ASTM E108 or UL-790 for Class A roof coverings.

1505.8.2 Border zones. Border zones consisting of perimeter and corner roof areas as defined in ASCE-7 that are free of vegetation and growth media shall be provided in accordance with 1505.8.2.1 through 1505.8.2.3.

1505.8.2.1 Rooftop structures. A minimum 3 ft (0.9 m) wide continuous border zone shall be provided around rooftop structures, including but not limited to mechanical and machine rooms, penthouses, and adjacent facade walls.

1505.8.2.2 Rooftop equipment and penetrations. A minimum 1.5 ft (0.5 m) wide continuous border zone shall be provided surrounding all rooftop equipment, penetrations (e.g., ducts, drains, pipe, conduit), skylights, solar panels, antenna supports, expansion joints, roof area dividers, and interior parapet walls

1505.8.2.3 Roof area. Minimum 3 ft (0.9 m) wide continuous border zone strips shall be provided to divide the roof into areas not exceeding 15,625 ft2 (1,450 m^2), with each area not exceeding 125 ft (39 m) in length.

1505.8.3 Vegetation. Vegetation shall be maintained as described in Sections 1505.8.3.1 and 1505.8.3.2

1505.8.3.1 Irrigation. Supplemental irrigation shall be provided as necessary to maintain levels of hydration necessary to keep green roof plants alive and to keep dry foliage to a minimum.

1505.8.3.2 Dead foliage. Dead foliage and biomass shall be removed to eliminate buildup of flammable material.

Reason: This code proposal addresses the fire performance of green roof systems. **Section 1507.16 Roof gardens and landscaped roofs** requires that Green Roofs meet the requirements of Chapter 15, Section 1607.11.2.2 and Section 1607.11.2.3. This Code proposal is intended to provide requirements in Chapter 15 for green roofs to resist the spread of fire on the rooftop.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S25–07/08
1507.2.5; IRC R905.2.4

Proponent: Michael D. Fischer, The Kellen Company, representing the Asphalt Roofing Manufacturer's Association (ARMA)

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STUCTURAL AND IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

1507.2.5 Asphalt shingles. Asphalt shingles shall ~~have self-seal strips or be interlocking and~~ comply with ASTMD 225 or ASTM D 3462. ~~Asphalt shingle packaging shall bear labeling indicating compliance withASTMD3161 or a listing by an approved testing agency in accordance with the requirements of Section 1609.5.2.~~

PART II – IRC BUILDING/ENERGY

R905.2.4 Asphalt shingles. Asphalt shingles shall ~~have self-seal strips or be interlocking, and~~ comply with ASTM D 225 or D 3462.

Reason: This proposal provides a correlation fix, and removes redundant language. The prescriptive text regarding self seal strips is inappropriate. The labeling requirement already occurs within Chapter 15. The self-seal strip or interlocking terms are not defined, and are unnecessary with the introduction of requirements for testing to ASTM D7158 and ASTM D3161. Removing the terms defers appropriately to the test standards and the performance requirements.

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

PART I – IBC STRUCTURAL

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IRC BUILDING/ENERGY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S26–07/08
Table 1507.2.7; IRC Table R905.2.4.1

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

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STRUCTURAL AND IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise table as follows:

TABLE 1507.2.7 (Supp)
CLASSIFICATION OF ASPHALT ROOF SHINGLES a

MAXIMUM BASIC WIND SPEED FROM FIGURE 1609	ASTM D 3161	ASTM D 7158[b]
85	~~A,~~ D or F	D, G or H
90	~~A,~~ D or F	D, G or H
100	~~A, D or~~ F	G or H
110	F	G or H
120	~~F~~ Not permitted	G or H
130	~~F~~ Not permitted	H
140	~~F~~ Not permitted	H
150	~~F~~ Not permitted	H

For SI: 1 foot = 304.8 mm.

a. Asphalt Shingles shall be tested in accordance with ASTM D 3161 or ASTM D 7158. Refer to this table for selection of the appropriate product classification(s).
b. The standard calculations contained in ASTM D 7158 assume exposure category B or C and building height of 60 feet or less. Additional calculations are required for conditions outside of these assumptions.

PART II – IRC BUILDING/ENERGY

Revise table as follows:

TABLE R905.2.4.1 (Supp)
CLASSIFCATION OF ASPHALT ROOF SHINGLES[a]

MAXIMUM BASIC WIND SPEED FROM FIGURE 1609	ASTM D 3161	ASTM D 7158[b]
85	~~A,~~ D or F	D, G or H
90	~~A,~~ D or F	D, G or H
100	~~A, D or~~ F	G or H
110	F	G or H
120	~~F~~ Not permitted	G or H
130	~~F~~ Not permitted	H
140	~~F~~ Not permitted	H
150	~~F~~ Not permitted	H

For SI: 1 foot = 304.8 mm.

a. Asphalt shingles shall be tested in accordance with ASTM D3161 or ASTM D7158. Refer to this table for selection of the appropriate product classification(s).
b. The standard calculations contained within ASTM D7158 assume exposure category B or C and building height of 60 feet or less. Additional calculations are required for conditions outside of these assumptions.

Reason: (IBC) This proposed code change is intended correct a technical deficiency in the Code relating to the use of ASTM D3161 for determining the wind resistance of asphalt shingles. Currently, Table 1507.2.7 permits asphalt shingles classified in accordance with ASTM D3161, Class A, to be used in regions up to 110 mph basic wind speed, when these specific shingles have only been tested and are classified to a 60 mph test velocity. Similarly, Table 1507.2.7 permits asphalt shingles classified in accordance with ASTM D3161, Class D, to be used in regions up to 110 mph basic wind speed, when these specific shingles have only been tested and are classified to a 90 mph test velocity. Again, similarly, Table 1507.2.7 permits asphalt shingles classified in accordance with ASTM D3161, Class F, to be used in regions up to 150 mph basic wind speed, when these specific shingles have only been tested and are classified to a 110 mph test velocity.

ASTM D3161 includes three classifications. Shingles classified as Class A pass a test velocity of 60 mph. Shingles classified as Class D pass a test velocity of 90 mph. Shingles classified as Class F pass a test velocity of 110 mph.

Removal of allowing Class A for 85 mph and 90 mph, Class A and D for 110 mph and Class F for 120 mph through 150 mph makes the Code's implementation of ASTM D3161 consistent with the standard.

The table's implementation of classes for ASTM D7158 is correct and not in need of revision.

(IRC) This proposed code change is intended correct a technical deficiency in the Code relating to the use of ASTM D3161 for determining the wind resistance of asphalt shingles. Currently, Table R905.2.4.1 permits asphalt shingles classified in accordance with ASTM D3161, Class A, to be used in regions up to 110 mph basic wind speed, when these specific shingles have only been tested and are classified to a 60 mph test velocity. Similarly, Table R905.2.4.1 permits asphalt shingles classified in accordance with ASTM D3161, Class D, to be used in regions up to 110 mph basic wind speed, when these specific shingles have only been tested and are classified to a 90 mph test velocity. Again, similarly, Table R905.2.4.1 permits asphalt shingles classified in accordance with ASTM D3161, Class F, to be used in regions up to 150 mph basic wind speed, when these specific shingles have only been tested and are classified to a 110 mph test velocity.

ASTM D3161 includes three classifications. Shingles classified as Class A pass a test velocity of 60 mph. Shingles classified as Class D pass a test velocity of 90 mph. Shingles classified as Class F pass a test velocity of 110 mph.

Removal of allowing Class A for 85 mph and 90 mph, Class A and D for 110 mph and Class F for 120 mph through 150 mph makes the Code's implementation of ASTM D3161 consistent with the standard.

The table's implementation of classes for ASTM D7158 is correct and not in need of revision.

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

PART I – IBC STRUCTURAL

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IRC BUILDING/ENERGY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S27–07/08
1507.2.9.2

Proponent: Michael D. Fischer, The Kellen Company, representing the Asphalt Roofing Manufacturer's Association (ARMA)

Revise as follows:

1507.2.9.2 (Supp) Valleys. Valley linings shall be installed in accordance with the manufacturer=s instructions before applying shingles. Valley linings of the following types shall be permitted:

1. For open valleys (valley lining exposed) lined with metal, the valley lining shall be at least ~~16~~ 24 inches (~~406~~ 610 mm) wide and of any of the corrosion-resistant metals in Table 1507.2.9.2.
2. For open valleys, valley lining of two plies of mineral-surfaced roll roofing complying with ASTM D 3909 or ASTM D 6380 shall be permitted. The bottom layer shall be 18 inches (457 mm) and the top layer a minimum of 36 inches (914 mm) wide.
3. For closed valleys (valleys covered with shingles), valley lining of one ply of smooth roll roofing complying with ASTM D 6380, and at least 36 inches (914 mm) wide or types as described in Items 1 or 2 above shall be permitted. Self-adhering polymer modified bitumen underlayment complying with ASTM D 1970 shall be permitted in lieu of the lining material.

Reason: The IBC and IRC currently contain differing provisions for the width of exposed valley linings. This change to modify the minimum required width for exposed valley lining brings consistency between the codes and improve the weather protection performance of asphalt roofing. The 24" requirement is consistent with the Asphalt Roofing Manufacturing Association (ARMA) recommendation, reflects industry standard product widths, and will provide consistency between the IRC and IBC.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S28–07/08
1507.4.2

Proponent: Thomas W. Harding, PE, Parkline, Inc.

Revise as follows:

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

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

   <u>**Exception:** Standing seam metal roofs constructed of cold-formed steel with lap sealant applied in accordance with the roof manufacturer's installation instructions, where the roof deck acts as the roof covering and provides both weather protection and support for structural loads.</u>

Reason: 1. The purpose of this code change is to allow the continuance of good and reliable building practices that have been in effect for over fifty years but have been rendered unusable due to the wording of the code.

2. The current code is overly restrictive in face of existing evidence.

3. Parkline alone has over thirty years of experience including over 17,000 buildings using the roof system currently precluded by the current wording of the code. Manufacturers such as Armco Steel Corporation Metal Products Division (Tec-Line buildings), Butler Manufacturing (Panl-Line buildings), Parkersburg Rig & Reel ("F" and Shed buildings), Phoenix Buildings Systems, L.L.C. (Tough Line buildings), Walker-Parkersburg ("F" and Shed buildings). and Parkline, Inc. ("F", "S", and "MW" buildings) have all successfully used buildings with roof slopes less than 2-percent when installed in accordance with the requested exception.

4. A list of Parkline "F", "S" and "MW" buildings by year (quantity only) is attached. This list does not include those buildings produced by the other manufacturers referenced above. Letters from two long term erectors in support of the exemption are attached.

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

Public Hearing:				
	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S29–07/08
1507.4.4; IRC R905.10.4

Proponent: Eli P. Howard, III, Sheet Metal & Air Conditioning Contractors National Association, Inc. (SMACNA)

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STUCTURAL AND IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

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

1. Galvanized fasteners shall be used for steel roofs.
2. 300 series <u>passivated</u> stainless-steel fasteners <u>or copper nails</u> shall be used for copper roofs.
3. Stainless-steel fasteners are acceptable for all types of metal roofs.

<u>Nails shall have a minimum 12 gage [0.105 inch (3 mm)] shank with a minimum 3/8-inch (10 mm) diameter head, ASTM F 1667, of a length to penetrate through the roofing materials and a minimum of 3/4 inch (19 mm) into the roof sheathing. Where the roof sheathing is less than 3/4 inch (19 mm) thick, the fasteners shall penetrate through the sheathing. Fasteners shall comply with ASTM F 1667.</u>

PART II – IRC BUILDING/ENERGY

R905.10.4 Attachment. Metal roof panels shall be secured to the supports in accordance with this chapter and the manufacturer's installation instructions. In the absence of manufacturer's installation instructions, the following fasteners shall be used:

1. Galvanized fasteners shall be used for steel roofs.
2. Three hundred series passivated stainless steel fasteners or copper nails shall be used for copper roofs.
3. Stainless steel fasteners are acceptable for metal roofs.

Nails shall have a minimum 12 gage [0.105 inch (3 mm)] shank with a minimum 3/8-inch (10 mm) diameter head, ASTM F 1667, of a length to penetrate through the roofing materials and a minimum of 3/4 inch (19 mm) into the roof sheathing. Where the roof sheathing is less than 3/4 inch (19 mm) thick, the fasteners shall penetrate through the sheathing. Fasteners shall comply with ASTM F 1667.

Reason: (IBC) Stainless nails should be passivated to be compatible with copper panels and cleats, refer to the galvanic chart below to see that not all 300 series stainless is compatible with copper. While most 300 series fasteners meet this requirement it needs to be clearly stated in the code to assure compliance.

Copper nails should also be a code-approved fastening method for copper panels as they have been used for decades successfully and copper nails are allowed for shingled roofs – see **1507.2.6 Fasteners.** If there is concern for specifying nails for copper panels then there should also be similar language included such as that in **1507.2.6 Fasteners** to assure that the appropriate type and length fasteners are used.

ANODIC/CORRODED END/LEAST NOBLE
Zinc
Aluminum
Galvanized Steel
Cadmium
Mild Steel, Wrought Iron
Cast Iron
Stainless Steel, types 304 and 316 (active)*
Lead–tin Solder
Lead
Brass, Bronze
Copper
Stainless Steel, types 304 and 316 (passive)
CATHODIC/PROTECTED END/MOST NOBLE
*Active Stainless Steel has not been chemically cleaned.

(IRC) Stainless nails should be passivated to be compatible with copper panels and cleats, refer to the galvanic chart below to see that not all 300 series stainless is compatible with copper. While most 300 series fasteners meet this requirement it needs to be clearly stated in the code to assure compliance.

Copper nails should also be a code-approved fastening method for copper panels as they have been used for decades successfully and copper nails are allowed for shingled roofs – see **R905.2.5 Fasteners.** If there is concern for specifying nails for copper panels then there should also be similar language included such as in R905.25 to assure that the appropriate type and length fasteners are used.

ANODIC/CORRODED END/LEAST NOBLE
Zinc
Aluminum
Galvanized Steel
Cadmium
Mild Steel, Wrought Iron
Cast Iron
Stainless Steel, types 304 and 316 (active)*
Lead–tin Solder
Lead
Brass, Bronze
Copper
Stainless Steel, types 304 and 316 (passive)
CATHODIC/PROTECTED END/MOST NOBLE
*Active Stainless Steel has not been chemically cleaned.

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

PART I – IBC STRUCTURAL

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IRC BUILDING/ENERGY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S30–07/08
1507.4.4; IRC R905.10.4

Proponent: Craig Thompson, Copper Development Association

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STRUCTURAL AND IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

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

1. Galvanized fasteners shall be used for steel roofs.
2. <u>Copper, Brass, Bronze, copper alloy and</u> 300 series stainless-steel fasteners shall be used for copper roofs.
3. Stainless-steel fasteners are acceptable for all types of metal roofs.

PART II – IRC BUILDING/ENERGY

Revise as follows:

R905.10.4 Attachment. Metal roof panels shall be secured to the supports in accordance with this chapter and the manufacturer's installation instructions. In the absence of manufacturer's installation instructions, the following fasteners shall be used:

1. Galvanized fasteners shall be used for steel roofs.
2. <u>Copper, Brass, Bronze, copper alloy and</u> Three hundred series stainless steel fasteners shall be used for copper roofs.
3. Stainless steel fasteners are acceptable for metal roofs.

Reason: (IBC) This change is being submitted to overcome an omission in the code regarding industry recommended fastener materials for use with copper and copper alloy roof systems. Traditionally, copper and copper alloy roofs have been installed using only copper alloy fasteners (copper, brass, bronze, etc.) in order to avoid issues related to dissimilar metal or galvanic corrosion problems between the copper alloy roof material and the fastener. More recently, stainless steel fasteners have also been allowed, and are acceptable in addition to copper and copper alloy fasteners, not in exclusion of those fasteners. As currently written, this section of the code is unnecessarily restrictive. As shown in the three references listed below, copper and copper alloy fasteners remain the industry recommended materials in addition to stainless steel for installing copper and copper alloy roofs.
<u>Reference Publications:</u>
The following publications list the types of fasteners to be used with copper roofs:
A. SMACNA's "Architectural Sheet Metal Manual"
B. Revere's "Copper and Common Sense"
C. Copper Development Association's "Copper in Architecture"

(IRC) This change is being submitted to overcome an omission in the code regarding industry recommended fastener materials for use with copper and copper alloy roof systems. Traditionally, copper and copper alloy roofs have been installed using only copper alloy fasteners (copper, brass, bronze, etc.) in order to avoid issues related to dissimilar metal or galvanic corrosion problems between the copper alloy roof material and the fastener. More recently, stainless steel fasteners have also been allowed, and are acceptable in addition to copper and copper alloy fasteners, not in exclusion of those fasteners. As currently written, this section of the code is unnecessarily restrictive. As shown in the three references listed below, copper and copper alloy fasteners remain the industry recommended materials in addition to stainless steel for installing copper and copper alloy roofs.
<u>Reference Publications:</u>
The following publications list the types of fasteners to be used with copper roofs:
A. SMACNA's "Architectural Sheet Metal Manual"
B. Revere's "Copper and Common Sense"
C. Copper Development Association's "Copper in Architecture"

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

PART I – IBC STRUCTURAL

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IRC BUILDING/ENERGY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S31 –07/08
1507.8, Table 1507.8, 1507.9

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

1. Revise as follows:

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

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

2. Delete table without substitution:

~~TABLE 1507.8 (Supp)~~
~~WOOD SHINGLE AND SHAKE INSTALLATION~~

~~ROOF ITEM~~	~~WOOD SHINGLES~~	~~WOOD SHAKES~~
~~1. Roof slope~~	~~Wood shingles shall be installed on slopes of three units vertical in 12 units horizontal (3:12) or greater.~~	~~Wood shakes shall be installed on slopes of four units vertical in 12 units horizontal (4:12) or greater.~~
~~2. Deck requirement~~	~~--~~	~~--~~
~~Temperate climate~~	~~Shingles shall be applied to roofs with solid or spaced sheathing. Where spaced sheathing is used, sheathing boards shall not be less than 1" x 4" nominal dimensions and shall be spaced on center equal to the weather exposure to coincide with the placement of fasteners.~~	~~Shakes shall be applied to roofs with solid or spaced sheathing. Where spaced sheathing is used, sheathing board shall not be less than 1" x 4" nominal dimensions and shall be spaced on center equal to the weather exposure to coincide with the placement of fasteners. When 1" x 4" spac3ed sheathing is installed at 10 inches, boards must be installed between the sheathing boards.~~
~~In areas where the average daily temperature in January is 25F or less or where there is a possibility of ice forming along the eaves causing a backup of water~~	~~Solid sheathing is required.~~	~~Solid sheathing is required.~~
~~3. Interlayment~~	~~No requirements.~~	~~Interlayment shall comply with ASTM D226, Type I.~~
~~4. Underlayment~~	~~--~~	~~--~~
~~Temperate climate~~	~~Underlayment shall comply with ASTM D226, Type I.~~	~~Underlayment shall comply with ASTM D226, Type I.~~
~~In areas where there is a possibility of ice forming along the eaves causing a backup of water.~~	~~An ice shield that consists of at least two layers of underlayment cemented together or a self-adhering polymer modified bitumen sheet shall extend from the eave's edge to a point at least 24 inches inside the exterior wall line of the building.~~	~~An ice shield that consists of at least two layers of underlayment cemented together or of a self-adhering polymer-modified bitumen sheet shall extend from the eave's edge to a point at least 24 inches inside the exterior wall line of the building.~~
~~5. Application~~	~~--~~	~~--~~
~~Attachment~~	~~Fasteners for wood singles shall be corrosion resistant with a minimum penetration of 0.75 inch into the sheathing. For sheathing less than 0.5 inch thick, the fasteners shall extend through the sheathing.~~	~~Fasteners for wood shakes shall be corrosion resistant with a minimum penetration of 0.75 inch into the sheathing. For sheathing less than 0.5 inch thick, the fasteners shall extend through the sheathing.~~
~~No. of fasteners~~	~~Two per shingle.~~	~~Two per shake.~~
~~Exposure~~	~~Weather exposures shall not exceed those set forth in Table 1507.8.6.~~	~~Weather exposures shall not exceed those set forth in Table 1507.9.7.~~
~~Method~~	~~Shingles shall be laid with a side lap of not less than 1.5 inches between joints in courses, and no two joints in any three adjacent courses shall be in direct alignment. Spacing between shingles shall be 0.25 to 0.375 inch.~~	~~Shakes shall be laid with a side lap of not less than 1.5 inches between joints in adjacent courses. Spacing between shakes shall not be less than 0.375 inch or more than 0.625 inch for shakes and tapersawn shakes of naturally durable wood and shall be 0.25 to 0.375 inch for preservative taper sawn shakes.~~
~~Flashing~~	~~In accordance with Section 1507.8.7.~~	~~In accordance with Section 1507.9.8.~~

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

Reason: This proposed code change is intended to clarify the intent of the Code's requirements for wood shingle and wood shake roof systems. When the Code was originally developed, Table 1507.8 was added to provide a summary of the requirements in the text portions of Section 1507.8—Wood Shingles and Section 1507.9—Wood Shakes. During the code development cycles since the Code was originally published, changes to the text in Sec. 1507.8 and Section 1507.9 have not consistently been added to Table 1507.8. As a result, the Code's requirements in Table 1507.8 are no longer consistent with Sections 1507.8 and Section 1507.9. Eliminating Table 1507.8 eliminates this inconsistency within the Code.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S32–07/08

1507.10.1, 1507.10.3 (New)

Proponent: Michael D. Fischer, The Kellen Company, representing the Asphalt Roofing Manufacturer's Association (ARMA)

Revise as follows:

1507.10 Built-up roofs. The installation of built-up roofs shall comply with the provisions of this section.

1507.10.1 Slope. Built-up roofs shall have a design slope of a minimum of one-fourth unit vertical in 12 units horizontal (2-percent slope) for drainage~~,~~ <u>.</u>

> **<u>Exception:</u>** ~~except for~~ Coal-tar or <u>coal-tar surfaced</u> built-up roofs that shall have a design slope of a minimum one-eighth unit vertical in 12 units horizontal (1-percent slope).

1507.10.2 Material standards. Built-up roof covering materials shall comply with the standards in Table 1507.10.2.

<u>**1507.10.3 Liquid-applied roof coverings.** Liquid-applied roof coverings used on built-up roofs shall comply with 1507.15.</u>

Reason: The design slope exception for coal-tar is for surface on BUR, not for a coal-tar roof system. The addition of 1507.15 creates a pointer to the roof-coating requirements.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S33–07/08

1507.11.2, Chapter 35 (New)

Proponent: Kenneth R. Hunt, Performance Roof Systems, Inc., representing himself

1. Revise as follows:

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

2. Add standard to Chapter 35 as follows:

ASTM

<u>D 6509-00</u> <u>Standard Specification for Atactic Polypropylene (APP) Modified Bituminous Base Sheet Materials Using Glass Fiber Reinforcements</u>

Reason: 1) The inclusion of ASTM D 6509 is to revised the current material standards to include all current ASTM Modified Bitumen roof covers. 2) The current code does not include the approval of an ASTM D 6509 APP Modified Bitumen Membrane thus restricting the use of a modified base sheet/ply to SBS only. 3) See attached copy of ASTM 6509.

Cost Impact: The code change proposal will not increase the cost of construction. The inclusion of ASTM D 6509 into the code will not increase cost of construction.

Analysis: A review of the standard proposed for inclusion in the code, ASTM D6509, for compliance with ICC criteria for referenced standards given in Section 3.6 of Council Policy #CP 28 will be posted on the ICC website on or before January 15, 2008.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S34–07/08

1507.12.3, 1507.13.3, Chapter 35 (New)

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

1. Revise as follows:

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

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

2. Add standard to Chapter 35 as follows:

ASTM

D 448-03a Standard Classification for Sizes of Aggregate for Road and Bridge Construction

Reason: This proposed code change is intended to clarify the intent of the Code's current requirements for ballasted single-ply membrane roof systems by providing specific requirements for stone ballast. ASTM D448 is already included in ANSI/SPRI RP-4, which is referenced in Sec. 1504.4, as a requirement for the size gradation of stone used as ballast. The proposed code change does not change the Code's size gradation requirement. It only includes it within the Code's text for ease of compliance and enforcement.

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

Analysis: A review of the standard proposed for inclusion in the code, ASTM D488, for compliance with ICC criteria for referenced standards given in Section 3.6 of Council Policy #CP 28 will be posted on the ICC website on or before January 15, 2008.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S35–07/08

1507.14.2; IRC R905.14.2

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

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STRUCTURAL AND IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

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

PART II – IRC BUILDING/ENERGY

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.

PART I – IBC STRUCTURAL

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IRC BUILDING/ENERGY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S36–07/08
1507.15.1

Proponent: Michael D. Fischer, The Kellen Company, representing the Asphalt Roofing Manufacturer's Association (ARMA)

Revise as follows:

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

1507.15.1 Slope. Liquid-applied roofs shall have a design slope of a minimum of one-fourth unit vertical in 12 units horizontal (2-percent slope).

> **Exception:** Coal-tar surfaced roofs shall have a design slope of a minimum one-eighth unit vertical in 12 units horizontal (1-percent slope).

Reason: This proposal reconciles a slope difference between the text in 1507.15.1 and 1507.10.1 in order to clarify the applicable slope limitations.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S37–07/08
1507.15.2, Chapter 35 (New); IRC R905.15.2, Chapter 43 (New)

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

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STRUCTURAL AND IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

1. Revise as follows:

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

1507.15.1 Slope. Liquid-applied roofs shall have a design slope of a minimum of one-fourth unit vertical in 12 units horizontal (2-percent slope).

1507.15.2 Material standards. Liquid-applied roof coatings shall comply with ASTM C 836, ASTM C 957, ASTM D 1227 or ASTM D 3468, ASTM D 6083, ASTM D 6694, or ASTM D 6947.

2. Add standard to Chapter 35 as follows:

ASTM

D 6947-07 Standard Specification for Liquid Applied Moisture Cured Polyurethane Coating Used in Spray Polyurethane Foam Roofing System

PART II – IRC BUILDING/ENERGY

1. Revise as follows:

R905.15 Liquid-applied coatings. The installation of liquid applied coatings shall comply with the provisions of this section.

R905.15.1 Slope. Liquid-applied roofs shall have a design slope of a minimum of one-fourth unit vertical in 12 units horizontal (2-percent slope).

R905.15.2 Material standards. Liquid-applied roof coatings shall comply with ASTM C 836, C 957, D 1227, D 3468, D 6083, D 6694, or ASTM D 6947.

R905.15.3 Application. Liquid-applied roof coatings shall be installed according to this chapter and the manufacturer's installation instructions.

2. Add standard to Chapter 43 as follows:

ASTM

D 6947-07 Standard Specification for Liquid Applied Moisture Cured Polyurethane Coating Used in Spray Polyurethane Foam Roofing System

Reason: Moisture cured polyurethane coatings are widely used in the SPF industry and this code change would specifically allow the use of a viable alternative to the coatings already referenced in IBC section 1507.15.2 and IRC section R905.15.2

The code change allows the use of moisture cured polyurethane coatings over spray polyurethane foam roof systems meeting the ASTM standard 6947.

ASTM D 6947, is a new ASTM standard adopted in 2006 to describe liquid-applied moisture cured polyurethane coatings that are used in spray polyurethane foam roofing.

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

Analysis: A review of the standard proposed for inclusion in the code, ASTM D6947, for compliance with ICC criteria for referenced standards given in Section 3.6 of Council Policy #CP 28 will be posted on the ICC website on or before January 15, 2008.

PART I – IBC STRUCTURAL

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IRC BUILDING/ENERGY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S38–07/08

1507.15.2, Table 1507.15.2 (New), Chapter 35 (New)

Proponent: Michael D. Fischer, The Kellen Company, representing the Roof Coating Manufacturer's Association (RCMA)

1. Revise as follows:

1507.15.2 Material standards. Liquid-applied roof coatings shall comply with the standards in Table 1507.15.2 ~~ASTM C 836, ASTM C 957, ASTM D 1227 or ASTM D 3468, ASTM D 6083 or ASTM D 6694~~.

2. Add new table as follows:

TABLE 1507.10.2
ROOF COATING MATERIAL STANDARDS

MATERIAL STANDARD	STANDARD
Acrylic Roof Coatings	ASTM D 6083
Aluminum Coatings	ASTM D 2824
Aluminum-Pigmented Emulsified Asphalt	ASTM D 6848
Asphalt & Modified Bitumen Adhesives	ASTM D 3019
Asphalt Cements	ASTM D 3409 ASTM D 4586
Asphalt Emulsion Coatings	ASTM D 1227
Asphalt Emulsions	ASTM D 1187
Asphalt Primer	ASTM D 41
Asphalt Roof Coatings	ASTM D 4479
Coal Tar Cements	ASTM D 4022
Coal Tar Roof Coatings	ASTM D 450A
Elastomeric Membranes	ASTM C 836
Elastomeric Waterproofing Membrane with Integral Wearing Surface	ASTM C 957
Liquid Applied Neoprene Coating	ASTM D 3468
Liquid Silicone Coating	ASTM D 6694

3. Add standards to Chapter 35 as follows:

ASTM International

D 1187 97(2002)e1	Standard Specification for Asphalt-Base Emulsions for Use as Protective Coatings for Metal
D 2824 06	Standard Specification for Aluminum-Pigmented Asphalt Roof Coatings, Nonfibered, Asbestos Fibered, and Fibered without Asbestos
D3409-93(2002)^{e1}	Standard Test Method for Adhesion of Asphalt-Roof Cement to Damp, Wet, or Underwater Surfaces
D 6848 02	Standard Specification for Aluminum Pigmented Emulsified Asphalt Used as a Protective Coating for Roofing

Reason: This proposal will include a reorganization of the referenced standards requirements for roof coatings, and adopts the similar approach as the existing BUR standards referenced table.

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

Analysis: A review of the standards proposed for inclusion in the code, ASTM D2824, D6848, D1187 and D3409, for compliance with ICC criteria for referenced standards given in Section 3.6 of Council Policy #CP 28 will be posted on the ICC website on or before January 15, 2008.

Public Hearing:				
	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S39–07/08
1507.15.3 (New), Chapter 35 (New)

Proponent: Michael D. Fischer, The Kellen Company, representing The Roof Coating Manufacturer's Association (RCMA)

1. Add new text as follows:

1507.15.3 Application. Roof coatings shall be applied in accordance with approved manufacturers installation instructions.

1507.15.3.1 Surface preparation. All surfaces shall be prepared in accordance with approved manufacturers installation instructions prior to the application of roof coatings.

1507.15.3.1.1 Concrete surface preparation. Concrete surfaces shall be prepared in accordance with approved manufacturers installation instructions and ASTM D4261**.**

1507.15.3.2 Aluminum-Pigmented Asphalt Roof Coatings. Aluminum-pigmented asphalt roof coatings shall be applied in accordance with approved manufacturers installation instructions and ASTM D3805.

2. Add standards to Chapter 35 as follows:

ASTM

D 3805-97(2003)^{e1}	Standard Guide for Application of Aluminum-Pigmented Asphalt Roof Coatings
D 4261-05	Standard Practice for Surface Cleaning Concrete Unit Masonry for Coating

Reason: This new section will help ensure that roof coating manufacturer's instructions and proper surface preparation are considered and followed.

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

Analysis: A review of the standard proposed for inclusion in the code, ASTM D4261 and D3805, for compliance with ICC criteria for referenced standards given in Section 3.6 of Council Policy #CP 28 will be posted on the ICC website on or before January 15, 2008.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S40–07/08
1507.16

Proponent: Steven Peck, Green Roofs for Healthy Cities

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. An accredited Green Roof professional shall design these roofs for wind and fire resistance.

Reason: The Code requires that roof gardens and landscaped roofs be designed to resist wind loads and fire spread. No standards exist to meet this requirement. Green Roofs for Healthy Cities conducts training programs on the design and maintenance of these systems. Once the course work is completed the participant is registered as an accredited green roof professional and will properly design these roof systems to resist design wind loads and fire spread.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S41–07/08
1507.16

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.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S42–07/08
1502.1, 1509.2, 1509.2.1 through 1509.2.3, 1509.2.4 (New)

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

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC GENERAL AND IBC MEANS OF EGRESS CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC GENERAL

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. <u>A penthouse or penthouses in compliance with Section 1509.2 shall not be considered as a portion of the story below.</u>

<u>1509.2.1 Height above roof.</u> 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.

<u>1509.2.2 Area limitation.</u> The aggregate area of penthouses and other rooftop structures shall not exceed one-third the area of the supporting roof. <u>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.</u>

<u>1509.2.3 Use limitations.</u> 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.

PART II – IBC MEANS OF EGRESS

Add new text as follows:

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.

Exception: A single means of egress shall be permitted in accordance with Section 1015.1.

(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 I – IBC GENERAL

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IBC MEANS OF EGRESS

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S43–07/08
1509.2.1

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

THIS PROPOSAL IS ON THE AGENDA OF THE IBC GENERAL CODE DEVELOPMENT COMMITTEE. SEE THE TENTATIVE HEARING ORDER FOR THIS COMMITTEE.

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 IA 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 IA construction two stories above grade plane or less in height, and Type IB 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.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S44–07/08
1510.3; IRC R907.3

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

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STRUCTURAL AND IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTUAL

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, cement or asbestos-cement tile.
3. Where the existing roof has two or more applications of any type of roof covering.
4. Where the basic wind speed equals or exceeds 110 mph and the new roof covering relies on adhesive sealants for all or part of it's wind resistance, unless the new roof covering has be tested indicating equivalent performance for installation over existing roofs.

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 buildings 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.

PART II – IRC BUILDING/ENERGY

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 or asbestos-cement tile.
3. Where the existing roof has two or more applications of any type of roof covering.
4. For asphalt shingles, when the building is located in an area subject to moderate or severe hail exposure according to Figure R903.5.
5. <u>Where the basic wind speed equals or exceeds 110 mph and the new roof covering relies on adhesive sealants for all or part of it's wind resistance, unless the new roof covering has be tested indicating equivalent performance for installation over existing roofs.</u>

   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: (IBC/IRC) This proposal is intended to ensure that when new roof coverings that are required to be tested are installed over existing roof coverings, that the new roof covering has been tested and shown to perform equivalently over an existing roof as compared to installation directly to the roof deck. The primary concern is the wind resistance of roof coverings, such as asphalt shingles, installed over existing roof coverings. The tests for asphalt shingles wind resistance are conducted with the shingles attached directly to the roof deck. We are concerned about the ability of these tested systems to achieve the same level of resistance when installed over another roof covering.

This proposal specifically complies with intent of the code in that, while not stated, it is the intent that tested products, assemblies, etc. are to be installed in the same manner for which the tests were performed. During the last code cycle, the code committees argued that there were no studies or evidence to justifying this change. However, there also are no studies or evidence suggesting that new roof coverings installed over existing roofs will perform equivalently to new roof coverings installed directly to the roof deck. We believe this proposal meets the intent of the code.

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

PART I – IBC STRUCTURAL

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IRC BUILDING/ENERGY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S45–07/08

1502 (New), 1511 (New)

Proponent: Mark J. Blomquist, Top Of The Line Unlimited, LLC, representing Snowgrip Snow Retention Coating

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

1. Add new definitions as follows:

SECTION 1502
DEFINITIONS

<u>**DISCHARGE POINT.** The lowest edge of a roof surface that allows roof drainage to leave the roof.</u>

DISCHARGE ZONE. Any areas of the site, neighboring sites, public way or building structure that are in the path of a hazardous roof discharge.

HAZARDOUS ROOF DISCHARGE. Any material, such as rain, snow and ice, that leaves a roof or other upward facing horizontal surfaces and possesses a weight, volume, mass or velocity that is likely to cause great bodily harm to any person that is in the path of such discharge.

SNOW RETENTION. Building design and/or construction means and methods that prevent hazardous roof discharge.

2. Add new section as follows:

SECTION 1511
REQUIREMENTS FOR SNOW RETENTION

1511.1 General. Building roofs and other upward facing horizontal surfaces that provide support for the accumulation of a significant volume of snow and ice shall be designed and constructed to prevent a hazardous roof discharge onto any public-use areas of the site, neighboring sites, public way or exterior occupiable space on elevated building structures, such as decks or walkways.

Exceptions:

1. Snow retention is not required when the Discharge Point is less than 3 feet (914 mm) above the walking surface of the Discharge Zone.
2. Snow retention is not required when physical barriers and signage are provided to prevent the occupancy of Discharge Zones.
3. Snow retention is not required on vertical surfaces and upward facing horizontal surfaces that have a slope steeper than 12 vertical units for each horizontal unit.
4. Snow retention is not required on roof surfaces that have a sufficiently abrasive surface that bonds the snow layer to the roof to prevent sliding and hazardous roof discharge.

Reason: 1. The purpose of this proposed new code requirement is to require measures to reduce or eliminate the preventable injury caused by the uncontrolled discharge of snow and ice from buildings.

2. The Current code is silent on requirements to prevent hazardous roof discharges. Far too many designers and builders are ignoring very predictable, reoccurring, natural conditions that create the high probability of personal injury including death from hazardous roof discharges. The insurance industry and others have preferred to label the problem discharges as an “act of god”, but many designers with experience in snow country know that locating a pedestrian way below the discharge point of a standing seem metal roof is a recipe for disaster. The disaster is predictable and therefore preventable. Now all that remains is for the IBC to enact requirements to impose reasonable and logical measures to direct more response design and construction on individuals in the industry that either don’t understand the problem or chose to ignore it.

3. a) Attachments show photos of damage caused by hazardous roof discharges. The type of damage is so significant that it is obvious such an occurrence could have easily caused personal injury or death. b) Attachments show photos of conditions that show conditions that represent an obvious health safety risk. c) Attachments show 3 signs that thousands of people just walking the sidewalk to work are exposed to significant risk that is uncontrolled in our high-rise urban districts and disaster could strike at any time. d) Attachments show 3 research articles. e) Attachments show 2 published reports of personal injury including death.

4. Bibliography: a) & b) Personal photos. c) Public photos from internet sources. d) 3 articles from internet sources with references attached. e) 2 articles from internet sources with references attached.

In some cases, there will be little or no additional costs for proactive design and the development of safe unoccupied discharge zones. In others, particularly dense urban areas, designers are going to have to create fewer conditions that create potential hazards, but the resulting design solutions will not necessarily increase construction costs. The average construction cost increase is likely to be less than 1% of the cost of construction and will be easily paid back in reduced energy and maintenance costs over the first couple years. The reduction in the immeasurable pain and suffering associated with an unnecessary and preventable accident is well worth the minor and controllable initial increase in construction costs.

Blomquist-FS1-3-3.jpg

Blomquist-FS1-4-2.jpg


Blomquist-FS1-falling_ice_1.jpg

Blomquist-FS1-ice_forming_on_roof.jpg


Blomquist-FS1-not_good_at_all.jpg


Blomquist-FS1-on_sill_wo_snowgrip.jpg


Blomquist-FS1-power_of_ice_damage.jpg


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Blomquist-FS1-wo_6-2.jpg

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S46–07/08
1602.1, 1609.2, 1613.2, 1808.1, 2302.1

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

Revise as follows:

1602.1 Definitions. The following words and terms shall, for the purposes of this chapter <u>and as used elsewhere in this code</u>, have the meanings shown herein.

1609.2 Definitions. The following words and terms shall, for the purposes of Section 1609 <u>and as used elsewhere in this code</u>, have the meanings shown herein.

1613.2 Definitions. The following words and terms shall, for the purposes of this section <u>and as used elsewhere in this code</u>, have the meanings shown herein.

1808.1 Definitions. The following words and terms shall, for the purposes of this section <u>and as used elsewhere in this code</u>, have the meanings shown herein.

2302.1 Definitions. The following words and terms, shall, for the purposes of this chapter <u>and as used elsewhere in this code</u>, have the meanings shown herein.

Reason: The charging language in each of the sections of the nonstructural chapters (other than Chapters 16-23) of the 2006 IBC listing definitions consistently states "for the purposes of this chapter and as used elsewhere in this code." This is not the case, however, in the structural chapters. The changes above are proposed to achieve consistency in the charging language throughout the code. The one exception to this is Section 1612.1 listing the definitions unique to flood loads. It appears that a distinction between structural and nonstructural definitions is intended but "as used elsewhere in this code" appears in the current 2006 language of Sections 1702.1, 1902.1, 2102.1 and 2202.1. An alternative would be to delete the phrase from these sections to clarify the distinction but it also appears that such a distinction will not be recognized by most code users and may be counter-productive.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S47–07/08

1603.1, 1603.1.6 (New), 1802.6

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. Design load-bearing values of soils.

2. Add new text as follows:

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

(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.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S48–07/08
1603.2, 1603.3, 1603.4, 106 (New)

Proponent: Philip Brazil, PE, SE, Reid Middleton, representing himself

Revise as follows:

SECTION 106
FLOOR AND ROOF DESIGN LOADS

~~1603.2~~ **106.1 Restrictions on loading.** It shall be unlawful to place, or cause or permit to be placed, on any floor or roof of a building, structure or portion thereof, a load greater than is permitted by these requirements.

~~1603.3~~ **106.2 Live loads posted.** Where the live loads for which each floor or portion thereof of a commercial or industrial building is or has been designed to exceed 50 psf (2.40 kN/m^2), such design live loads shall be conspicuously posted by the owner in that part of each story in which they apply, using durable signs. It shall be unlawful to remove or deface such notices.

~~1603.4~~ **106.3 Occupancy permits for changed loads.** Occupancy permits for buildings hereafter erected shall not be issued until the floor load signs, required by Section ~~1603.3~~ 106.2, have been installed.

(Renumber subsequent sections)

Reason: Chapter 16 governs the structural design of buildings and structures and consists primarily of provisions for the determination of minimum structural design loads. Section 1603, in particular contains requirements for specifying in the construction documents design loads for each building or structure. Sections 1603.2 through 1603.4, however, are concerned with restrictions on the loads imposed on floors and roofs, the posting of live loads and withholding of the issuance of occupancy permits until live load signs are posted. These particular provisions are more related to the enforcement of codes and standards by building officials rather than the specification of structural design loads. They are better located in the administrative provisions of Chapter 1 rather than in the structural design provisions of Chapter 16.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S49–07/08
1604.1, 1604.2, 1609.1

Proponent: Philip Brazil, PE, SE, Reid Middleton, representing himself

Revise as follows:

1604.1 General. Building, structures and ~~parts~~ portions thereof shall be designed and constructed in accordance with strength design, load and resistance factor design, allowable stress design, empirical design or conventional construction methods, as permitted by the applicable material chapters.

1604.2 Strength. Buildings, ~~and other~~ structures, and ~~parts~~ portions thereof~~,~~ shall be designed and constructed to support safely the factored loads in load combinations defined in this code without exceeding the appropriate strength limit states for the materials of construction. Alternatively, buildings and other structures, and parts thereof, shall be designed and constructed to support safely the nominal loads in load combinations defined in this code without exceeding the appropriate specified allowable stresses for the materials of construction.

Loads and forces for occupancies or uses not covered in this chapter shall be subject to the approval of the building official.

1609.1 Applications. Buildings, structures and ~~parts~~ portions thereof shall be designed to withstand the minimum wind loads prescribed herein. Decreases in wind loads shall not be made for the effect of shielding by other structures.

Reason: The changes are proposed for consistency with the use of "portions thereof" elsewhere in the 2006 IBC (approximately 25 code sections). The sections above contain the only instances of "parts thereof" in conjunction with buildings or structures in the 2006 IBC.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S50–07/08

Table 1604.3; IRC Table R301.7

Proponent: Daniel J. Walker, PE, Thomas Associates, Inc., representing National Sunroom Association

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STRUCTURAL AND IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise table footnotes as follows:

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

h. For aluminum structural members or aluminum panels used in skylights and sloped glazing framing, roofs or walls of sunroom additions or patio covers, not supporting edge of glass or aluminum sandwich panels, the total load deflection shall not exceed l/60. <u>For continuous aluminum structural members supporting edge of glass, the total load deflection shall not exceed l/175 for each glass lite or l/60 for the entire length of the member, whichever is more stringent.</u> For aluminum sandwich panels used in roofs or walls of sunroom additions or patio covers, the total load deflection shall not exceed l/120.

(Portions of table and footnotes not shown remain unchanged)

PART II – IRC BUILDING/ENERGY

Revise table footnotes as follows:

TABLE R301.7 (Supp)
ALLOWABLE DEFLECTION OF STRUCTURAL MEMBERS[a,b,c]

c. For aluminum structural members or panels used in roofs or walls of sunroom covers, not supporting edge of glass or sandwich panels, the total load deflection additions or patio shall not exceed L/60. <u>For continuous aluminum structural members supporting edge of glass, the total load deflection shall not exceed L/175 for each glass lite or L/60 for the entire length of the member, whichever is more stringent.</u> For sandwich panels used in roofs or walls of sunroom additions or patio covers, the total load deflection shall not exceed L/120.

(Portions of table and footnotes not shown remain unchanged)

Reason: The proposed modification provides clarification that the deflection limit for any edge of glass application shall not exceed l/175. The way the code currently reads is ambiguous and does not specifically reference the l/175 limit from ASTM E 1300, "Standard Practice for Determining Load Resistance of Glass in Buildings", which is referenced in IBC Chapter 24 and is widely accepted in the glazing industry. The added language improves code enforcement by including this important deflection limit

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

PART I – IBC STRUCTURAL

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IRC BUILDING/ENERGY

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S51–07/08
1604.3.3

Proponent: Bonnie Manley, American Iron and Steel Institute, representing American Iron and Steel Institute

Revise as follows:

1604.3.3 Steel. The deflection of steel structural members shall not exceed that permitted by AISC 360, AISI S100 ~~NAS, AISI-General, AISI-Truss~~, ASCE 3, ASCE 8, SJI JG-1.1, SJI K-1.1 or SJI LH/DLH-1.1, as applicable.

Reason: This code change proposal updates the reference to AISI's *North American Specification for the Design of Cold-Formed Steel Structural Members*, 2007 edition, which has been given the new number designation of AISI S100. It also deletes the references to AISI General and AISI Truss, since neither document addresses deflection of cold-formed steel light frame construction from a serviceability standpoint. AISI S100 handles deflection criteria for cold-formed steel members.

Please see companion change to IBC Section 2209.1 for a summary of changes to AISI S100-07.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S52–07/08
1604.5, 1604.5.1

Proponent: Philip Brazil, PE, SE, Reid Middleton, representing himself

Revise as follows:

1604.5 Occupancy category. Buildings and structures shall be assigned ~~an~~ occupancy ~~categoryies~~ categories in accordance with Table 1604.5.

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

Reason: Table 1604.5 consistently references buildings and structures in assigning occupancy categories. Section 1604.5, however, references buildings but not structures. Section 1604.5.1 references structures but not buildings. The purpose of this proposal is to correct these oversights so that Sections 1604.5 and 1604.5.1 are consistent with Table 1604.5.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S53–07/08
Table 1604.5

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: • Agricultural facilities. • Certain temporary facilities. • Minor storage facilities. • <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.

Public Hearing:				
	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S54–07/08
Table 1604.5

Proponent: Thomas Kinsman, T. A. Kinsman Consulting Company, representing himself

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~~ Occupancies and uses that represent a ~~low~~ minimum hazard to human life in the event of building or structure failure, including but not limited to: • Agricultural ~~facilities~~ uses. • Certain temporary ~~facilities~~ uses. • Minor storage ~~facilities~~ uses.
II	~~Buildings and other structures~~ Occupancies and uses that represent low hazard to human life are all occupancies and uses except those listed in Occupancy Categories I, III and IV
III	~~Buildings and other structures~~ Occupancies and uses that represent a ~~substantial~~ moderate hazard to human life in the event of building or structure failure due to (1) inability of occupants to self evacuate in an emergency, (2) a large occupant load, (3) people in the vicinity, or (4) that represent a substantial public need including but not limited to: • ~~Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300.~~ ~~• Buildings and other structures containing elementary school, secondary school or day care facilities with an occupant load greater than 250.~~ ~~• Buildings and other structures containing adult education facilities, such as colleges and universities with an occupant load greater than 500.~~ • Group I-2 ~~occupancies~~ nursing homes, mental hospitals, and detoxification facilities with an occupant load of 50 or more resident patients ~~but not having surgery or emergency treatment facilities~~. • Group I-3 occupancies. • ~~Any other occupancy~~ Occupancies and uses with ~~an~~ a total occupant load greater than 5,000[a] in one building or structure. • Power-generating stations, water treatment facilities for potable water, waste water treatment facilities and other public utility facilities not included in Occupancy Category IV. • Buildings and other structures not included in Occupancy Category IV containing sufficient quantities of toxic ~~or explosive~~ substances to be dangerous to the public if released[b].
IV	~~Buildings and other structures designated as essential facilities~~ Occupancies and uses that represent either (1) an essential public need during or immediately after a disaster or (2) a substantial hazard to the public in the vicinity of a building in the event of building failure, including but not limited to: • Group I-2 occupancies ~~having~~ , with or without surgery or emergency treatment facilities. • Fire, rescue, ambulance and police stations and emergency vehicle garages. • Designated earthquake, hurricane or other emergency shelters. • Designated emergency preparedness, communications, and operations centers and other facilities required for emergency response. • Power-generating stations and other public utility facilities required as emergency backup facilities for Occupancy Category IV structures. • Structures containing highly toxic materials as defined by Section 307 where the quantity of the material exceeds the maximum allowable quantities of Table 307.1(2)[b]. • Aviation control towers, air traffic control centers and emergency aircraft hangars. • Buildings and other structures having critical national defense functions. • Water storage facilities and pump structures required to maintain water pressure for fire suppression.

a. For purposes of occupant load calculation, occupancies required by Table 1004.1.1 to use gross floor area calculations shall be permitted to use net areas to determine the total occupant load.
b. The building official shall require a hazardous material report that provides an opinion regarding the acceptability of risk from toxic and highly toxic materials to the public surrounding the building or structure in the event of a failure of the building or structure. The report shall be written by a qualified professional.

Reason: General

The concern addressed in this proposal is the need to update and clarify the intent of the Occupancy Categories in Table 1604.5. The provisions have significant impact on existing building alteration projects undergoing changes in occupancy and uses. Some have expressed the thought that solutions to problems should therefore be addressed in Chapter 34 and/or the IEBC. However the underlying principals of Table 1604.5 for new buildings need to be better understood by design professionals as well as building officials. In certain areas the current provisions may be inappropriately tied to the legacy codes.

One of the original underlying ATC 3 and NEHRP concepts in the evolution of Occupancy Categories is the notion that the nonstructural regulations found elsewhere in the code involving occupancy classification primarily relate to fire hazard and therefore are not appropriate for considerations of seismic hazard in structural engineering. There is not much in the record of these associations that provide a detailed discussion of the differences between occupancy related fire risk and seismic risk. In some cases life safety concepts found in Chapter 10 such as egress redundancy and exit separation would seem to address both a fire risk as well as a seismic risk. On the other hand there is a distinct difference between the suddenness of a seismic event and a fire emergency which has the benefit of early warning from fire alarms required in most moderate to large buildings

Additionally, since the early 1990's the legacy code expanded the concern beyond seismic events to include hazards due to wind, flood, snow, and ice loads. This has made the occupancy related distinctions between the fire risks and these structural engineering based risks less clear. While the factors themselves were originally in the building code, they are now found in the ASCE 7 document.

One reason why these provisions were not clarified under the legacy code is because there was no driving need to do so. This was due to the fact that most of the engineering factors associated with Occupancy Categories were 1.0. When significant distinctions between the factors were incorporated into the IBC, the need to clarify these provisions has become obvious.

One important aspect of this proposal is to address "occupancy and uses" in the descriptions rather than "buildings and structures" because that is indeed what the purpose of the table is all about.

Another important aspect of the proposal is to better clarify in the heading of each Category so that the code user will understand whether or not the concern about the occupancy/use is disaster response, a high occupant load, inability of self evacuate, etc, etc. If for nothing else, this will help the critique of this proposal.

A third aspect is to better clarify the distinctions between the Categories. The descriptor "low hazard" currently assigned to Category I sounds good but seems to imply something higher than a "low hazard" description for Category II. This seems odd given the fact that most buildings fall into Category II. As a result the term "minimum hazard" is proposed to apply to Category I and the term "low hazard" has been assigned to Category II. The term "moderate hazard" is proposed to replace "substantial hazard" to describe most Category III occupancies, although the term "substantial" was retained to describe the important public need of power generating stations, potable and waste water treatment (i.e., Category III's 7th bullet). Compare these with the last bullet in Category IV. In the author's opinion, the occupancies and uses in Category III's 7th bullet are "essential" for the health and welfare of society as a whole and could easily fall into Category IV. However in this proposal, it was left in Category III.

Occupant Loads – Category III

The current occupant load thresholds in Category III are varied and the reasoning behind them are unclear. Hopefully critiques of this proposal will bring forth any rational that exists.

The 300 occupant load threshold in the 1st bullet undoubtedly came from the legacy code's distinction between sub groups in the assembly occupancies. This no longer has a relational meaning in the non structural provisions of the IBC.

The phrase "more than 300 people…in one area" in the 2003 IBC was replaced in the 2006 IBC by "public assembly with an occupant load greater than 300".

The term "public assembly" is not defined in the code. The term "assembly" isn't specifically defined in the code but Section 303.1 states that it includes the "gathering of persons for purposes such as civic, social or religious functions; recreation, food, or drink consumption; or awaiting transportation." The term "public", also not specifically defined in the code, generally means "for all". There is no need to add this confusion (i.e., private assembly vs public assembly) to the determination of appropriate structural factors unless definitions of these terms can be agreed upon and set forth in the code.

The 300 occupant load threshold in the 06 code is assumed to be associated with the portion of the building associated only with those involved directly with the public assembly use. And this could be in one room in the building or in multiple rooms throughout the building.

The term "primary occupancy' is easy to determine in many buildings, but the determination is unclear in common mixed use buildings. Should it be based on the occupancy with the greatest assigned floor area? Or the portion with the greatest occupant load? If a building has two significant uses, are there no primary uses or are there two primary uses? Why should a single story primary use restaurant with an occupant load of 301 have higher structural engineering factors than a ballroom in a mid rise mixed use building with an occupant load of 2000?

The original intent of the phrase "in one area" (however ill-defined this may have otherwise been in the 03 code) may have been generated by a concern of local failure rather than global building failure, but that is not clearly discussed in the underlying ATC and NEHRP record. The 06 change appears to have redirected the concern to situations where the "primary" occupancy is "public assembly" in a building with an occupant load greater than 300. As an example, this could mean a movie theater with more than 300 occupants in one room, or it could be a multiplex facility with multiple theaters with 75 occupants in each, or it could be a corporate training facility with multiple training rooms that add up to over 300 occupant load. Because it now involves a building wide occupant load, it seems like the issue is more about global failure rather than a local failure.

Because of the above, and in light of the 5000 occupant load threshold set in the 6th bullet, it is proposed to strike the 1st bullet all together. Other current triggering occupant load thresholds in Category III include 250 for Group E school and day care facilities (2nd bullet), 500 for adult education facilities (3rd bullet), 50 for I-2 occupancies (4th bullet), as well as the 5000 for other occupancies (6th bullet). In each of these instances the occupant load threshold applies to the occupant load of the whole building. The 250, the 500, and the 50 occupant load thresholds were likely chosen because the requirements of the higher category may have seemed excessive for smaller facilities. It is the author's opinion the general concern in the code for life safety should be more generic and not make distinctions between occupant load thresholds so that a consistent number is assigned to all occupancies. If the structural provisions in the code are in need of more rigorous criteria for a college (for example) of a certain occupant load, it seems that such provisions should also be applied to other occupancies with similar occupant loads. For reasons similar to the action taken in the 1st bullet, the provisions of for the E occupancies (2nd bullet) and the adult education (3rd bullet) are proposed to be deleted.

Inability to Self Evacuate and Educational – Category III and IV

The current code, the legacy codes, and the original ATC 3 and NEHRP documents address distinctions relating to age, mobility, and ability to self evacuate. Educational occupancies from the 12th grade down are E occupancies, including daycare for children 2.5 years old or older. If less than 2.5 years old child care falls into the I-2 or I-4 occupancy depending on whether or not the care is for more than a 24 hour basis. Group I occupancy involve people with health or age limitations or incarceration restraints but which also includes I -1 (assisted living, etc.).

However, by code definition, I -1 occupancies have self preservation capabilities. Contrary to this, earlier editions of the west coast legacy code required that daycare and K- 3rd grade students had to be located on levels near grade. In addition stairs and corridors were required to meet higher width minimums presumably to facilitate egress. For the most part the IBC treats the means of egress provisions of E occupancies quite similar to other common occupancies. This proposal seeks to strike any requirement for E occupancies to be more consistent with the non-structural provisions in the current I code. If this portion of the proposal is rejected, it is hoped that the critique will articulate the reason why school aged children need to be in stronger buildings (for earthquake, wind, flood, snow, and ice engineering) than the buildings housing the parents of the children. See separate discussion on occupant load.

The proposal also strikes the bullet (Category III, 3rd bullet) relating to educational facilities for higher than the 12th grade. While the current code sets forth "colleges and universities" as examples, it is written open ended such that corporate training facilities or an assembly hall used periodically for educational venue have been included by some building officials. Because the reason why colleges, universities, and other educational facilities are in Category III are not at all clear, it is proposed to be removed for reasons similar to the previous discussion of Group E occupancies. See separate discussion on occupant load.

All other occupancies (jails, nursing homes, mental institutions, etc.) where the ability to self evacuate is limited are proposed to remain in Category III.

Hospitals – Category III and IV

Category III hospitals are proposed to be moved to Category IV even if they do not have emergency rooms and operating rooms. The reason for this is that in event of a disaster, it is believed such facilities may be in critical need for hospital level care for those patients transferred from facilities with emergency and operating facilities. The occupant load threshold of 50 resident patients has been maintained.

Toxic Materials – Category III and IV

A new footnote b has been added such that the actual level of risk to those in the vicinity of the facility containing toxic (Category III) or highly toxic (Category IV) will be addressed in a report written by a knowledgeable professional.

The reference to explosion events in the last bullet in Category III is proposed to be deleted because the topic at hand seems to be toxic materials that may be released due to building structural collapse as opposed to explosions.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S55–07/08

202, 1504.2.1, 1602.1, 1603.1.4, 1603.1.5, 1604.5, Table 1604.5, 1604.5.5, 1609.1.2, 1613.2, 1613.5.6, Table 1613.5.6(1), Table 1613.5.6(2),1704.5, 1704.5.1, 1704.5.2, 1704.5.3, 1708.1.1, 1708.1.2, 1708.1.3, 1708.1.4, 1709.2, 1709.3, 1805.2.1, 2308.2, 3406.4; IEBC 305.4, Table 506.1.1.2,506.1.1.3, 902.1, 907.2, 907.3.1, 907.3.2, A102.2, A503, Table A5-A, Table A5-C

Proponent: Constadino Sirakis, PE, SECB, New York City Department of Buildings-Technical Affairs

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STRUCTURAL AND IEBC CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

SECTION 202
DEFINITIONS

<u>STRUCTURAL</u> OCCUPANCY CATEGORY. See Section 1602.1.

SECTION 1602
DEFINITIONS AND NOTATIONS

<u>STRUCTURAL</u> OCCUPANCY CATEGORY. A category used to determine structural requirements based on occupancy.

1504.2.1 Alternative test method. Testing the acceptability of special fastening methods using the methodology in this section is permitted. The wind-induced uplift force on the shingle shall be determined using the method in UL 2390. The resistance of the shingle to the uplift force shall be determined using ASTM D 6381. Shingles passing this test shall be considered suitable for roofs located where the basic wind speed per Figure 1609 is as given in Table 1504.2.1.

Classification requires that the resistance of the shingle to wind uplift, measured using the method in ASTM D 6381, exceed the calculated load imposed by wind in the applicable zone as determined using UL 2390.

Classification by this method applies to buildings less than 60 feet (18 288 mm) high and with Wind Exposures B and only in an Structural Occupancy Category of I or II. Wrappers of shingle bundles that have been qualified using this alternative method shall be labeled with the tested wind classification and reference UL 2390/ASTM D 6381.

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

1. Basic wind speed (3-second gust), miles per hour (km/hr).
2. Wind importance factor, *I*, and structural occupancy category.
3. Wind exposure. Where more than one wind exposure is utilized, the wind exposure and applicable wind direction shall be indicated.
4. The applicable internal pressure coefficient.
5. Components and cladding. The design wind pressures in terms of psf (kN/m^2) to be used for the design of exterior component and cladding materials not specifically designed by the registered design professional.

1603.1.5 Earthquake design data. The following information related to seismic loads shall be shown, regardless whether seismic loads govern the design of the lateral-force-resisting system of the building:

1. Seismic importance factor, *I*, and structural occupancy category.
2. Mapped spectral response accelerations, S_S and S_1. Site class.
4. Spectral response coefficients, S_{DS} and S_{D1}.
5. Seismic design category.
6. Basic seismic-force-resisting system(s).
7. Design base shear.
8. Seismic response coefficient(s), C_S.
9. Response modification factor(s), *R*.
10. Analysis procedure used.

1604.5 Structural occupancy category. Buildings shall be assigned an occupancy category in accordance with Table 1604.5.

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

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

STRUCTURAL 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: • Agricultural facilities. • Certain temporary facilities. • Minor storage facilities.
II	Buildings and other structures except those listed in Structural Occupancy Categories I, III and IV
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300. • Buildings and other structures containing elementary school, secondary school or day care facilities with an occupant load greater than 250. • Buildings and other structures containing adult education facilities, such as colleges and universities with an occupant load greater than 500. • Group I-2 occupancies with an occupant load of 50 or more resident patients but not having surgery or emergency treatment facilities. • Group I-3 occupancies.

STRUCTURAL OCCUPANCY CATEGORY	NATURE OF OCCUPANCY
	• Any other occupancy with an occupant load greater than 5,000[a]. • Power-generating stations, water treatment facilities for potable water, waste water treatment facilities and other public utility facilities not included in Structural Occupancy Category IV. • Buildings and other structures not included in Structural Occupancy Category IV containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released.
IV	Buildings and other structures designated as essential facilities, including but not limited to: • Group I-2 occupancies having surgery or emergency treatment facilities. • Fire, rescue, ambulance and police stations and emergency vehicle garages. • Designated earthquake, hurricane or other emergency shelters. • Designated emergency preparedness, communications, and operations centers and other facilities required for emergency response. • Power-generating stations and other public utility facilities required as emergency backup facilities for Structural Occupancy Category IV structures. • Structures containing highly toxic materials as defined by Section 307 where the quantity of the material exceeds the maximum allowable quantities of Table 307.1(2). • Aviation control towers, air traffic control centers and emergency aircraft hangars. • Buildings and other structures having critical national defense functions. • Water storage facilities and pump structures required to maintain water pressure for fire suppression.

a. For purposes of occupant load calculation, occupancies required by Table 1004.1.1 to use gross floor area calculations shall be permitted to use net areas to determine the total occupant load.

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

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

Exceptions:

1. Wood structural panels with a minimum thickness of 7/16 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 classified as Group R-3 or R-4 occupancy. Panels shall be precut so that they shall be attached to the framing surrounding the opening containing the product with the glazed opening. Panels shall be predrilled as required for the anchorage method and shall be secured with the attachment hardware provided. Attachments shall be designed to resist the components and cladding loads determined in accordance with the provisions of ASCE 7, with corrosion resistant attachment hardware provided and anchors permanently installed on the building. Attachment in accordance with Table 1609.1.2 with corrosion resistant attachment hardware provided and anchors permanently installed on the building is permitted for buildings with a mean roof height of 45 feet (13716 mm) or less where wind speeds do not exceed 140 mph (63 m/s).
2. Glazing in Structural 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 Structural 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.

1613.2 Definitions.

SEISMIC DESIGN CATEGORY. A classification assigned to a structure based on its structural occupancy category and the severity of the design earthquake ground motion at the site.

1613.5.6 (Supp) Determination of seismic design category. Structures classified as Structural Occupancy Category I, II or III that are located where the mapped spectral response acceleration parameter at 1-second period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category E. Structures classified as Structural Occupancy Category IV that are located where the mapped spectral response acceleration parameter at 1-second period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category F. All other structures shall be assigned to a seismic design category based on their occupancy category and the design spectral response acceleration coefficients, S_{DS} and S_{D1}, determined in accordance with Section 1613.5.4 or the site specific procedures of ASCE 7. Each building and structure shall be assigned to the more severe seismic design category in accordance with Table 1613.5.6(1) or 1613.5.6(2), irrespective of the fundamental period of vibration of the structure, *T*.

1704.5 Masonry construction. Masonry construction shall be inspected and evaluated 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 Structural 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 1613.5.6(1)
SEISMIC DESIGN CATEGORY BASED ON
SHORT-PERIOD RESPONSE ACCELERATIONS

VALUE OF S_{DS}	STRUCTURAL OCCUPANCY CATEGORY		
	I or II	III	IV

(Portions of table not shown remain unchanged)

TABLE 1613.5.6(2)
SEISMIC DESIGN CATEGORY BASED ON
1-SECOND PERIOD RESPONSE ACCELERATION

VALUE OF S_{D1}	STRUCTURAL OCCUPANCY CATEGORY		
	I or II	III	IV

(Portions of table not shown remain unchanged)

1704.5.1 Empirically designed masonry, glass unit masonry and masonry veneer in Structural Occupancy Category IV. The minimum special inspection program for 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, in structures classified as Structural Occupancy Category IV, in accordance with Section 1604.5, shall comply with Table 1704.5.1.

1704.5.2 Engineered masonry in Structural Occupancy Category I, II or III. The minimum special inspection program for masonry designed by Section 2107 or 2108 or by chapters other than Chapters 5, 6 or 7 of ACI 530/ASCE 5/TMS 402 in structures classified as Structural Occupancy Category I, II or III, in accordance with Section 1604.5, shall comply with Table 1704.5.1.

1704.5.3 Engineered masonry in Structural Occupancy Category IV. The minimum special inspection program for masonry designed by Section 2107 or 2108 or by chapters other than Chapters 5, 6 or 7 of ACI 530/ASCE 5/TMS 402 in structures classified as Structural Occupancy Category IV, in accordance with Section 1604.5, shall comply with Table 1704.5.3.

1708.1.1 Empirically designed masonry and glass unit masonry in Structural 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 Structural 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 Structural 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 Structural Occupancy Category IV, in accordance with Section 1604.5, shall comply with the requirements of Table 1708.1.2.

1708.1.3 Engineered masonry in Structural 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 Structural 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 Structural 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 Structural Occupancy Category IV, in accordance with Section 1604.5, shall comply with Table 1708.1.4.

1709.2 (Supp) Structural observations for seismic resistance. Structural observations shall be provided for those structures included in Seismic Design Category D, E or F, as determined in Section 1613, where one or more of the following conditions exist:

1. The structure is classified as Structural Occupancy Category III or IV in accordance with Table 1604.5.
2. The height of the structure is greater than 75 feet (22 860 mm) above the base.
3. The structure is assigned to Seismic Design Category E, is classified as Structural Occupancy Category I or II in accordance with Table 1604.5, and is greater than two stories above grade plane.
4. When so designated by the registered design professional responsible for the structural design.
5. When such observation is specifically required by the building official.

1709.3 (Supp) Structural observations for wind requirements. Structural observations shall be provided for those structures sited where the basic wind speed exceeds 110 mph (49 m/sec) determined from Figure 1609, where one more of the following conditions exist:

1. The structure is classified as Structural Occupancy Category III or IV in accordance with Table 1604.5.
2. The building height of the structure is greater than 75 feet (22 860 mm).
3. When so designated by the registered design professional responsible for the structural design.
4. When such observation is specifically required by the building official.

1805.2.1 Frost protection. Except where otherwise protected from frost, foundation walls, piers and other permanent supports of buildings and structures shall be protected by one or more of the following methods:

1. Extending below the frost line of the locality;
2. Constructing in accordance with ASCE 32; or
3. Erecting on solid rock.

   Exception: Free-standing buildings meeting all of the following conditions shall not be required to be protected:

   1. Classified in Structural Occupancy Category I, in accordance with Section1604.5;
   2. Area of 600 square feet (56 m^2) or less for light-frame construction or 400 square feet (37m^2) or less for other than light-frame construction; and 3. Eave height of 10 feet (3048 mm) or less. Footings shall not bear on frozen soil unless such frozen condition is of a permanent character.

2308.2 (Supp) Limitations. Buildings are permitted to be constructed in accordance with the provisions of conventional light-frame construction, subject to the following limitations, and to further limitations of Sections 2308.11 and 2308.12.

1. Buildings shall be limited to a maximum of three stories above grade plane. For the purposes of this section, for buildings in Seismic Design Category D or E as determined in Section 1613, cripple stud walls shall be considered to be a story.

Exception: Solid blocked cripple walls not exceeding 14 inches (356 mm) in height need not be considered a story.

2. Maximum floor-to-floor height shall not exceed 11 feet 7 inches (3531 mm). Bearing wall height shall not exceed a stud height of 10 feet (3048 mm).
3. Loads as determined in Chapter 16 shall not exceed the following:
   3.1. Average dead loads shall not exceed 15 psf (718 N/m2) for combined roof and ceiling, exterior walls, floors and partitions.

   Exceptions:

   1. Subject to the limitations of Sections 2308.11.2 and 2308.12.2, stone or masonry veneer up to the lesser of 5 inches (127 mm) thick or 50 psf (2395 N/m2) and installed in accordance with Chapter 14 is permitted to a height of 30 feet (9144 mm) above a noncombustible foundation, with an additional 8 feet (2438 mm) permitted for gable ends.
   2. Concrete or masonry fireplaces, heaters and chimneys shall be permitted in accordance with the provisions of this code.

   3.2. Live loads shall not exceed 40 psf (1916 N/m2) for floors.
   3.3. Ground snow loads shall not exceed 50 psf (2395 N/m2).
4. Wind speeds shall not exceed 100 miles per hour (mph) (44 m/s) (3-second gust).

   Exception: Wind speeds shall not exceed 110 mph (48.4 m/s) (3-second gust) for buildings in Exposure Category B that are not located in a hurricane prone region.

5. Roof trusses and rafters shall not span more than 40 feet (12 192 mm) between points of vertical support.
6. The use of the provisions for conventional light-frame construction in this section shall not be permitted for Structural Occupancy Category IV buildings assigned to Seismic Design Category B, C, D, E or F, as determined in Section 1613.
7. Conventional light-frame construction is limited in irregular structures in Seismic Design Category D or E, as specified in Section 2308.12.6.

3406.4 Change of occupancy. When a change of occupancy results in a structure being reclassified to a higher structural occupancy category, the structure shall conform to the seismic requirements for a new structure.

Exceptions:

1. Specific seismic detailing requirements of this code or ASCE 7 for a new structure shall not be required to be met where it can be shown that the level of performance and seismic safety is equivalent to that of a new structure. Such analysis shall consider the regularity, over strength, redundancy and ductility of the structure within the context of the existing and retrofit (if any) detailing provided.
2. When a change of use results in a structure being reclassified from Structural Occupancy Category I or II to Structural Occupancy Category III and the structure is located in a seismic map area where $S_{DS} < 0.33$, compliance with the seismic requirements of this code and ASCE 7 are not required.

PART II – IEBC

1. Revise as follows:

305.4 Structural. When a change of occupancy results in a structure being reclassified to a higher structural occupancy category, the structure shall conform to the seismic requirements for a new structure.

Exceptions:

1. Specific seismic detailing requirements of this code or ASCE 7 for a new structure shall not be required to be met where it can be shown that the level of performance and seismic safety is equivalent to that of a new structure. Such analysis shall consider the regularity, over strength, redundancy and ductility of the structure within the context of the existing and retrofit (if any) detailing provided.
2. When a change of use results in a structure being reclassified from Structural Occupancy Category I or II to Occupancy Category III and the structure is located in a seismic map area where $S_{DS} < 0.33$, compliance with the seismic requirements of this code and ASCE 7 are not required.

TABLE 506.1.1.2 (Supp)
ASCE 41 AND ASCE 31 PERFORMANCE LEVELS

STRUCTURAL OCCUPANCY CATEGORY (BASED ON IBC TABLE 1604.5)	PERFORMANCE LEVEL FOR USE WITH ASCE 31 AND WITH ASCE 41 BSE-1 EARTHQUAKE HAZARD LEVEL	PERFORMANCE LEVEL FOR USE WITH ASCE 41 BSE-2 EARTHQUAKE HAZARD LEVEL
I	Life Safety (LS)	Collapse Prevention (CP)
II	Life Safety (LS)	Collapse Prevention (CP)
III	Note a	Note a
IV	Immediate Occupancy (IO)	Life Safety (LS)

a. Performance levels for Structural Occupancy Category III shall be taken as halfway between the performance levels specified for Structural Occupancy Category II and IV.

506.1.1.3 (Supp) Reduced IBC level seismic forces. When seismic forces are permitted to meet reduced *International Building Code* levels, they shall be one of the following:

1. Seventy-five percent of the forces prescribed in the *International Building Code*. The *R*-factor used for analysis in accordance with Chapter 16 of the *International Building Code* shall be the *R*-factor as specified in Section 506.1.1.2 of this code.
2. In accordance with the applicable chapters in Appendix A of this code as specified in Items 2.1 through 2.5 below. Structures or portions of structures that comply with the requirements of the applicable chapter in Appendix A shall be deemed to comply with the requirements for reduced *International Building Code* force levels.
   2.1. The seismic evaluation and design of unreinforced masonry bearing wall buildings in Structural Occupancy Category I or II are permitted to be based on the procedures specified in Appendix Chapter A1.
   2.2. Seismic evaluation and design of the wall anchorage system in reinforced concrete and reinforced masonry wall buildings with flexible diaphragms in Structural Occupancy Category I or II are permitted to be based on the procedures specified in Appendix Chapter A2.
   2.3. Seismic evaluation and design of cripple walls and sill plate anchorage in residential buildings of light frame wood construction in Structural Occupancy Category I or II are permitted to be based on the procedures specified in Appendix Chapter A3.
   2.4. Seismic evaluation and design of soft, weak or open-front wall conditions in multiunit residential buildings of wood construction in Structural Occupancy Category I or II are permitted to be based on the procedures specified in Appendix Chapter A4.
   2.5. Seismic evaluation and design of concrete buildings and concrete with masonry infill buildings in all structural occupancy categories are permitted to be based on the procedures specified in Appendix Chapter A5.
3. In accordance with ASCE 31 based on the applicable performance level as shown in Table 506.1.1.2.
4. Those associated with the BSE-1 Earthquake Hazard Level defined in ASCE 41 and the performance level as shown in Table 506.1.1.2. Where ASCE 41 is used, the design spectral response acceleration parameters S_{XS} and S_{X1} shall not be taken less than 75 percent of the respective design spectral response acceleration parameters S_{DS} and S_{D1} defined by the *International Building Code* and its reference standards.

902.1 Compliance with the building code. Where the character or use of an existing building or part of an existing building is changed to one of the following special use or occupanc~~y~~ies ~~categories~~ as defined in the *International Building Code*, the building shall comply with all of the applicable requirements of the *International Building Code*:

1. Covered mall buildings.
2. Atriums.
3. Motor vehicle-related occupancies.
4. Aircraft-related occupancies.
5. Motion picture projection rooms.
6. Stages and platforms.
7. Special amusement buildings.
8. Incidental use areas.
9. Hazardous materials.

907.2 Snow and wind loads. Buildings and structures subject to a change of occupancy where such change in the nature of occupancy results in higher wind or snow structural occupancy categories based on Table 1604.5 of the *International Building Code* shall be analyzed and shall comply with the applicable wind or snow load provisions of the *International Building Code*.

Exception: Where the new occupancy with a higher importance factor is less than or equal to 10 percent of the total building floor area. The cumulative effect of the area of occupancy changes shall be considered for the purposes of this exception.

907.3.1 Compliance with the *International Building Code*. When a building or portion thereof is subject to a change of occupancy such that a change in the nature of the occupancy results in a higher seismic ~~occupancy~~ importance factor based on Table 1604.5 of the *International Building Code*; or where such change of occupancy results in a reclassification of a building to a higher hazard category as shown in Table 912.4; or where a change of a Group M occupancy to a Group A, E, I-1 R-1, R-2 or R-4 occupancy with two-thirds or more of the floors involved in Level 3 alteration work, the building shall conform to the seismic requirements of the *International Building Code* for the new seismic use group.

Exceptions:

1. Group M occupancies being changed to Group A, E, I-1, R-1, R-2 or R-4 occupancies for buildings less than six stories in height and in Seismic Design Category A, B or C.
2. Specific detailing provisions required for a new structure are not required to be met where it can be shown that an acceptable level of performance and seismic safety is obtained for the applicable seismic use group using reduced *International Building Code* level seismic forces as specified in Section 506.1.1.3. The rehabilitation procedures shall be approved by the code official and shall consider the regularity, over strength, redundancy and ductility of the lateral-load-resisting system within the context of the existing detailing of the system.
3. Where the area of the new occupancy with a higher hazard category is less than or equal to 10 percent of the total building floor area and the new occupancy is not classified as ~~Seismic Use Group~~ Structural Occupancy Category IV. For the purposes of this exception, where a structure is occupied for two or more occupancies not included in the same ~~seismic use group~~ structural occupancy category, the structure shall be assigned the classification of the highest ~~seismic use group~~ structural occupancy category corresponding to the various occupancies. Where structures have two or more portions that are structurally separated, each portion shall be separately classified. Where a structurally separated portion of a structure provides required access to, required egress from or shares life safety components with another portion having a higher ~~seismic use group~~ structural occupancy category, both portions shall be assigned the higher seismic use group. The cumulative effect of the area of occupancy changes shall be considered for the purposes of this exception.
4. Unreinforced masonry bearing wall buildings in Structural Occupancy Category III when assigned to Seismic Design Category A or B shall be allowed to be strengthened to meet the requirements of Appendix Chapter A1 of this code [*Guidelines for the Seismic Retrofit of Existing Buildings* (GSREB)].

907.3.2 Access to ~~Seismic Use Group~~ Structural Occupancy Category IV. Where the change of occupancy is such that compliance with Section 907.3.1 is required and the ~~seismic use group~~ structural occupancy category is a Category IV, the operational access to such ~~Seismic Use Group~~ Structural Occupancy Category IV existing structure shall not be through an adjacent structure.

Exception: Where the adjacent structure conforms to the requirements for ~~Seismic Use Group~~ Structural Occupancy Category IV structures.

Where operational access is less than 10 feet (3048 mm) from an interior lot line or less than 10 feet (3048 mm) from another structure, access protection from potential falling debris shall be provided by the owner of the ~~Seismic Use Group~~ Structural Occupancy Category IV structure.

A102.2 Essential and hazardous facilities. The provisions of this chapter shall not apply to the strengthening of buildings or structures in Structural Occupancy Category III when assigned to Seismic Design Category C, D, or E or buildings or structures in Structural Occupancy Category IV. Such buildings or structures shall be strengthened to meet the requirements of the *International Building Code* for new buildings of the same structural occupancy category or other such criteria that have been established by the jurisdiction.

2. Delete without substitution:

SECTION A503
DEFINITIONS

~~**SEISMIC USE GROUP III.** Those buildings categorized as essential facilities or hazardous facilities, or as designated by the building official.~~

3. Revise as follows:

TABLE A5-A—BASIC STRUCTURAL CHECKLIST

	Conditions of Materials
	CONCRETE WALL CRACKS: All existing diagonal cracks in wall elements shall be less than 1/8 inch for ~~Seismic Use Groups~~ Structural Occupancy Categories other than ~~Group III~~ Category IV and less than 1/16 inch for ~~Seismic Use Group III~~ Structural Occupancy Category IV; shall not be concentrated in one location; and shall not form a X pattern.
	REINFORCED MASONRY WALL CRACKS: All existing diagonal cracks in wall elements shall be less than 1/8 inch for ~~Seismic Use Groups~~ Structural Occupancy Categories other than ~~Group III~~ Category IV and less than 1/16 inch for ~~Seismic Use Group III~~ Structural Occupancy Category IV; shall not be concentrated in one location; and shall not form a X pattern.
	CRACKS IN BOUNDARY COLUMNS: There shall be no existing diagonal cracks wider 1/8 inch for ~~Seismic Use Groups~~ Structural Occupancy Categories other than ~~Group III~~ Category IV and wider than 1/16 inch for ~~Seismic Use Group III~~ Structural Occupancy Category IV in concrete columns that encase masonry infills.
	LATERAL-FORCE-RESISTING SYSTEM **Moment Frames** **General**
	REDUNDANCY: The number of lines of moment frames in each principal direction shall be greater than or equal to two for ~~Seismic Use Groups I, II and III~~ all structural occupancy categories. The number of bays of moment frames in each line shall be greater than or equal to two for ~~Seismic Use Groups~~ Structural Occupancy Categories other than ~~Group III~~ Category IV, and three for ~~Seismic Use Group III~~ Structural Occupancy Category IV.
	CONNECTIONS **Shear Transfer**
	TRANSFER TO SHEAR WALLS: The diaphragm shall be reinforced and connected for transfer of loads to the shear walls for ~~Seismic Use Groups~~ Structural Occupancy Categories I, ~~and~~ II, and III. ~~the~~The connections shall be able to develop the shear strength of the walls for ~~Seismic Use Group III~~ Structural Occupancy Category IV.
	Vertical Components CONCRETE COLUMNS: All concrete columns shall be doweled into the foundation for ~~Seismic Use Groups~~ I, ~~and~~ II, and III. ~~the~~ The dowels shall be able to develop the tensile capacity of the column for ~~Seismic Use Group III~~ Structural Occupancy Category IV.
	WALL REINFORCING: Walls shall be doweled into the foundation for ~~Seismic Use Groups~~ Structural Occupancy Category I~~, and~~ II, and III. ~~the~~ The dowels shall be able to develop the strength of the walls for ~~Seismic Use Group III~~ Structural Occupancy Category IV.

(Portions of table not shown remain unchanged)

TABLE A5-B—SUPPLEMENTAL STRUCTURAL CHECKLIST

LATERAL-FORCE-RESISTING SYSTEM **Moment Frames** **Concrete Moment Frames**	
	SHORT CAPTIVE COLUMNS: There shall be no columns at a level with height-depth ratios less than 50 percent of the nominal height-depth ratio of the typical columns at that level for ~~Seismic Use Groups~~ structural occupancy categories other than ~~Group III~~ Category IV, and less than 75 percent for ~~Seismic Use Group III~~ Structural Occupancy Category IV.
	COLUMN-BAR SPLICES: All column bar lap-splice lengths shall be greater than 35 db for ~~Seismic Use Groups~~ structural occupancy categories other than ~~Group III~~ Category IV and greater than 50 db for ~~Seismic Use Group III~~ Structural Occupancy Category IV, and shall be enclosed by ties spaced at or less than 8 db for all ~~Seismic Use Groups~~ structural occupancy categories.
	JOINT ECCENTRICITY: For ~~Seismic Use Group III~~ Structural Occupancy Category IV, there shall be no eccentricities larger than 20 percent of the smallest column plan dimension between girder and column centerlines.
	STIRRUP AND TIE HOOKS: For ~~Seismic Use Group III~~ Structural Occupancy Category IV, the beam stirrups and column ties shall be anchored into the member cores with hooks of 135° or more.
Frames Not Part of the Lateral-Force-Resisting System	
	DEFORMATION COMPATIBILITY: Nonlateral-force-resisting components shall have the shear capacity to develop the flexural strength of the elements for ~~Seismic Use Groups~~ structural occupancy categories other than ~~Group III~~ Category IV and shall have ductile detailing for ~~Seismic Use Group III~~ Structural Occupancy Category IV.
	FLAT SLABS: Flat slabs/plates classified as nonlateral-force-resisting components shall have continuous bottom steel through the column joints for Seismic Use Groups structural occupancy categories other than ~~Group III~~ Category IV. Flat slabs/plates shall not be permitted for ~~Seismic Use Group III~~ Structural Occupancy Category IV.
Shear Walls **Concrete Shear Walls**	
	COUPLING BEAMS: The stirrups in all coupling beams over means of egress shall be spaced at or less than d/2 and shall be anchored into the core with hooks of 135° or more. In addition, the beams shall have the capacity in shear to develop the uplift capacity of the adjacent wall for ~~Seismic Use Group III~~ Structural Occupancy Category IV.
	OVERTURNING: For ~~Seismic Use Group III~~ Structural Occupancy Category IV, all shear walls shall have aspect ratios less than 4:1. Wall piers need not be considered.
	CONFINEMENT REINFORCING: For shear walls in ~~Seismic Use Group III~~ Structural Occupancy Category IV with aspect ratios greater than 2.0, boundary elements shall be confined with spirals or ties with spacing less than 8 db.
	REINFORCING AT OPENINGS: For ~~Seismic Use Group III~~ Structural Occupancy Category IV, there shall be added trim reinforcement around all wall openings.
	WALL THICKNESS: For ~~Seismic Use Group III~~ Structural Occupancy Category IV, thickness of bearing walls shall not be less than 1/25 the minimum unsupported height or length, or less than 4 inches.
DIAPHRAGMS **General**	
	DIAPHRAGM OPENINGS ADJACENT TO SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls shall be less than 25 percent of the wall length for ~~Seismic Use Groups I and II~~ Structural Occupancy Categories I, II and III, and less than 15 percent of the wall length for ~~Seismic Use Group III~~ Structural Occupancy Category IV.
	PLAN IRREGULARITIES: There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other locations of plan irregularities. This statement shall apply to ~~Seismic Use Group III~~ Structural Occupancy Category IV only.
	DIAPHRAGM REINFORCEMENT AT OPENINGS: There shall be reinforcement around all diaphragm openings larger than 50 percent of the building width in either major plan dimension. This statement shall apply to ~~Seismic Use Group III~~ Structural Occupancy Category IV only.
CONNECTIONS **Vertical Components**	
	LATERAL LOAD AT PILE CAPS: Pile caps shall have top reinforcement, and piles shall be anchored to the pile caps for ~~Seismic Use Groups I and II~~ Structural Occupancy Categories I, II, and III. The pile cap reinforcement and pile anchorage shall be able to develop the tensile capacity of the piles for ~~Seismic Use Group III~~ Structural Occupancy Category IV.

(Portions of table not shown remain unchanged)

TABLE A5-C—GEOLOGIC SITE HAZARD AND FOUNDATION CHECKLIST

Condition of Foundations	
	The following statement shall be completed for buildings in regions of high or moderate seismicity being evaluated to ~~Seismic Use Group III~~ Structural Occupancy Category IV:
Capacity of Foundations	
	POLE FOUNDATIONS: Pole foundations shall have a minimum embedment depth of 4 feet for ~~Seismic Use Groups~~ Structural Occupancy Categories I, II, ~~and~~ III, and IV. The following statements shall be completed for buildings in regions of high seismicity and for buildings in regions of moderate seismicity evaluated to ~~Seismic Use Group III~~ Structural Occupancy Category IV:
	DEEP FOUNDATIONS: Piles and piers shall be capable of transferring the lateral forces between the structure and the soil. This statement shall apply to ~~Seismic Use Group III~~ Structural Occupancy Category IV only.
	SLOPING SITES: The grade difference from one side of the building to another shall not exceed one-half the story height at the location of embedment. This statement shall apply to ~~Seismic Use Group III~~ Structural Occupancy Category IV Performance Level only.

(Portions of table not shown remain unchanged)

Reason: The purpose of this code change proposal is to clarify the terminology used with regards to occupancy classifications as defined in chapter 3 of the International Building Code (IBC), and the structural classification also based on occupancy defined in chapter 16. The term "Occupancy Category" used for the structural classification in IBC section 1604.5 is often confused with the " Occupancy Classification" of IBC chapter 3 by design professionals. As these two related but distinctly different categories are both required pieces of information for code officials and design professionals to properly implement and enforce the code provisions, there is a need to better distinguish between the two categories.

"Occupancy Classification" is a term that has been used in IBC and other codes for some time, and design professionals are quite familiar with its use. "Occupancy Category" was a term that was introduced in the 2004/2005 IBC Code Change Cycle to replace the term "Importance Factor Category". This change was based on the 2005 edition of the American Society of Civil Engineers (ASCE) ASCE 7 reference standard. While "Importance Factor Category" did not properly define the category, as it is used for much more than determining importance factors for structural design, the title was distinct and not confused with others.

The author is proposing to change the term "Occupancy Category" defined in chapter 16 to "Structural Occupancy Category" to make a clear distinction from the "Occupancy Classification" of a building or other structure. This will distinguish the "Structural Occupancy Category" as the category describing the level of structural performance required, while maintaining the connection to the current edition (2005) of the ASCE 7 reference standard. This change will add clarity to the IBC until terminology that works for both the International Code Council and ASCE can be developed and implemented in both standards.

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

PART I – IBC STRUCTURAL

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

PART II – IEBC

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S56–07/08

1604.6 (New)

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

Add new text as follows:

1604.6 Component importance factor. Components shall be assigned a component importance factor, I_p, in accordance with Section 13.1.3 of ASCE 7. The component importance factor shall be taken as 1.5 where any of the following conditions apply:

1. The component is required to function for life-safety purposes after an earthquake, including automatic sprinkler systems;
2. The component conveys hazardous materials; or
3. The component is in or attached to an Occupancy Category IV building or structure and it is needed for, or its failure could impair, the continued operation of the facility.

(Renumber subsequent sections)

Reason: The proposed language is adapted from Section 13.1.3 of ASCE 7-05. IBC Section 1604.5 assigns occupancy categories to buildings based on their function as specified in Table 1604.5. ASCE 7-05, in turn, specifies importance factors for the buildings based on their occupancy category and the type of design load (i.e., wind, snow or earthquake). The general effect is to increase design loads for which the building is required to resist.

ASCE 7-05 also specifies importance factors for components. For most components, the component importance factor is 1.0, which will not lead to an increase in design load. There are cases, however, where the component importance factor is greater than one. What is missing from the IBC is charging language for the component importance factor similar to what is now effectively accomplished by Section 1604.5 for building importance factors.

The purpose of this proposal is to provide charging language that will link the IBC with the corresponding technical provisions of ASCE 7-5 for component importance factor. This is especially important for the design of components to resist earthquake loads because not all designers are registered design professionals and may be less familiar with the provisions of ASCE 7 than are registered design professionals.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S57–07/08
1604.8.2

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

Revise as follows:

1604.8.2 ~~Concrete and masonry~~ <u>W</u>alls. ~~Concrete and~~ masonry <u>W</u>alls 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~~ <u>the</u> minimum strength design horizontal force ~~of 280 plf (4.10 kN/m) of wall~~ <u>specified in Section 11.7.3 of ASCE 7</u>, substituted for "*E*" in the load combinations of Section 1605.2 or 1605.3. <u>Concrete and masonry w</u>alls 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.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S58–07/08
1604.8.3

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.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S59–07/08
1604.11, 1605 (New)

Proponent: Gary Lewis, Chair, ICC Ad Hoc Committee on Terrorism Resistant Buildings

Add new text follows:

1604.11 Disproportionate collapse. Design for structural integrity to protect against disproportionate collapse shall be in accordance with Section 1605.

SECTION 1605
DISPROPORTIONATE COLLAPSE

1605.1 General. The building, structure or portion thereof shall be constructed so the building will not suffer collapse as the result of an accident or incident to an extent disproportionate to the cause. Buildings shall be designed for sufficient robustness to sustain a limited extent of damage or failure, depending on the class of the building, without collapse.

1605.2 DEFINITIONS.

ALTERNATE LOAD PATH METHOD. A design approach that assumes that a local failure occurs, but demonstrates an alternate load path so that damage is absorbed and spread of collapse is arrested.

DISPROPORTIONATE COLLAPSE. The spread of damage from an initiating event from element to element resulting in the collapse of an entire structure or a disproportionately large portion of it.

LOAD-BEARING CONSTRUCTION. Load-bearing construction shall include masonry cross-wall construction and walls of lightweight steel Section studs.

KEY ELEMENT. A structural element essential to the integrity and stability of the structure that resists abnormal loading without failure.

STRUCTURAL FRAME. The columns and other structural members including the girders, beams, trusses, and spandrels having direct connections to the columns and bracing members designed to carry gravity loads, together with their connections.

TIES. Structural elements that mechanically connect the building components to enhance continuity, ductility and redundancy.

1605.3 Building class. Buildings shall be classified in accordance with Table 1605.3. Buildings with occupancy groups within more than one classification shall be designed as the higher class.

TABLE 1605.3
BUILDING CLASS

CLASS	BUILDING TYPE AND OCCUPANCY
1	Group R-3 or R-5 not exceeding 4 stories Agricultural buildings Unoccupied buildings that are separated from other buildings by a distance of 1.5 times the buildings height.
2	Group R-3 not exceeding 5 stories Group R-1 not exceeding 4 stories Group R-2 not exceeding 4 stories Group B not exceeding 4 stories Group F not exceeding 3 stories Group M not exceeding 3 stories of less than 21,500 square feet floor area in each story. Group E not exceeding one story All buildings of Group A not exceeding 2 stories which contain floor areas not exceeding 21,500 square feet at each story. Group S buildings not exceeding 6 stories
3	Group R-1 and R-2 buildings greater than 4 stories but not exceeding 15 stories Group E buildings greater than 1 story but not exceeding 15 stories. Group M buildings greater than 3 stories but not exceeding 15 stories. Group I-2 buildings not exceeding 3 stories. Group B buildings greater than 4 stories but not exceeding 15 stories. Group A buildings which contain floors of more than 21,500 square feet but less than 54,000 square feet per floor.
4	All buildings that exceed the limits on area or number of stories for class 1-3. Grandstands accommodating more than 5000 spectators. Building containing hazardous substances and/or processes.

1605.4 Design approach: Design to protect against disproportionate collapse shall be in accordance with Section 1605.5. Alternative design approaches may be used provided that it is demonstrated that the alternative(s) chosen result in a level of structural robustness at least equivalent to that specified in Section 1605.5. For all collapse resistance approaches, verification of acceptable damage to the remaining structure outside of the collapse extent shall be determined by an analysis that allows a comparison of residual inelastic capacity to initial capacity (or a similar metric.) In every case, post-event stability of the structural system shall be verified.

1605.5 Prescriptive design approach. Design of new buildings to protect against disproportionate collapse shall be in accordance with the requirements specified below for each building class.

1605.5.1 Class 1 buildings. Class 1 buildings are not required to comply with this section.

1605.5.2 Class 2 buildings. Class 2 buildings shall be provided with horizontal ties in accordance with Section 1605.5.2.1 or with anchorage in accordance with Section 1605.5.2.2.

1605.5.2.1 Class 2 horizontal ties. Horizontal ties shall be provided in accordance with Sections 1605.6.1, 1605.6.2, and 1605.6.3, as applicable.

1605.5.2.2 Class 2 anchorage. Anchorage of suspended floors to walls shall be provided in accordance with Sections 1605.6.1, 1605.6.2, and 1605.6.3, as applicable, for load-bearing construction.

1605.5.3 Class 3 buildings. Class 3 buildings shall be provided with horizontal ties, in accordance with Section 1605.5.3.1, anchorage in accordance with Section 1605.5.3.2, and vertical ties in accordance with Section 1605.5.3.3 or shall be designed utilizing alternate load path analysis in accordance with Section 1605.5.3.4.

1605.5.3.1 Class 3 horizontal ties. Horizontal ties shall be provided in accordance with Sections 1605.6.1, 1605.6.2, and 1605.6.3, as applicable.

1605.5.3.2 Class 3 anchorage. Anchorage of suspended floors to walls shall be provided in accordance with Sections 1605.6.1, 1605.6.2, and 1605.6.3, as applicable, for load-bearing construction.

1605.5.3.3 Class 3 vertical ties. Vertical ties shall be provided in accordance with Sections 1605.6.1, 1605.6.2, and 1605.6.3, as applicable.

1605.5.3.4 Class 3 alternate load path analysis. An alternate load path analysis shall be performed in accordance with Sections 1605.6.1.8, 1605.6.2.4, 1605.6.3.1, as applicable.

1605.5.3.4.1 Class 3 Scope. For the purpose of applying the alternate load path analysis, collapse shall be deemed / when the removal of any supporting column or beam supporting one or more columns, or any nominal length of load-bearing wall (one at a time in each story of the building) causes the building to become unstable or the floor area at risk of collapse exceeds 15percent of the area of that story or 750 square feet whichever is smaller, or extends furthers than the immediate adjacent story.

1605.5.3.4.2 Class 3 key element analysis. Where the removal of columns and lengths of walls would result in an extent of damage in excess of the limit established in 1605.5.3.4.1, then such elements shall be designed as “key elements” in compliance with Section 1605.6.4.

1605.5.4 Class 4 buildings. Class 4 buildings shall comply with the requirements for Class 3 buildings in accordance with Section 1605.5.3 and a systematic risk assessment of the building shall be undertaken, identified by the risk assessment shall be accounted for in the design. A peer review of the risk assessment and of the design shall be submitted.

1605.6 Prescriptive building design requirements The details of the effective anchorage, horizontal and vertical ties, together with the design approaches for checking the integrity of the building following the removal of vertical members and the design of key elements, shall be in accordance with Section 1605.6.1 through Section 1605.6.4:

1605.6.1 Structural use of reinforced and unreinforced masonry. Design to protect against disproportionate collapse for unreinforced masonry construction shall be in accordance with 1605.6.1.1 through 1605.6.1.8. For internal masonry walls, the distance between lateral supports that are subject to a maximum length shall not exceed 2.25 times the height of the wall. For an external masonry wall, the length shall be measured between vertical lateral supports.

1605.6.1.1 Masonry general. For composite construction, such as masonry load-bearing walls with other materials for the floor and roof systems, the application of both the requirements of this section and those provided for the other materials are required. The masonry walls shall comply with the tie (vertical, peripheral, and wall) requirements or alternate load path requirements. Peripheral, internal, and column or wall ties shall be provided at each floor level and at roof level, except where the roof is of lightweight construction, no such ties need be provided at that level. Horizontal ties shall be provided by structural members or by reinforcement that is provided for other purposes.

1605.6.1.2 Masonry tie force design requirements. Load-bearing walls shall be tied from the lowest to the highest level. Reinforcement that is provided for other purposes and shall be regarded as forming part or whole of the required ties. Splices in longitudinal reinforcing bars that provide tie forces shall be lapped, welded or mechanically joined in accordance with ACI 318. Splices are not to be located near connections or mid-span. Tie reinforcing bars that provide tie forces at right angles to other reinforcing bars shall use~~d~~ 135 degree hooks with six-diameter extension, but not less than 3 inch, extension. Use the strength reduction factors □ for development and splices of reinforcement and for anchor bolts as specified in Section 3-1 of ACI 530.

1605.6.1.3 Masonry internal ties. Internal ties shall be anchored to peripheral ties at each end, or must continue as wall or column ties. Internal ties shall be straight and continuous through the entire length of the slab, beam or girder. Internal ties can be arranged in accordance with one of the following:

1. Uniformly throughout the floor or roof width, or

2. Concentrated, with a 20 foot maximum horizontal tie spacing, or
3. Within walls no more than 20 inches above or below the floor or roof and at 20 foot maximum horizontal spacing (in addition to peripheral ties spaced evenly in the perimeter zone).

1605.6.1.3.1 Masonry two-way spans. For two-way spans the internal ties shall be design to resist a required tie strengths equal to the greater of:

1. (1.0D + 1.0L)LaFt/(8475) (Kips/ft)
or
2. 1.0Ft/3.3 (Kips/ft)

Where:
D = Dead load (psf)
L = Live load (psf)
La = Lesser of: i) the greatest distance in the direction of the tied between the centers of columns or other vertical load-bearing members where this distance is spanned by a single slab or by a system of beams and slabs, or ii) 5h (ft).
h = Clear story height (ft).
Ft = "Basic Strength" = Lesser of 4.5 + 0.9 Ns) or 13.5.
Ns = Number of stories including basement(s)

1605.6.1.3.2 Masonry one-way spans. For one-way spans the internal ties shall be designed to resist a required tie strengths greater than specified in Section 1605.6.1.3.1. In the direction perpendicular to the span, the internal ties shall resist a required tie strength of Ft.

1605.6.1.4 Masonry peripheral ties. Peripheral ties shall have a required tie strength of 1.0Ft. Peripheral ties shall be 4 feet from the edge of a floor or roof or in the perimeter wall and anchor at re-entrant corners or changes of construction.

1605.6.1.5 Masonry horizontal ties to external columns and walls. Each external column and every 3.33 feet length of external load-bearing wall shall be anchored or tied horizontally into the structure at each floor and roof level with design tie strength equal to:
2.0Ft or (h/8.2)Ft, whichever is smaller (kips)

Where:
H = Clear story height (ft)
Ft = "Basic Strength" = Lesser of (4.5 + 0.9Ns) or 13.5
Ns = Number of stories including basement(s)

The tie connection to masonry shall be in accordance with ACI 530. Tie corner columns in both directions. Space wall ties, where required, uniformly along the length of the wall or concentrated at centers not more than 16.5 feet on center and not more than 8.25 feet from the end of the wall. External column and wall ties can be provided partly or wholly by the same reinforcement as peripheral and internal ties.

1605.6.1.6 Masonry vertical ties. Vertical ties shall be in accordance with this 1605.6.1.6.1 through 1605.6.1.6.3.

1605.6.1.6.1 Masonry wall requirements. Columns and load-bearing walls shall have vertical ties as required by Table 1605.6.1.6.1. Vertical ties shall be spaced at a maximum of 16.5 feet on center along the wall, and a maximum of 8.25 feet from any free end of any wall. Vertical ties shall extend from the roof level to the foundation. Vertical ties shall be fully anchored at each end and at each floor level. All joints shall be design to transmit the required tensile forces. The wall shall be constrained between concrete surfaces or other similar construction capable of providing resistance to lateral movement and rotation across the full width of the wall. Vertical ties shall be designed to resist a horizontal tensile force of Ft (kips) per 3.33 feet width.

1605.6.1.6.2 Masonry columns. A column or every 3.33 feet length of a load-bearing wall that complies with the minimum requirements of Table 1605.6.1.6.1, shall provide a required tie strength equal to:
6.2 x 10-4A(ha/t)2 or 22.5 whichever is larger. (kips)

Where:
A = Horizontal cross sectional area of the column or wall including piers, but excluding the non-load-bearing width, if any of an external wall for cavity construction (ft).

ha = Clear height of a column or wall between restraining surfaces (ft).
t = Wall thickness or column dimension (ft).

TABLE 1605.6.1.6.1
MINIMUM PROPERTIES FOR MASONRY WALLS WITH VERTICAL TIES

PROPERTY	REQUIREMENTS
Minimum thickness of a solid wall or one load-bearing wythe of a cavity wall.	6 inches
Minimum characteristic compressive strength of masonry	725 psi
Maximum ratio h_a/t	20
Allowable mortar designations	S, N

1605.6.1.6.3 Masonry load-bearing walls and columns with deficient vertical tie forces. Loadbearing elements that do not comply with the required vertical tie strength, shall be designed in accordance with Section 1605.6.1.8, the alternate load path method. Each deficient element from the structure shall be removed, one at a time, and an alternate load path analysis shall be performed to verify that the structure can bridge over the missing element. The required number of elements to be removed from the structure is given in Table 1605.6.1.6.3.

TABLE 1605.6.1.6.3
REMOVAL OF DEFICIENT MASONRY VERTICAL TIE ELEMENTS

VERTICAL LOAD-BEARING ELEMENT TYPE	DEFINITION OF ELEMENT	EXTENT OF STRUCTURE TO REMOVE IF DEFICIENT
Column	Primary structural support member acting alone	Clear height between lateral restraints
Wall Incorporating One or More Lateral Supports[a]	All external and internal load-bearing walls	Length between lateral supports or length between a lateral support and the end of the wall. Remove clear height between lateral restraints.
Wall Without Lateral Supports	All external and internal load-bearing walls	For internal walls: length not exceeding 2.25H, anywhere along the wall where H is the clear height of the wall. For external walls: Full length. For both wall types: clear height between lateral restraints.

a. Lateral supports shall be provided by the following:
  1. An intersecting or return wall tied to a wall to which it affords support, with connections capable of resisting a force of Ft in 0.45Ft in kips per foot height of wall, having a length without openings of net less than H/2 at right angles to the supported wall and having an average weight of not less than 70 psf.
  2. A pier or stiffened section of the wall not exceeding 3.33 feet in length, capable of resisting a horizontal force of 0.45Ft in kips per foot height of wall.
  3. A substantial partition at right angles to the wall having average weight of not less than 31 psf, tied with connections capable of resisting a force of 0.15Ft in kips per foot height of wall, and having a length without openings of not less than H at right angles to the supported wall.

1605.6.1.7 Masonry detailed connections for tie forces. Reinforced masonry connections and joints shall be ductile. Unreinforced masonry connections and joints shall have continuous reinforcement to ensure ductile behavior.

1605.6.1.8 Masonry alternate load path method design requirements. Alternate load path method is used to verify that the structure can bridge over removed elements. The design strengths shall be determined from ACI 530. If the design strengths are less than those in Table 1605.6.1.8, then compliance shall be in accordance with the alternate load path Section 1605.6.1.8.1 through 1605.6.1.8.8.

TABLE 1605.6.1.8
ACCEPTABILITY CRITERIA AND SUBSEQUENT ACTION FOR MASONRY

STRUCTURAL BEHAVIOR	ACCEPTABILITY CRITERIA	SUBSEQUENT ACTION FOR ALTERNATE METHOD MODEL
Element Flexure	φM_n[a]	Section 1605.6.1.8.1
Element Axial	φP_n[a]	Section 1605.6.1.8.2
Element Shear	φVnA	Section 1605.6.1.8.3
Connections	Connection Design Strength[a]	Section 1605.6.1.8.4
Deformation	Deformation Limits, defined in Table 1605.6.1.8.1.8	Section 1605.6.1.8.5

a. Nominal strengths are calculated with the appropriate material properties and over-strength factor Ω; all factors are defined per Chapter 3 of ACI 530.

1605.6.1.8.1 Masonry flexural resistance of masonry. The flexural design strength shall be equal to the nominal flexural strength multiplied by the strength reduction factor . The nominal flexural strength shall be determined in accordance with ACI 530.

1605.6.1.8.2 Masonry linear static analysis. An effective plastic hinge shall be added to the model by inserting a discrete hinge into the member at an offset from the member end if the required moment exceeds the flexural design strength and if the reinforcement layout is sufficient for a plastic hinge to form and undergo significant rotation. The location of the hinge is determined through engineering analysis.

1605.6.1.8.3 Masonry non-linear static analysis. Non-linear static analysis shall be modeled to represent post-peak flexural behavior. Flexural design strength must develop before shear failure occurs.

1605.6.1.8.4 Flexural design strength. The structural element shall be removed when the required moment exceeds the flexural design strength and shall redistribute in accordance with Section 1605.6.1.8.1.9, if the structural element is not able to develop a constant moment while undergoing continued deformation.

1605.6.1.8.5 Masonry axial resistance of masonry. The axial design strength with the applicable strength reduction factor shall be determined in accordance with Chapter 3 of ACI 530. If the connection exceeds the design strengths of Table 1605.6.1.8, remove the connection from the model. If the connections at each end of an element fail, remove the element and redistribute the loads in accordance with Section 1605.6.1.8.1.9.

1605.6.1.8.6 Masonry shear resistance of masonry. The shear design strength of the cross-section with the applicable strength reduction factor is determined in accordance with ACI 530. If the connection exceeds the design strengths of Table 1605.6.1.8, remove the connection from the model. If the connections at each end of an element fail, remove the element and redistribute the loads in accordance with Section 1605.6.1.8.1.9.

1605.6.1.8.7 Masonry connections. The connections design strength with the applicable strength reduction factor is determined in accordance with ACI 530. If the connection exceeds the design strengths of Table 1605.6.1.8, remove the connection from the model. If the connections at each end of an element fail, remove the element and redistribute the loads in accordance with Section 1605.6.1.8.1.9.

1605.6.1.8.8 Masonry deformation limits for masonry. Deformation limits shall be applied to structural members in accordance with Table 1605.6.1.8.1.8.

TABLE 1605.6.1.8.1.8
DEFORMATION LIMITS FOR MASONRY

	Class 2 and 3 buildings		Class 4 buildings	
Component	Ductility υ	Rotation, Degrees θ	Ductility υ	Rotation, Degrees θ
Unreinforced Masonry[a]	-	2	-	1
Reinforced Masonry[b]	-	7	-	2

a. Response of unreinforced masonry walls is also limited by D/t, the maximum member displacement to thickness ratio. This ratio is limited to 0.75. Compare this limit, with the rotation limits and use the most restrictive condition.
b. The ultimate resistance is based on the moment capacity using 90percent of Fy for reinforcement.

1605.6.1.8.9 Masonry loads associated with failed elements. Nonlinear Dynamic, and Linear or Nonlinear Static Analysis shall be in accordance with Section 1605.6.1.8.1.9.1 through 1605.6.1.8.1.9.3.

1605.6.1.8.9.1 Masonry nonlinear dynamic. For a Nonlinear Dynamic analysis, double the loads from the failed element to account for impact and apply them instantaneously to the section of the structure directly below the failed element, before the analysis continues. Apply the loads from the area supported by the failed element to an area equal to or smaller than the area from which they originated.

1605.6.1.8.9.2 Masonry linear or nonlinear static analysis. For a Linear or Nonlinear Static analysis, if the loads on the failed element are already doubled, as shown in Section 1605.6.1.8.9.3, then the loads from the failed element are applied to the section of the structure directly below the failed element before the analysis is re-run or continued. If the loads on the failed element are not doubled, then double them and apply them to the section of the structure directly below the failed element, before the analysis is re-run or continued. In both cases, apply the loads from the area supported by the failed element to an area equal to and smaller than the area from which they originated.

1605.6.1.8.9.3 Masonry linear and nonlinear static analysis load case. Linear and nonlinear static analysis shall have a factored load combination applied to the immediate adjacent bays and at all the floors above the removed element, using the following formula.
2.0[(0.9 or 1.2)D + (0.5L or 0.2S)] + 0.2W

Where:

D = Dead load (psf)
L = Live load (psf)
S = Snow load (psf)
W = Wind load (psf)

The adjacent bay for load-bearing wall systems shall be defined as the plan area that spans between the removed wall and the nearest load-bearing wall.

1605.6.1.8.10 Masonry loading. Perimeter ground floor columns and load-bearing walls shall be designed so that the lateral uniform load, which defines the shear capacity, is greater than the load associated with the flexural capacity.

1605.6.2 Structural use of steel. Design against disproportionate collapse for structural steel shall be in accordance with Sections 1605.6.2.1 through 1605.6.2.4.

1605.6.2.1 Steel general. For composite construction, such as concrete deck slabs on steel beams, sheet steel decking with an integral slab, and columns reinforced with structural steel shapes, the application of both the requirements of this section and those provided for reinforced concrete in ACI 318 are required. For a concrete deck slab on steel beam in which the slab is used to provide internal tie capacity, the floor system and roof system shall comply with the internal tie requirements of ACI 318, while the steel frame shall comply with the other tie requirements (vertical, peripheral, and external column) and the alternate load path requirements of this section.

1605.6.2.2 Steel material properties. The over-strength factor specified in Table 1605.6.2.2 shall be applied to calculations of the design strength for both tie forces and alternate load path method.

TABLE 1605.6.2.2
OVER-STRENGTH FACTORS FOR STRUCTURAL STEEL

STRUCTURAL STEEL	ULTIMATE OVER-STRENGTH FACTOR, Ω_u	YIELD OVER-STRENGTH FACTOR Ω_y
Hot-Rolled Structural Shapes and Bars	1.05	
ASTM A36/A36M	1.05	1.5
ASTM A572/A572M Grade 42	1.05	1.3
ASTM A992/A992M	1.05	1.1
All grades	1.05	1.1
Hollow Structural Sections	1.05	
ASTM A500, A501, A618, and A847	1.05	1.3
Steel Pipes	1.05	
ASTM A53/A53M	1.05	1.4
Plates	1.05	1.1
All other products	1.05	1.1

1605.6.2.3 Steel tie force requirements. All buildings shall be effectively tied together at each principal floor level. Each column shall be effectively held in position by means of horizontal ties in two directions, approximately at right angles, at each principal floor level supported by that column. Horizontal ties shall similarly be provided at the roof level, except where the steelwork only supports cladding that weighs not more than 14.6 psf and that carries only imposed roof loads and wind loads. Ties shall be effectively straight. Arrange continuous lines of ties as close as practical to the edges of the floor or roof and to each column line. At re-entrant corners, anchor the tie members nearest to the edge into the steel framework.

1605.6.2.3.1 Steel strength reduction factor Φ for steel tie forces. For the steel members and connections that provide the design tie strengths, use the applicable tensile strength reduction factors Φ from AISC 360.

1605.6.2.3.2 Steel horizontal steel ties. The horizontal ties may be either steel members, including those also used for other purposes, or steel reinforcement that is anchored to the steel frame and embedded in concrete, designed in accordance with ACI 318 and meeting the continuity and anchorage requirements of Section 1605.6.2.3.2.1.

1605.6.2.3.2.1 Steel continuity and anchorage of ties. Ties shall comply with Section 1605.6.2.3.2.1.1 through 1605.6.2.3.2.1.2.

1605.6.2.3.2.1.1 Splices. Splices in longitudinal steel reinforcement used to provide the design tie strength shall be lapped, welded or mechanically joined with Type 1 or Type 2 mechanical splices, in accordance with ACI 318. Splices shall be located away from joints or regions of high stress and shall be staggered.

1605.6.2.3.2.1.2 Hooks. Use seismic hooks, as defined in Chapter 21 of ACI 318, and seismic development lengths, as specified in Section 21.5.4 of ACI 318, to anchor ties to other ties. At re-entrant corners or at substantial changes in construction, ties shall be adequately developed.

1605.6.2.3.3 Steel internal ties. Design steel members acting as internal ties and their end connections shall be capable of resisting the following required tie strength, which need not be considered as additive to other loads. The required tie strength is calculated as follows:
0.5(1.2D + 1.6L)stLl but not less than 16.9 kips

Where:
D = Dead load (psf)
L = Live load (psf)
Ll = Span (ft.)
st = Mean transverse spacing of the ties adjacent to the ties being checked (ft.)

1605.6.2.3.4 Steel peripheral ties. Peripheral ties shall be capable of resisting the following load:
0.25(1.2D + 1.6L)stLl but not less than 8.4 kips

Where:
D = Dead load (psf)
L = Live load (psf)
Ll = Span (ft.)
st = Mean transverse spacing of the ties adjacent to the ties being checked (ft.)

1605.6.2.3.5 Steel tying of external columns. The required tie strength for horizontal ties anchoring the column nearest to the edges of a floor or roof and acting perpendicular to the edge is equal to the greater of the load calculated in Section 1605.6.2.3.3 or 1percent of the maximum factored vertical dead and live load in the column that is being tied, considering all load combinations used in the design.

1605.6.2.3.6 Steel vertical ties. All columns shall be continuous through each beam-to-column connection. All column splices shall provide a design tie strength equal to the largest factored vertical dead and live load reaction (from all load combinations used in the design) applied to the column at any single floor level located between that column splice and the next column splice down or the base of the column.

1605.6.2.3.7 Steel columns with deficient vertical tie forces. The alternate load path method shall be used in each deficient column, where it is not possible to provide the vertical required tie strength. Remove each deficient column from the structure, one at a time, and perform an alternate load path analysis to verify that the structure can bridge over the missing column.

1605.6.2.4 Steel alternate load path method design requirements. Alternate load path method is used to verify that the structure can bridge over removed elements. The design strengths shall be determined in accordance with AISC 360. If the design strengths are less than those in Table 1605.6.2.4.1, then compliance shall be in accordance with the alternate load path model Sections 1605.6.2.4.1 through 1605.6.2.4.5.

TABLE 1605.6.2.4.1
ACCEPTABILITY CRITERIA AND SUBSEQUENT ACTION FOR STRUCTURAL STEEL

STRUCTURAL BEHAVIOR	ACCEPTABILITY CRITERIA	SUBSEQUENT ACTION FOR VIOLATION OF CRITERIA
Element Flexure	φM_n[a]	Section 1605.6.2.4.1
Element Combined Axial and Bending	AISC LRFD Chapter H Interaction Equations[a]	Section 1605.6.2.4.2
Element Shear	φV_n[a]	Section 1605.6.2.4.3
Connections	Connection Design Strength[a]	Section 1605.6.2.4.4
Deformation	Deformation Limits, defined in Table 1605.6. 2.5(1)	Section 1605.6.2.4.5

a. Nominal strengths are calculated with the appropriate material properties and over-strength factors Ωy and Ωu depending upon the limit state; all Φ factors are defined per AISC 360.

1605.6.2.4.1 Steel flexural resistance of structural steel. A flexural member can fail by reaching its full plastic moment capacity, or it can fail by lateral-torsional buckling (LTB), flange local buckling (FLB), or web local buckling (WLB). Calculate nominal moment strength, Mn, in accordance with AISC 360. If a flexural member's capacity is governed by a buckling mode of failure, remove the element when the internal moment reaches the nominal moment strength. Distribute the loads associated with the element in accordance with Section 1605.6.2.4.1.1. If the member strength is not governed by buckling, the strength will be governed by plastification of the cross-section and it may be possible for a plastic hinge to form. Deformation of primary members shall not cause premature failure in secondary members, due to geometric interference. Torsional rotation of a girder shall not cause excessive deformation and stresses in any beam that frames into the girder with a simple shear tab connection.

1605.6.2.4.1.1 Steel formation of plastic hinge. If hinge formation, i.e. material non-linearity, is included in the alternate load path analysis, the requirements of Section A5.1 of the AISC 360 for plastic design shall be met. AISC 360 permits plastic analysis only when the structure can remain stable, both locally and globally, up to the point of plastic collapse or stabilization. Where the analysis indicates the formation of multiple plastic hinges, ensure each cross-section or connection that is assumed to form a plastic hinge is capable of not only forming the hinge, but is also capable of the deformation demands created by rotation of the hinge as additional hinges are formed in the element or structure. Since the element could be required to undergo large deformations as plastic hinges are being formed, special lateral bracing is required. The magnitude of the plastic moment, Mp, used for analysis shall consider the influence of axial or shear force when appropriate. Further information on plastic design is provided in The Plastic Methods of Structural Analysis (Neal 1963) and Plastic Design of Steel Frames (Beedle 1958).

1605.6.2.4.1.2 Steel modeling of a plastic hinge. Plastic hinges shall be modeled in accordance with Sections 1605.6.2.4.1.2.1 through 1605.6.2.4.1.2.2.

1605.6.2.4.1.2.1 Steel linear static analysis. For Linear Static analyses, when the calculated moment exceeds the nominal moment strength and it is determined that the element is capable of forming a plastic hinge, insert an "equivalent" plastic hinge into the model by inserting a discrete hinge in the member at an offset from the member end and add two constant moments, one at each side of the new hinge, in the appropriate direction for the acting moment. The magnitude of the constant moments is equal to the determined plastic moment capacity of the element. Determine the location of the plastic hinge through engineering analysis and judgment or with the guidance provided for seismic connections in FEMA 350, Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings and AISC 341, Seismic Provisions for Structural Steel Buildings.

1605.6.2.4.1.2.2 Steel nonlinear static and dynamic analysis. For Nonlinear Static and Dynamic Analysis, use software capable of representing post-peak flexural behavior and considering interaction effects of axial loads and moment. Ensure that shear failure will not occur prior to developing the full flexural design strength.

1605.6.2.4.2 Steel combined axial and flexural resistance of structural steel. The response of an element under combined axial force and flexural moment can be force controlled (i.e. non-ductile) or deformation controlled (i.e. ductile). The response is determined by the magnitude of the axial force, cross sectional properties, magnitude/direction of moments, and the slenderness of the element. If the element is sufficiently braced to prevent buckling and the ratio of applied axial force to the axial force at yield (Pu/Py where Py = AgFy) is less than 0.15, the member can be treated as deformation controlled with no reduction in plastic moment capacity, i.e. as a flexural member in accordance with Section 1605.6.2.4.1. For all other cases, treat the element as a beam-column and make the determination of whether the element is deformation or force controlled in accordance with the provisions of FEMA 356 Chapter 5.

1. When the controlling action for the element is force controlled, evaluate the strength of the element using the interaction equations in Chapter H of AISC 360, incorporating the appropriate strength reduction factors Φ and the over-strength factor Ω. Remove the element from the model when the acceptability criteria is violated and redistribute the loads associated with the element in accordance with Section 1605.6.2.4.6.
2. When the controlling action for the element is deformation controlled, the element can be modeled for inelastic action using the modeling parameters for nonlinear procedures in Table 5-6 in FEMA 356. In linear analyses, take the force deformation characteristics of the elements as bilinear (elastic – perfectly plastic), ignoring the degrading portion of the relationship specified in FEMA 356. The modeling of plastic hinges for beam-columns in linear static analyses must include a reduction in the moment capacity due to the effect of the axial force (in accordance with FEMA 356 Equation 5-4). For nonlinear analysis, the modeling of elements, panel zones, or connections must follow the guidelines in FEMA 356. Nonlinear analyses must utilize coupled (P-M-M) hinges that yield based on the interaction of axial force and bending moment. In no cases shall the deformation limits established in FEMA 356 exceed the deformation limits established in Table 1605.6.2.5(1).

1605.6.2.4.3 Shear resistance of structural steel. The acceptability criteria for shear of structural steel are based on the nominal shear strength of the cross-section, in accordance with AISC 360, multiplied by the strength reduction factor Φ and the over-strength factor Ω. If the element exceeds the design strengths of Table 1605.6.2.4.1, remove the element and redistribute the loads associated with the element in accordance with Section 1605.6.2.4.6.

1605.6.2.4.4 Steel connections. All connections shall meet the requirements of AISC 360; employ the applicable strength reduction factor Φ for each limit state and over-strength factor Ω. If a connection exceeds the design strengths of Table 1605.6.2.4.1, remove it from the model. If the connections at each end of an element fail, remove the element and redistribute the loads associated with the element in accordance with Section 1605.6.2.4.6.

1605.6.2.4.5 Deformation limits for structural steel. The Deformation Limits are given in Table 1605.6.2.5(1). Fully Restrained and Partially Restrained connections are given in Table 1605.6.2.5(2). Verify and quantify the rotational capacities of connections that are not listed in Table 1605.6.2.5(2) in accordance with the testing requirements of Appendix S of AISC 341.

1605.6.2.4.6 Steel loads associated with failed elements. Nonlinear Dynamic, and Linear or Nonlinear Static Analysis shall be in accordance with Section 1605.6.2.4.6.1through 1605.6.2.4.6.2.

1605.6.2.4.6.1 Steel nonlinear dynamic. For a Nonlinear Dynamic analysis, double the loads from the failed element to account for impact and apply them instantaneously to the section of the structure directly below the failed element, before the analysis continues. Apply the loads from the area supported by the failed element to an area equal to or smaller than the area from which they originated.

1605.6.2.4.6.2 Steel linear or nonlinear static analysis. For a Linear or Nonlinear Static analysis, if the loads on the failed element are already doubled as shown in Section 1605.6.2.4.6.3, then the loads from the failed element are applied to the section of the structure directly below the failed element before the analysis is re-run or continued. If the loads on the failed element are not doubled, then double them and apply them to the section of the structure directly below the failed element, before the analysis is re-run or continued. In both cases, apply the loads from the area supported by the failed element to an area equal to or smaller than the area from which they originated.

1605.6.2.4.6.3 Steel linear and nonlinear static analysis load case. Linear and nonlinear static analysis shall have a factored load combination applied to the immediate adjacent bays and at all the floors above the removed element, using the following formula.
2.0[(0.9 or 1.2)D + (0.5L or 0.2S)] + 0.2W]

Where:
D = Dead load (psf)
L = Live load (psf)
S = Snow load (psf)
W = Wind load (psf)

TABLE 1605.6.2.5(1)
DEFORMATION LIMITS FOR STRUCTURAL STEEL

	CLASS 2 AND 3 BUILDINGS		CLASS 4 BUILDINGS	
Component	**Ductility** μ	**Rotation,** Degrees θ	**Ductility** μ	**Rotation,** Degrees θ
Beams – Seismic Section[a]	20	12	10	6
Beams – Compact Section[a]	5		3	
Beams – Non-Compact Section[a]	1.2		1	
Plates	40	12	20	6
Columns and Beam-Columns	3		2	
Steel Frame Connections; Fully Restrained				
Welded Beam Flange or Coverplated (all types)		2.0		1.5
Reduced Beam Section		2.6		2
Steel Frame Connections; Partially Restrained				
Limit State governed by rivet shear or flexural yielding of plate, angle or T-section		2.0		1.5
Limit State governed by high strength bolt shear, tension failure of rivet or bolt, or tension failure of plate, angle or T-section		1.3		0.9

a. As defined in AISC 341.

TABLE 1605.6.2.5(2)
STEEL MOMENT FRAME CONNECTION TYPES

CONNECTION	**DESCRIPTION**	**TYPE**
Strong Axis		
Welded Unreinforced Flange	Full penetration welds between beams and columns, flanges, bolted or welded web.	FR
Welded Flange Plates	Flange plate with full-penetration weld at column and fillet welded to beam flange.	FR
Welded Cover-Plated Flanges	Beam flange and cover-plate are welded to column flange.	FR
Bolted Flanges Plates	Flange plate with full-penetration weld at column and field bolted to beam flange.	FR or PR
Improved Welded Unreinforced Flange – Bolted Web	Full-penetration welds between beam and column flanges, bolted web.	FR
Improved Welded Unreinforced Flange – Welded Web	Full-penetration welds between beam and column flanges, welded web.	FR
Free Flange	Web is coped at ends of beam to separate flanges; welded web tap resists shear and bending moment due to eccentricity due to coped web.	FR
Welded Top and Bottom Haunches	Haunched connection at top and bottom flanges.	FR
Reduced Beam Section	Connection in which net area of beam flange is reduced to force plastic hinging away from column face.	FR
Top and Bottom Clip Angles	Clip angle bolted or riveted to beam flange and column flange.	PR
Double Split Tee	Split tees bolted or riveted to beam flange and column flange.	PR
Composite Top and Clip Angle Bottom	Clip angle bolted or riveted to column flange and beam bottom flange with composite slab.	PR
Bolted Flange Plates	Flange plate with full-penetration weld at column and bolted to beam flange.	PR
Bolted End Plates	Stiffened or unstiffened end plate welded to beam and bolted to column flange.	PR
Shear Connection with or without Slab	Simple connection with shear tab, may have composite slab.	PR
Weak Axis		
Fully Restrained	Full-penetration welds between beams and columns, flanges, bolted or welded web.	FR
Shear Connection	Simple connection with shear tab.	PR

Note: PR = Partially Restrained Connections
FR = Fully Restrained Connections

1605.6.3 Structural use of plain, reinforced and prestressed concrete. Design against disproportionate collapse for concrete shall be in accordance with ACI 318 or 1605.6.3.1. For a reinforced concrete wall, the distance between lateral supports that are subject to a maximum length shall not exceed 2.25 times the height of the wall. For composite construction, such as concrete deck slabs on steel beams, sheet steel decking with an integral slab, and columns reinforced with structural steel shapes, the application of both the requirements of this section and those provided for structural steel in Section 1605.6.2 are required. For a concrete deck slab on steel beam in which the slab is used to provide internal tie capacity, the floor system and roof system shall comply with the internal tie requirements of ACI 318, while the steel frame shall comply with the other tie requirements (vertical, peripheral, and external column).

1605.6.3.1 Concrete alternate load path method design requirements. Alternate load path method is used to verify that the structure can bridge over removed elements. The design strengths shall be determined in accordance with ACI 318. If the design strengths are less than those in Table 1605.6.3.1, then compliance shall be in accordance with the alternate load path model Sections 1605.6.3.1.1 through 1605.6.3.1.6.

TABLE 1605.6.3.1
ACCEPTABILITY CRITERIA AND SUBSEQUENT ACTION FOR REINFORCED CONCRETE

STRUCTURAL BEHAVIOR	ACCEPTABILITY CRITERIA	SUBSEQUENT ACTION FOR VIOLATION OF CRITERIA
Element Flexure	φM_n[a]	Section 1605.6.3.1.2
Element Combined Axial and Bending	ACI 318 Chapter 10 Provisions[a]	Section 1605.6.3.1.3
Element Shear	φV_n[a]	Section 1605.6.3.1.4
Connections	Connection Design Strength[a]	Section 1605.6.3.1.5
Deformation	Deformation Limits, defined in Table 1605.6.3.1.6	Section 1605.6.3.1.6

Nominal strengths are calculated with the appropriate material properties and over-strength factors Ωy and Ωu depending upon the limit state; all Φ factors are defined in accordance with ACI 318.

1605.6.3.1.1 Over-strength factors for reinforced concrete. The applicable over-strength factor shall be applied to calculations of the design strength alternate load path method. The over-strength factors are given in Table 1605.6.3.1.1.

TABLE 1605.6.3.1.1
OVER-STRENGTH FACTORS FOR REINFORCED CONCRETE

REINFORCED CONCRETE	OVER-STRENGTH FACTOR, Ω
Concrete Compressive Strength	1.25
Reinforcing Steel (ultimate and yield strength)	1.25

1605.6.3.1.2 Flexural resistance of reinforced concrete. The flexural design strength shall be equal to the nominal flexural strength calculated with the appropriate material properties and over-strength factor Ω, multiplied by the strength reduction factor of 0.75. The nominal flexural strength shall be calculated in accordance with ACI 318.

1605.6.3.1.2.1 Concrete linear static analysis. For linear static analysis when the required moment exceeds the flexural design strength and when the reinforcement layout is sufficient for a plastic hinge to form and undergo significant rotation, an equivalent plastic hinge shall be added to the model by inserting a discrete hinge at the correct location within the member. The location of the hinge shall be determined through engineering analysis, but shall be less than ½ the depth of the member from the face of the column. Apply two constant moments, one at each side of the new hinge, in the appropriate direction of the acting moment.

1605.6.3.1.2.2 Concrete non-linear static and dynamic analysis. For non-linear static and dynamic analysis shall be model to represent post-peak flexural behavior. Flexural design strength must develop before shear failure occurs.

1605.6.3.1.2.3 Flexural design strength. The structural element shall be removed when the required moment exceeds the flexural design strength and shall be redistributed in accordance with Section 1605.6.3.2, when the structural element is not able to develop a constant moment while undergoing continued deformation.

1605.6.3.1.3 Combined axial and bending resistance of reinforced concrete. The acceptability criteria for elements undergoing combined axial and bending loads are based on the provisions given in Chapter 10 of ACI 318, including the appropriate strength reduction factor Φ and the over-strength factor Ω. If the combination of axial load and flexure in an element exceeds the design strength and the un-factored axial load is greater than the nominal axial load strength at balanced strain Pb, remove the element and redistribute the loads associated with the element in accordance with Section 1605.6.3.2. If the un-factored axial load is less than Pb, then insert an equivalent plastic hinge into the column, in accordance with the procedure in Section 1605.6.3.1.2.

1605.6.3.1.4 Shear resistance of reinforced concrete. The acceptability criteria for shear are based on the shear design strength of the cross-section, in accordance with ACI 318, using the appropriate strength reduction factor Φ and the over-strength factor Ω. When the element violates the shear criteria, remove the element and redistribute the loads associated with the element in accordance with Section 1605.6.3.2.

1605.6.3.1.5 Concrete connections. The connections design strength with the applicable strength reduction factor □ shall be determined in accordance with ACI 318. The effects of embedment length, reinforcement continuity, and confinement of reinforcement in the joint shall be considered when determining the joint design strength. When the connection exceeds the design strengths of Table 1605.6.3.1, remove it from the model. When the connections at each end of an element fail, remove the element and redistribute the loads associated with the element in accordance with Section 1605.6.3.2.

1605.6.3.1.6 Deformation limits for reinforced concrete. Deformation limits shall be applied to structural members in accordance with Table 1605.6.3.1.6. When the element or the connections at each end of an element exceed the deformation limit in Table 1605.6.3.1.6, remove the element and redistribute the loads associated with the element in accordance with Section 1605.6.3.2. Deformation limits are applied only to the structural elements, not to the connections.

TABLE 1605.6.3.1.6
DEFORMATION LIMITS FOR REINFORCED CONCRETE

	CLASS 2 & 3 BUILDINGS		CLASS 4 BUILDINGS	
Component	**Ductility** υ	**Rotation,** Degrees θ	**Ductility** υ	**Rotation,** Degrees θ
Slab and Beam Without Tension Membrane[a]				
Single-Reinforced or Double-Reinforced without Shear Reinforcing[b]	-	3	-	2
Double-Reinforced with Shear Reinforcing[c]	-	6	-	4
Slab and Beam with Tension Membrane[a]				
Normal Proportions (L/h ≥ 5)	-	20	-	12
Deep Proportions (L/h < 5)	-	12	-	8
Compression Members				
Walls and Seismic Columns[d,e]	3	-	2	-
Non-Seismic Columns[e]	1	-	0.9	-

a. The tension membrane effect is an extension of the yield line theory of slabs and it increases the ultimate resistance. It cannot be developed when the slab has a free edge.
b. Single-reinforced members have flexural bars in one face or mid-depth only. Double-reinforced members have flexural reinforcing in both faces.
c. Stirrups or ties meeting ACI 318 minimums must enclose the flexural bars in both faces, otherwise use the response limits for Double-Reinforced without shear reinforcing.
d. Seismic columns have ties or spirals in accordance with ACI 318 Chapter 21 seismic design provisions for special moment frames.
e. Ductility of compression members is the ratio of total axial shortening to axial shortening at the elastic limit.

1605.6.3.2 Concrete loads associated with failed elements. The following procedure shall be met for Nonlinear Dynamic, and Linear or Nonlinear Static Analysis.

1605.6.3.2.1 Concrete nonlinear dynamic. For a Nonlinear Dynamic analysis, double the loads from the failed element to account for impact and apply them instantaneously to the section of the structure directly below the failed element, before the analysis continues. Apply the loads from the area supported by the failed element to an area equal to or smaller than the area from which they originated.

1605.6.3.2.2 Concrete linear or nonlinear static analysis. For a Linear or Nonlinear Static analysis, when the loads on the failed element are already doubled as shown in Section 1605.6.2.4.7.3, then the loads from the failed element are applied to the section of the structure directly below the failed element, before the analysis is re-run or continued. When the loads on the failed element are not doubled, then double them and apply them to the section of the structure directly below the failed element, before the analysis is re-run or continued. In both cases, apply the loads from the area supported by the failed element to an area equal to and smaller than the area from which they originated.

1605.6.3.2.3 Concrete linear and nonlinear static analysis load case. Linear and nonlinear static analysis shall have a factored load combination applied to the immediate adjacent bays and at all the floors above the removed element, using the following formula.
2.0[(0.9 or 1.2)D + (0.5L or 0.2S)] + 0.2W

Where:
D = Dead load (psf)
L = Live load (psf)
S = Snow load (psf)
W = Wind load (psf)

1605.6.4 Key elements analysis. When applying the alternate load path method design requirements from Sections 1605.6.1.8, 1605.6.2.4 or 1605.6.3.1 and the removal of columns and lengths of walls result in a disproportionate collapse, then such element shall be designed to withstand an accidental design loading of 700 psf applied in the horizontal and vertical directions (in one direction at a time) to the member and any attached components.

1605.6.4.1 Load combinations. The following load combinations shall be used in addition to the accidental design loading in the key element analysis:
1.2D + Ak + (0.5L or 0.2S)
(0.9 or 1.2)D + Ak + 0.2W
As per the definition of key element, *Ak* = 700 psf.

Reason: This is the second time this proposal is being brought forward for adoption by the International Code Council Ad Hoc Committee on Terrorism Resistant Buildings. There is a commitment from NCSEA Ad Hoc Joint Industry Committee on Structural Integrity to develop an alternative to this proposal. There is also a commitment from ASCE/SEI Progressive Collapse Guidance and Standard Committee to develop a standard that will most likely be available by 2009.

The purpose of this proposal is to increase the robustness of building structural systems to guard against the possibility of collapse, property loss, and casualties that are disproportionate to the original damaging event. Such a scenario is often called progressive collapse. Incredible as it may seem, our codes and standards do not, in any way prohibit a structural system that is, literally, the proverbial “house of cards”.

This proposal is intended to implement the very first recommendation of the National Institute of Standards and Technology’s (NIST) report on the World Trade Center (WTC) tragedy. It is very important to understand that neither the NIST Report nor the proponents of this change seek to make buildings immune to attack by airliners. Rather, the WTC event resulted in a detailed examination of the adequacy of our codes in connection with a wide variety of much less dramatic damage scenarios, including now, for the first time, some that might be willful and deliberate. The Code and the many standards that it references deal comprehensively and thoroughly with the live and dead loads that buildings routinely encounter, including exceptional but predictable extreme loads such as wind and seismic. The Code does not deal at all with damage, accidental or deliberate. The possibility of deliberate damage was brought home by the WTC tragedy but it has always existed. The same is true with accidental damage. Whether a bomb, a gas explosion, or a vehicle accidentally taking out a ground level column, it is simply unacceptable that the current code would permit structural systems that are prone to total progressive collapse following a relatively minor initiating event.

The proponents believe that the Code should establish a strong public policy against disproportionate damage and progressive collapse. This proposal also includes detailed technical requirements. Those would be better included as standards that could be referenced. The near complete absence of detailed technical design requirements from American standards means that they have to be included here. Only ACI 318-02 contains any technical requirements, and those are only applicable to the “tie forces” approach in concrete design. That standard is referenced by this proposal and detailed technical requirements for that subject are not included in the proposal.

The need for such standards has been debated for years in the technical community. That debate has resulted in little but inaction. While the American debate droned on, the rest of the English speaking world, indeed much of the rest of the world has adopted effective provisions to guard against progressive collapse. Key federal agencies, such as the General Services Administration and the Department of Defense, have prepared and adopted workable and effective provisions for their buildings. The International Building Code remains silent on the issue. The time for silence has long since passed. The proponents believe that the Code Officials who are the International Codes Council, and who are those upon whom the American public relies for their safety in buildings, need to take the lead on this very important issue.

The approach to preventing disproportionate damage and progressive collapse taken by this proposal is not new. It is based upon provisions that have been a part of British Codes for a generation. The approaches have been adopted by most of the nations of the Commonwealth and are incorporated within the Eurocodes. Over the last thirty (30) years they have proven to be workable, readily applied, and have little impact on hard construction cost. They do require additional engineering analysis and careful detailing of connections. They are not unlike the seismic provisions of the code in that respect.

The proposal provides for two approaches to design for limiting disproportionate damage. The first incorporated in proposed Section 1605.4, sets forth criteria for a performance design approach to be carried out in accordance with accepted engineering practice. The second, incorporated in proposed Section 1605.5, lays out a prescription “deemed to comply” approach. Either is acceptable to demonstrate compliance. The provisions of proposed Section 1605.5 are largely based on the methods prescribed by the General Services Administration and the Department of Defense’s Uniform Facilities Criteria that have been in use for a number of years, but also references relevant provisions of ACI 318-02.

1604.11 – establishes the basic requirement that structures be designed to resist disproportionate collapse.
1605.1 – sets forth the basic standard that the Code will require be met
1605.2 – provides definitions needed to understand and apply the Sections.
1605.3 – establishes a four level classification system for all buildings by size and by occupancy group.

It is generally true that, in the Code, requirements vary by risk. Risk includes both the probability of an issue and the scale of its consequences. The higher the risk (either probability or consequences), the higher the code requirements that can be justified. It is well settled in the Code that risk varies by occupancy group and by size. Numerous Code provisions are differentiated along those lines. So it is with disproportionate collapse.

The four classifications provided are not arbitrary nor do they rely upon “seat of the pants” judgment. They reflect the classifications found in the British Codes. Those classifications were established through a very detailed and scientific risk analysis. The analysis is an available public document and is listed in the bibliography.

1605.4 – sets forth the criteria for the performance design approach.
Different requirements are set forth for each of the four (4) classes established by Section 1605.5

> Class 1 buildings are not required to comply.
> Class 2 buildings are required to have effective horizontal ties.
> Class 3 buildings are required to have effective horizontal and vertical ties or be analyzed in accordance with the alternate load path approach.
> Class 4 buildings are required to comply with the same requirements as Class 3 buildings, but they are also required to be analyzed in accordance with a peer reviewed systematic risk assessment which takes into account the hazards associated with that specific building and its specific structural system.

Specific requirements are set forth for masonry (1605.6.1), steel (1605.6.2), and concrete (1605.6.3). 1605.4 sets forth the prescription "deemed to comply" designs approach.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S60–07/08
1605.1, 1808.2.23.1.1, 1808.2.23.2.1, 1808.2.23.2.2, 1808.2.23.2.3, 1908.1.12

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

Revise as follows:

1605.1 (Supp) General. Buildings and other structures and portions thereof shall be designed to resist:

1. The load combinations specified in Section 1605.2, 1605.3.1 or 1605.3.2,
2. The load combinations specified in Chapters 18 through 23, and
3. The load combinations with overstrength factor specified in Section 12.4.3.2 of ASCE 7 where required by Section 12.2.5.2, 12.3.3.3 or 12.10.2.1 of ASCE 7. <u>With the simplified procedure of ASCE 7 Section 12.14, the load combinations with overstrength factor of Section 12.14.3.2 of ASCE 7 shall be used.</u>

~~With the simplified procedure of ASCE 7 Section 12.14, the overstrength factor load combinations of Section 12.14.3.2 of ASCE 7 shall be used.~~ Applicable loads shall be considered, including both earthquake and wind, in accordance with the specified load combinations. Each load combination shall also be investigated with one or more of the variable loads set to zero.

The load combinations with overstrength factor in Section 12.4.3.2 of ASCE 7 shall be used ~~in lieu of the following~~ <u>as follows</u>:

1. The ~~load~~ <u>Basic</u> Combinations for Strength Design <u>with Overstrength Factor</u> in lieu of Equations 16-5 and 16-7 in Section 1605.2.1.
2. The ~~load~~ <u>Basic</u> Combinations for Allowable Stress Design <u>with Overstrength Factor</u> in lieu of Equations 16-12, 16-13 and 16-15 in Section 1605.3.1.
3. The ~~load~~ <u>Basic</u> Combinations for Allowable Stress Design <u>with Overstrength Factor</u> in lieu of Equations 16-20 and 16-21 in Section 1605.3.2.

1808.2.23.1.1 Connection to pile cap. Concrete piles and concrete-filled steel pipe piles shall be connected to the pile cap by embedding the pile reinforcement or field-placed dowels anchored in the concrete pile in the pile cap for a distance equal to the development length. For deformed bars, the development length is the full development length for compression or tension, in the case of uplift, without reduction in length for excess area. Alternative measures for laterally confining concrete and maintaining toughness and ductile-like behavior at the top of the pile will be permitted provided the design is such that any hinging occurs in the confined region.

Ends of hoops, spirals and ties shall be terminated with seismic hooks, as defined in Section 21.1 of ACI 318, turned into the confined concrete core. The minimum transverse steel ratio for confinement shall not be less than one-half of that required for columns.

For resistance to uplift forces, anchorage of steel pipe (round HSS sections), concrete-filled steel pipe or H-piles to the pile cap shall be made by means other than concrete bond to the bare steel section.

> **Exception:** Anchorage of concrete-filled steel pipe piles is permitted to be accomplished using deformed bars developed into the concrete portion of the pile.

Splices of pile segments shall develop the full strength of the pile, but the splice need not develop the nominal strength of the pile in tension, shear and bending when it has been designed to resist axial and shear forces and moments from the load combinations ~~of Section 1605.4~~ with overstrength factor in Section 12.4.3.2 of ASCE 7.

1808.2.23.2.1 (Supp) Design details for piers, piles and grade beams. Piers or piles on Site Class E or F sites, as determined in Section 1613.5.2, shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains modified for soilpile structure interaction coupled with pier or pile deformations induced by lateral pier or pile resistance to structure seismic forces.

> **Exception:** Piers or piles that satisfy the following additional detailing requirements shall be deemed to comply with the curvature capacity requirements of this section.
>
> 1. Precast prestressed concrete piles detailed in accordance with Section 1809.2.3.2.2.
> 2. Cast-in-place concrete piles with a minimum longitudinal reinforcement ratio of 0.005 extending the full length of the pile and detailed in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 as required by this section.

Where constructed of nonprestressed concrete such piers or piles shall be designed and detailed in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within seven pile diameters of the pile cap and within seven pile diameters of the interfaces of strata that are hard or stiff and strata that are liquefiable or are composed of soft to medium stiff clay.

Grade beams shall comply with the provisions in Section 21.10.3 of ACI 318 for grade beams, except where they have the capacity to resist the forces from the load combinations ~~in Section 1605.4.4~~ with overstrength factor in Section 12.4.3.2 of ASCE 7.

1808.2.23.2.2 Connection to pile cap. For piles required to resist uplift forces or provide rotational restraint, design of anchorage of piles into the pile cap shall be provided considering the combined effect of axial forces due to uplift and bending moments due to fixity to the pile cap. Anchorage shall develop a minimum of 25 percent of the strength of the pile in tension. Anchorage into the pile cap shall be capable of developing the following:

1. In the case of uplift, the lesser of the nominal tensile strength of the longitudinal reinforcement in a concrete pile, or the nominal tensile strength of a steel pile, or the pile uplift soil nominal strength factored by 1.3 or the axial tension force resulting from the load combinations ~~of Section 1605.4~~ with overstrength factor in Section 12.4.3.2 of ASCE 7.
2. In the case of rotational restraint, the lesser of the axial and shear forces, and moments resulting from the load combinations ~~of Section 1605.4~~ with overstrength factor in Section 12.4.3.2 of ASCE 7 or development of the full axial, bending and shear nominal strength of the pile.

1808.2.23.2.3 Flexural strength. Where the vertical lateral-force-resisting elements are columns, the grade beam or pile cap flexural strengths shall exceed the column flexural strength.

The connection between batter piles and grade beams or pile caps shall be designed to resist the nominal strength of the pile acting as a short column. Batter piles and their connection shall be capable of resisting forces and moments from the load combinations ~~of Section 1605.4~~ with overstrength factor in Section 12.4.3.2 of ASCE 7.

1908.1.12 ACI 318, Section 21.12.5. Modify ACI 318, Section 21.12.5, by adding new Section 21.12.5.6 to read as follows:

21.12.5.6 – Columns supporting reactions from discontinuous stiff members, such as walls, shall be designed for the ~~special~~ load combinations <u>with overstrength factor in Section 12.4.3.2 of ASCE 7</u> ~~in Section 1605.4 of the International Building Code~~ and shall be provided with transverse reinforcement at the spacing, s_o, as defined in 21.12.5.2 over their full height beneath the level at which the discontinuity occurs. This transverse reinforcement shall be extended above and below the column as required in 21.4.4.5.

Reason: Code clarification.

Removes vestigial references to the now absent Section 1605.4. Removes the incorrect "in lieu of" that introduces a series of items containing "in lieu of". Revises the terminology to be consistent with ASCE Section 12.14.3.2. In the second numbered list, revises the terminology for load combinations (including capitalization) to be perfectly consistent with the designations in Section 12.4.3.2 of ASCE 7.

For three reasons, moves the first sentence of the second paragraph so that it appears in item 3 of the numbered list. First, items 1 and 2 apply where the simplified procedure is used. Second, the first sentence of the second paragraph specifies how overstrength comes into play where the simplified procedure is used. Third, the last two sentences of the second paragraph are meant to apply in all cases, not just where the simplified procedure is used.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S61–07/08
1605.1.1 (New)

Proponent: William Sherman, CH2M HILL, representing himself

Add new text as follows:

<u>**1605.1.1 Stability.** 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.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S62–07/08
1602, 1605.2.2, 1605.3.1.2, 1612.4

Proponent: Rebecca C. Quinn, RC Quinn Consulting, Inc., representing U.S. Homeland Security, Federal Emergency Management Agency

Revise as follows:

SECTION 1602
DEFINITIONS AND NOTATIONS

F_a = Flood load <u>in accordance with Chapter 5 of ASCE 7</u>.

1605.2.2 ~~Other~~ <u>Flood</u> loads. Where <u>flood loads</u> F_a ~~is~~ <u>are</u> to be considered in the design, the load combinations of Section 2.3.3 of ASCE 7 shall be used.

1605.3.1.2 ~~Other~~ Flood loads. Where flood loads F_a ~~is~~ are to be considered in design, the load combinations of Section 2.4.2 of ASCE 7 shall be used.

1612.4 Design and construction. The design and construction of buildings and structures located in flood hazard areas, including flood hazard areas subject to high velocity wave action, shall be in accordance with Chapter 5 of ASCE 7 and with ASCE 24.

Reason: The purpose of the code changes in 1605 is to clarify use of the notation for flood loads. Section 1602 defines F_a to mean flood load (see under "Notations"). Sections 1605.2.2 and 1605.3.1.2 no longer contain provisions for loads other than flood loads, and IBC Section 1613 uses " F_a" to mean something other than flood loads. The proposed change makes it clear where F_a means flood loads.

The purpose of the code change in 1612 is to more clearly direct the user to Chapter 5 of ASCE 7, which is where flood loads are specified.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S63–07/08
1605.3.1

Proponent: W. Lee Shoemaker, Ph.D., PE, Thomas Associates, Inc., representing Metal Building Manufacturers Association

Revise as follows:

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

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

Exceptions:

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

Reason: Equation 16-13 of the allowable stress design basic load combinations is not consistent with the LRFD and alternate basic load combinations because roof live loads are not additive with earthquake loads in those combinations (Equation 16-5 and Equation 16-20, respectively). The reason that roof live load is not considered in conjunction with earthquake loads is that roof live loads are produced during maintenance and are of low probability of occurring simultaneously with earthquake loads. This proposed change makes all three load combination sets correct and consistent. An alternate change was considered to revise the load combinations directly and not put this in the exception, but it would require splitting Equation 16-13 into two equations because the case with wind should include the roof live load. It was felt that the addition to the exception was the cleanest solution and not requiring an additional load combination.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S64–07/08
1605.3.2, 1605.3.2.2 (New), 2306.5 (New)

Proponent: Dennis Pitts, American Forest & Paper Association, representing American Forest & Paper Association

1. Revise as follows:

1605.3.2 Alternative basic load combinations. In lieu of the basic load combinations specified in Section 1605.3.1, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following combinations. When using these alternative basic load combinations that include wind or seismic loads, allowable stresses are permitted to be increased or load combinations reduced where permitted by the material chapter of this code or the referenced standards. Stress increases in accordance with Chapter 23 are permitted to be used in combination with load reductions in Section 1605.3.2.2. For load combinations that include the counteracting effects of dead and wind loads, only two-thirds of the minimum dead load likely to be in place during a design wind event shall be used. Where wind loads are calculated in accordance with Chapter 6 of ASCE 7, the coefficient ω in the following equations shall be taken as 1.3. For other wind loads, ω shall be taken as 1. When using these alternative load combinations to evaluate sliding, overturning and soil bearing at the soil-structure interface, the reduction of foundation overturning from Section 12.13.4 in ASCE 7 shall not be used. When using these alternative basic load combinations for proportioning foundations for loadings, which include seismic loads, the vertical seismic load effect, *Ev*, in Equation 12.4-4 of ASCE 7 is permitted to be taken equal to zero.

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

Exceptions:

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

1605.3.2.2 Load reduction. Where permitted by the material chapter of this code or the referenced standard, the combined effects of two or more loads, excluding dead load, shall be permitted to be multiplied by 0.75. The combined load used in design shall not be less than the sum of the effects of dead load and any single load that produces the largest effect.

2. Add new text as follows:

2306.5 Alternative basic load combination. The load reduction in 1605.3.2.2 shall be permitted where load combinations of 1605.3.2 are used. Stress increases in accordance with Chapter 23 are permitted to be used in combination with load reductions in Section 1605.3.2.2.

Reason: Load reductions for load cases involving two or more transient loads are defined consistent with load reductions incorporated in basic load combinations for Allowable Stress Design and those included in Appendix A of AISI North American Cold-Formed Steel specification (AISI-NAS 2001).

Section 1605.3.2.2 is added to allow consistent application of load reductions for materials designed in accordance with the alternative basic load combinations where such reductions are permitted by the material chapter of the code or the referenced standard.

A new section is added to 2306 (Allowable Stress Design) specifically permitting use of the load reduction in 1605.3.2.2 where alternative basic load combinations for ASD are used. The sentence “Stress increases in accordance with Chapter 23 are permitted to be used in combination with load reductions in Section 1605.3.2.2.” is repeated in 2306.5 for clarity. Load reductions address reduced probability of occurrence of maximum values of multiple transient loads. Stress increases in Chapter 23 adjust reference design values for wood construction to applicable design condition such as to account for load duration and repetitive member use.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S65–07/08
1605.2.1, 1605.4 (New)

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

1. Add new text as follows:

1605.4. Load combinations for fire-resistance-rated building elements. Where a fire-resistance-rated building element is required to be designed to resist the effects of a design fire, the building element shall be designed to resist the load combinations specified in Section 1605.4.1 or 1605.4.2 in addition to the load combinations of Section 1605.2 or 1605.3 for nonfire design conditions. Each load combination shall be investigated with one or more of the variable loads set to zero.

1605.4.1. Load combinations using strength design or load and resistance factor design. Where strength design or load and resistance factor design is used, the building elements shall resist the most critical effects from the following combinations of factored loads:

$1.2D + A_k + (0.5L \text{ or } 0.2S)$	**Equation 16-22**
$1.2D + A_k + 0.2W$	**Equation 16-23**
$0.9D + A_k + 0.2W$	**Equation 16-24**

1605.4.2. Load combinations using allowable stress design. Where allowable stress design, as permitted by this code, is used, the building elements shall resist the most critical effects from the following combinations of loads:

$D + A_k + 0.5L \text{ or } 0.2S$	**Equation 16-25**
$D + A_k + 0.2W$	**Equation 16-26**
$0.6D + A_k + 0.2W$	**Equation 16-27**

Exception: Crane hook loads are not required to be included with the additional load combinations.

where:

A_k = load due to the design fire.

(Renumber subsequent sections)

2. Revise as follows:

1605.2.1 Basic load combinations. Where strength design or load and resistance factor design is used, structures and portions thereof shall resist the most critical effects from the following combinations of factored loads:

$1.4(D + F)$	**(Equation 16-1)**
$1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$	**(Equation 16-2)**
$1.2D + 1.6(L_r \text{ or } S \text{ or } R)+(f_1L \text{ or } 0.8W)$	**(Equation 16-3)**
$1.2D + 1.6W + f_1L + 0.5(L_r \text{ or } S \text{ or } R)$	**(Equation 16-4)**
$1.2D + 1.0E + f_1L + f_2S$	**(Equation 16-5)**
$0.9D + 1.6W + 1.6H$	**(Equation 16-6)**
$0.9D + 1.0E + 1.6H$	**(Equation 16-7)**

where:

f_1 = 1 for floors in places of public assembly, for live loads in excess of 100 pounds per square foot (4.79 kN/m^2), and for parking garage live load, and
= 0.5 for other live loads.

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

~~**Exception:** Where other factored load combinations are specifically required by the provisions of this code, such combinations shall take precedence.~~

Reason: Building elements are frequently required by the nonstructural provisions of the IBC and other codes promulgated by the International Code Council to be protected against the effects of fire and other hazards by means of fire-resistance-rated assemblies. The fire resistance ratings of building elements are determined in accordance with the test procedures of ASTM E 119 and other applicable standards (refer to IBC Section 703.2). There are also alternative methods of determining fire resistance including calculations in accordance with Section 721 (refer to IBC Section 703.3). "Building element" is not defined in the IBC but the term is an integral part of its fire safety provisions (i.e., Section 602 and Table 601).

Section 1605 specifies load combinations for the structural design of buildings and other structures. Included in the load combinations are loads for natural or manmade events that are typically associated with structural design (e.g., occupancy, snow, wind, earthquakes, earth pressure, etc.). Combinations that consider a fire in a building or structure, however, are not included. The primary purpose for the nonstructural provisions of the IBC is to protect buildings and structures from the effects of fire and other hazards but the structural provisions do not account for this by specifying load combinations applicable to these hazards.

The effects of a fire on a building or structure typically do not cause structural demands that need to be added to current structural load combinations. There are circumstances, however, when structural load combinations need to be applied to a building element subjected to a fire or other hazard. In these cases, the building element should be designed for load combinations appropriate to the fire (or other hazard) event. The IBC, however, does not specify load combinations for this design condition. For example, the live load on a floor member in combination with dead load and other design loads, as applicable, is unknown. Where a building element is designed to be load-bearing and fire-resistance-rated, should the floor live load in applicable load combinations be 100 percent of the design live load specified in Table 1607.1? Or, should the floor live load be assumed as zero?

This proposal resolves the problem by specifying load combinations for the fire condition. Sources for the load combinations are Commentary Section C2.5 of "Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)," by ASCE; Section X.3 of "Steel Design Guide 19: Fire Resistance of Structural Steel Framing," 2003, by AISC; and Appendix Section 4.1.1 of the "Specification for Structural Steel Buildings" (AISC 360-05), also by AISC. Note that Section 2.5 of ASCE 7-05 requires structures to be checked for strength and stability to ensure that they are capable of withstanding the effects of extraordinary events, such as fires, "where required by the applicable code, standard or authority having jurisdiction." This requirement, however, does not exist in the 2006 IBC and the lack of a requirement is what has prompted me to submit this proposal.

The load combinations recommended by ASCE and AISC in the documents noted above are intended for the design of buildings and structures subject to the general case of an extraordinary event. A load effect from the extraordinary event is included in each of the load combinations in those documents and is included in each of the load combinations in this proposal. The load combinations in this proposal, however, are intended for the specific case of a fire, not the general case of an extraordinary event. As noted above, the load effects due to fire are typically not additive when considering their effect together with structural load combinations. Their effects can also be counteracting but this is typically rare. Thus, the additive and counteracting effects of fire on a building element designed to be load-bearing and fire-resistance-rated could be assumed to be zero but this proposal does not specify such a result. It will be left to other provisions of the IBC with design conditions applicable to building elements required to resist the effects of a design fire or equivalent to specify the quantities of the load effect.

The overall effect of this proposal is that a building element will still be required to resist the load combinations specified in Section 1605.2 or 1605.3, which will continue to apply to nonfire design conditions. Where the same building element is fire-resistance-rated and is required by other provisions of the IBC to resist the effects of a design fire, however, the building element will also be required to resist the load combinations specified in Section 1605.4.1 or 1605.4.2. These load combinations will typically be utilized to meet the limitations on structural capacity imposed by the fire-resistance-rated assembly for the building element. A prominent example of this application are the limits placed on fire-resistance-rated wood stud bearing walls by certain fire-resistance-rated assemblies specified in IBC Table 720.1(2) and listed by nationally recognized testing laboratories.

It should be noted that the load combinations in proposed Section 1605.4.2 for allowable stress design are identical to the load combinations in proposed Section 1605.4.1 for strength design or load and resistance factor design. This is intentional. The literature on this subject typically focuses on strength design or load and resistance factor design. Until such time as the recommendations for allowable stress design are brought forward, the more conservative factored load combinations of Section 1605.4.1 are proposed for use with allowable stress design.

The exception to Section 1605.2.1 is judged to be archaic and is deleted. The exception served a purpose in the 2000 and 2003 editions of the IBC when the load combinations of Section 1605.2 were used in conjunction with the load combinations of Section 1617.1 that accounted for seismic load effects. In the 2006 IBC, however, Section 1617.1 is replaced by referencing ASCE 7 in Section 1613.1, and in the 2007 IBC Supplement, Section 1605.4 is replaced by references to the load combination with overstrength of ASCE 7 in Section 1605.1. The original purpose for the exception to Section 1605.2.2 has been supplanted by these references. Retaining the exception could lead to interpretations by practitioners that are not intended. It should be noted that, to my knowledge, there are currently no factored load combinations in the 2006 IBC as modified by the 2007 IBC Supplement other than in Section 1605.2.1.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S66–07/08

406.3.3, Table 1607.1, 1607.7, 1607.7.1, 1607.7.1.1, 1607.7.2, 1607.7.3

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

Revise as follows:

406.3.3 Construction. Open parking garages shall be of Type I, II or IV construction. Open parking garages shall meet the design requirements of Chapter 16. For vehicle barrier~~s~~ systems, see Section 406.2.4.

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
Vehicle barrier~~s~~ systems	See Section	1607.7.3

(Portions of table not shown remain unchanged)

1607.7 Loads on handrails, guards, grab bars, seats and vehicle barrier~~s~~ systems. Handrails, guards, grab bars ~~as designed in ICC A117.1~~, accessible seats, accessible benches and vehicle barrier~~s~~ systems shall be designed and constructed to the structural loading conditions set forth in this section.

1607.7.1 Handrails and guards. Handrails ~~assemblies~~ and guards shall be designed to resist a load of 50 plf (0.73 kN/m) applied in any direction at the top and to transfer this load through the supports to the structure. Glass handrail assemblies and guards shall also comply with Section 2407.

Exceptions:

1. For one- and two-family dwellings, only the single concentrated load required by Section 1607.7.1.1 shall be applied.
2. In Group I-3, F, H and S occupancies, for areas that are not accessible to the general public and that have an occupant load less than 50, the minimum load shall be 20 pounds per foot (0.29 kNm).

1607.7.1.1 Concentrated load. Handrails ~~assemblies~~ and guards shall be able to resist a single concentrated load of 200 pounds (0.89 kN), applied in any direction at any point along the top, and have attachment devices and supporting structure to transfer this loading to appropriate structural elements of the building. This load need not be assumed to act concurrently with the loads specified in the preceding paragraph.

1607.7.2 Grab bars, ~~shower~~ seats and ~~dressing room bench seats~~ benches. Grab bars in accessible toilet and bathing facilities, ~~shower~~ seats in accessible bathtubs and shower compartments and ~~dressing room bench seat systems~~ accessible benches in accordance with ICC A117.1 shall be designed to resist a single concentrated load of 250 pounds (1.11 kN) applied in any direction at any point.

1607.7.3 Vehicle barrier~~s~~ systems. Vehicle barrier systems for passenger cars shall be designed to resist a single load of 6,000 pounds (26.70 kN) applied horizontally in any direction to the barrier system and shall have anchorage or attachment capable of transmitting this load to the structure. For design of the system, the load shall be assumed to act at a minimum height of 1 foot, 6 inches (457 mm) above the floor or ramp surface on an area not to exceed 1 square foot (305 mm^2), and is not required to be assumed to act concurrently with any handrail or guard loadings specified in the preceding paragraphs of Section 1607.7.1. Garages accommodating trucks and buses shall be designed in accordance with an approved method that contains provision for traffic railings.

Reason: The purpose for this proposal is to align the provisions of the IBC on the structural design requirements for handrails, guards, grab bars, accessible seats and vehicle barrier systems with the corresponding provisions in ICC A117.1 and ASCE 7-05, and to update the charging language associated with the provisions.

In Sections 1607.7, “vehicle barriers” is changed to “vehicle barrier systems” for consistency with Section 1607.7.3, the definition for “vehicle barrier system” in Section 1602.1 and Chapter 4 of ASCE 7-05. Similar changes are proposed in Section 406.3.3 and Table 1607.1.

“Accessible seats and accessible benches” are added to the charging language of Section 1607.7 in conjunction with their technical requirements in Section 1607.7.2. The reference to ICC A117.1 is relocated to Section 1607.7.2 where it is more appropriately specified.

In Sections 1607.7.1 and 1607.1.1, “handrail assemblies” is changed to “handrails” for consistency with IBC Sections 1009.10 and 1010.8 on stairway handrails and ramp handrails, respectively. In Section 1607.7.1, “glass handrail assemblies” is retained for consistency with Section 2407.

The revisions to Section 1607.7.2 are proposed to align the provisions of Section 1607 with the applicable provisions of ICC A117.1. The current provisions of ICC A117.1-03 applicable to IBC Section 1607 consist of: (1) Section 609 on grab bars in accessible toilet and bathing facilities, which specifies requirements for structural strength in Section 609.8; (2) Section 610 on seats in accessible bathtubs and shower compartments, which specifies requirements for structural strength in Section 610.4; (3) Section 803 on benches in accessible dressing, fitting and locker rooms, which references Section 903 for structural strength in Section 803.4; (4) Section 806.2 on benches in holding cells and housing cells, which references Section 903 for structural strength in Section 806.2.2; and (5) Section 903 on accessible benches, which references Section 903.6 for structural strength. The provisions of Sections 609.8, 610.4 and 903.6 on structural strength each require the referenced component, its fastener mounting device and its supporting structure be designed to resist the application of a vertical or horizontal force of 250 pounds at any point. As can be seen by these provisions, the applicable provisions of ICC A117.1 are not limited to grab bars, shower seats and dressing room bench seats. They are also not intended to be applicable to grab bars, seats or benches other than those required to be accessible by ICC A117.1.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S67–07/08

Table 1607.1

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

Revise table as follows:

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
30. Roofs		
All roof surfaces subject to maintenance		300
workers		
Awnings and canopies		
Fabric construction supported by a	5	
lightweight rigid skeleton structure	nonreduceable	
All other construction	20	
Ordinary flat, pitched, and curved roofs	20	
Primary roof members, exposed to a		
work floor		
Single panel point of lower chord of		
roof trusses or any point along		
primary structural members		
supporting roofs:		
Over manufacturing, storage		
warehouses, and repair garages		2,000
All other occupancies		300
Roofs used for other special purposes	Note l	Note l
Roofs used for promenade purposes	60	
Roofs used for ~~roof gardens or~~	100	
assembly purposes		
Roofs used for roof gardens	20	

(Portions of table not shown remain unchanged)

Reason: This proposed code change is intended to clarify the intent of the Code as it relates to minimum live load requirements applicable to roof gardens and landscaped roofs (also, commonly referred to as vegetative roofs or greens roofs). Table 1607.1—Minimum Uniformly Distributed Live Loads and Minimum Concentrated Live Loads indicates a minimum live load of 100 psf for roof gardens, while Section 1607.11.2.3—Landscaped Roofs indicates a live load of 20 psf for rooftop landscaped areas.

As a solution to this apparent conflict, this proposal revises Table 1607.1 to allow for a 20 psf live load for roof gardens. This is consistent with Section 1607.11.2.3.

We have also submitted a companion proposal to this proposed code change that, as an alternative, revises Section 1607.11.2.3 to a 100 psf live load, making it consistent with the current Table 1607.1. We ask the code development committee and ICC membership to approve one or the other of these proposals to clarify the Code regarding this apparent conflict.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S68–07/08
Table 1607.1

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

Revise table as follows:

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

a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 1607.1 or the following concentrated loads: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 pounds acting on an area of 4.5 inches by 4.5 inches; (2) for mechanical parking structures without slab or deck which are used for storing passenger vehicles only, 2,250 pounds per wheel.
e. The concentrated wheel load shall be applied on an area of ~~20 square~~ 4.5 inches by 4.5 inches.

(Portions of table and footnotes not shown remain unchanged)

Reason: The revisions are proposed for consistency with the corresponding footnotes in Table 4-1 of ASCE 7-05. Compare Footnote (a) of Table 1607.1 with Footnote (a) of Table 4-1 and Footnote (e) of Table 1607.1 with Footnote (f) of Table 4-1.

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

Public Hearing:				
	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S69–07/08
Table 1607.1

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.

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S70–07/08
Table 1607.1, 1607.11.2.2

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

Revise table as follows:

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
30. Roofs		
All roof surfaces subject to maintenance workers		300
Awnings and canopies		
Fabric construction supported by a lightweight rigid skeleton structure	5 nonreduceable	
All other construction	20	
Ordinary flat, pitched, and curved roofs	20	
Primary roof members, exposed to a work floor		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs~~:~~		
Over manufacturing, storage warehouses, and repair garages		2,000
All other occupancies		300
Roofs used for other special purposes	Note l	Note l
Roofs used for promenade purposes	60	
Roofs used for roof gardens or assembly purposes	100	

(Portions of table and footnotes not shown remain unchanged)

1607.11.2.2 Special-purpose roofs. Roofs used for promenade purposes, roof gardens, assembly purposes or other special purposes shall be designed for a minimum live load as required in Table 1607.1. Such roof live loads are permitted to be reduced in accordance with 1607.9. Live loads of 100 psf (4.79 kN/m^2) at areas of roofs classified as Group A occupancies shall not be reduced.

Reason: Section 1607.9.1.3 prohibits floor live loads of 100 psf or less in public assembly occupancies from being reduced. Section 1607.11.2.2 on special purpose roofs permits the live loads of roofs used for promenade purposes, roof gardens, assembly purposes or other special purposes to be reduced in accordance with the provisions of Section 1607.9 for floor live loads. Table 1607.1 lists a live load of 100 psf for roofs used for roof gardens or assembly purposes. This creates a potential conflict between Sections 1607.9.1.3 and 1607.11.2.2. The purpose for this proposal is to resolve the conflict by prohibiting live loads of 100 psf at areas of roofs classified as Group A occupancies from being reduced.
Specifying roofs classified as Group A occupancies rather than roofs used for public assembly purposes avoids language that is vague and unenforceable in favor of a classification that is defined by the IBC (refer to Section 303). Roofs used for public assembly purposes can 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. Note that Section 303 classifies the use of a "building or structure, or a portion thereof, for the gathering of persons for purposes such as civic, social or religious functions recreation, food or drink consumption, or awaiting transportation, as a Group A occupancy." Occupied roofs are not intended to be excluded from such a classification.

Item #30 of Table 1607.1 is changed for consistency with Table 4-1 of ASCE 7-05.

This proposal was prepared in conjunction with related proposals on reduction of floor live loads, reduction of roof live loads, and marquee live loads.

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

Public Hearing:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF
