Code Technology Committee
2006/2007 Cycle
Area of study - NIST WTC Recommendations
Public comments

The following are code changes for which the CTC has established a position and testified at the 2006 Code Development Hearings. These code changes have received a public comment and will be considered at the 2007 Final Action Hearings. These are assembled for the CTC for determining their position, if any, at the 2007 Final Action Hearings.

S5-06/07
1604.11 (New), 1605 (New)

Proposed Change as Submitted:

Proponent: William M. Connolly, State of New Jersey, Department of Community Affairs, Division of Codes and Standards, representing International Code Council Ad Hoc Committee on Terrorism Resistant Buildings

Add new text as follows:

<u>**1604.11 Disproportionate collapse**. Design for structural integrity to protect against disproportionate collapse shall be in accordance with Section 1605.</u>

<u>**1605 DISPROPORTIONATE COLLAPSE**</u>

<u>**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. Design of new buildings in accordance with Section 1605.5 shall be deemed to comply with Section 1605.4.</u>

<u>**1605.2 DEFINITIONS**.</u>

<u>**DISPROPORTIONATE COLLAPSE.** Local failure of a member of the structural frame that leads to the collapse of the adjoining structural members, which then leads to additional collapse.</u>

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

<u>**KEY ELEMENT.** A structural element capable of sustaining an accidental design loading of 700 psf (34.5 kN/m^2) applied in the horizontal and vertical directions (in one direction at a time) to the member and any attached components (ie. cladding, etc.).</u>

STRUCTURAL FRAME. The columns and the girders, beams, trusses, and spandrels having direct connections to the columns and bracing members designed to carry gravity loads.

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 I-1, R-3 or R-4 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 I-3 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.
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. Group S buildings not exceeding 6 stories.
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 – Groups H-1, H-2, H-3, H-4, and H-5.

1605.4 Performance design approach: Design to protect against disproportionate collapse shall be designed in accordance with accepted engineering practice to meet the requirements of this section or shall be in accordance with Section 1605.5.

1605.4.1 Class 1 buildings (performance). Class 1 buildings are not required to comply with this section.

1605.4.2 Class 2 buildings (performance). Class 2 buildings shall be provided with horizontal ties or with anchorage.

1605.4.2.1 Class 2 structural use of reinforced and unreinforced masonry (performance). Design to protect against disproportionate collapse for unreinforced masonry construction shall be in accordance with Section 1605.4.2.1.1 through Section 1605.4.2.1.5.

1605.4.2.1.1 Class 2 masonry general (performance). 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. 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.

1605.4.2.1.2 Class 2 masonry tie force design requirements (performance). Load-bearing walls shall be tied from the lowest to the highest level.

1605.4.2.1.3 Class 2 masonry Internal ties (performance). Internal ties shall be anchored to peripheral ties at each end, or must continue as wall or column ties.

1605.4.2.1.4 Class 2 masonry peripheral ties (performance). Peripheral ties shall be provided at the edge of a floor or roof or in the perimeter wall and anchor at re-entrant corners or changes of construction.

1605.4.2.1.5 Class 2 masonry horizontal ties to external columns and walls (performance). Each external column and external load-bearing wall shall be anchored or tied horizontally into the structure at each floor and roof level.

1605.4.2.2 Class 2 structural use of steel (performance). *Design against disproportionate collapse for structural steel shall be in accordance with Section 1605.4.2.2.1 through Section 1605.4.2.2.2.*

1605.4.2.2.1 Class 2 steel general (performance). 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 (peripheral and external column) contained in Section 1605.4.2.2.2.

1605.4.2.2.2 Class 2 steel tie force requirements (performance). All buildings shall be tied together at each principal floor level. Each column shall be held in position by means of horizontal ties in two directions at each principal floor level supported by that column. Continuous lines of ties shall be provided at the edges of the floor or roof and to each column line.

1605.4.2.3 Class 2 structural use of plain, reinforced and prestressed concrete (performance). Design to protect against disproportionate collapse for concrete shall be in accordance with ACI 318. 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. 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 (peripheral and external column).

1605.4.3 Class 3 buildings (performance). Class 3 buildings shall be provided with horizontal ties, anchorage, and vertical ties or shall be designed utilizing alternate load path analysis.

1605.4.3.1 Class 3 structural use of reinforced and unreinforced masonry (performance). Design to protect against disproportionate collapse for unreinforced masonry construction shall be in accordance with Section 1605.4.3.1.1 through Section 1605.4.3.1.7.

1605.4.3.1.1 Class 3 masonry general (performance). 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.

1605.4.3.1.2 Class 3 masonry tie force design requirements (performance). Load-bearing walls shall be tied from the lowest to the highest level.

1605.4.3.1.3 Class 3 masonry internal ties (performance). Internal ties shall be anchored to peripheral ties at each end, or must continue as wall or column ties.

1605.4.3.1.4 Class 3 masonry peripheral ties (performance). Peripheral ties shall be provided at the edge of a floor or roof or in the perimeter wall and anchor at re-entrant corners or changes of construction.

1605.4.3.1.5 Class 3 masonry horizontal ties to external columns and walls (performance). Each external column and external load-bearing wall shall be anchored or tied horizontally into the structure at each floor and roof level.

1605.4.3.1.6 Class 3 masonry vertical ties (performance). Columns and load-bearing walls shall have vertical ties. Vertical ties shall extend from the roof level to the foundation. Vertical ties 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.

1605.4.3.1.6.1 Class 3 masonry load-bearing walls and columns with deficient vertical tie forces (performance). Load-bearing elements that do not comply with the required vertical tie strength, shall be designed in accordance with the alternate load path method.

1605.4.3.1.7 Class 3 masonry alternate load path method design requirements (performance). *Alternate load path method is used to verify that the structure can bridge over removed elements.*

1605.4.3.1.7.1 Class 3 masonry key element analysis (performance). When applying the alternate load path method design requirements and the removal of columns and lengths of walls results in a disproportionate collapse, then such elements shall be designed as a key element.

1605.4.3.2 Class 3 structural use of steel (performance). Design against disproportionate collapse for structural steel shall be in accordance with Section 1605.4.3.2.1 through Section 1605.3.2.3.

1605.4.3.2.1 Class 3 steel general (performance). 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 the internal tie requirements of ACI 318, while the steel frame shall comply the other tie requirements (vertical, peripheral, and external column) and the alternate load path requirements of this section.

1605.4.3.2.2 Class 3 steel tie force requirements (performance). 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 at each principal floor level supported by that column. Continuous lines of ties shall be provided at the edges of the floor or roof and to each column line.

1605.4.3.2.2.1 Class 3 steel vertical ties (performance). All columns shall be continuous through each beam-to-column connection.

1605.4.3.2.2.2 Class 3 steel columns with deficient vertical tie forces (performance). The alternate load path method shall be used in each deficient column, where it is not possible to provide the vertical required tie strength.

1605.4.3.2.3 Class 3 steel alternate load path method design requirements (performance). Alternate load path method is used to verify that the structure can bridge over removed elements.

1605.4.3.2.3.1 Class 3 steel key element analysis (performance). When applying the alternate load path method design requirements and the removal of columns and lengths of walls results in a disproportionate collapse, then such elements shall be designed as a key element.

1605.4.3.3 Class 3 concrete structural use of plain, reinforced and prestressed concrete (performance). Design to protect against disproportionate collapse for concrete shall be in accordance with ACI 318. 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. 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 the other tie requirements (vertical, peripheral, and external column).

1605.4.3.3.1 Class 3 concrete alternate load path method design requirements (performance). 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, then compliance shall be in accordance with the alternate load path model subsection.

1605.4.3.3.1.1 Class 3 concrete key element analysis (performance). When applying the alternate load path method design requirements and the removal of columns and lengths of walls results in a disproportionate collapse, then such elements shall be designed as a key element.

1605.4.4 Class 4 buildings (performance). Class 4 buildings shall comply with the requirements for Class 3 buildings and a systematic risk assessment of the building shall be undertaken taking into account all the normal hazards that may be reasonably foreseen, together with any abnormal hazard. A peer review shall be submitted with the risk assessment. Critical situations for design shall be selected that reflect the conditions that can reasonably be foreseen as possible during the life of the building.

1605.5 Prescriptive design approach. Design of new buildings to protect against disproportionate collapse shall be in accordance with this section or shall be in accordance with an approved engineering method in accordance with Section 1605.4.

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

1605.5.2 Class 2 buildings (prescriptive). 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 (prescriptive). 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 (prescriptive). 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 (prescriptive). 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 (prescriptive). 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 (prescriptive). 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 (prescriptive). 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 (prescriptive). 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 (prescriptive). For the purpose of applying the alternate load path analysis, collapse shall be deemed disproportionate 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 15% of the area of that story or 750 square feet whichever is smallest, or extends furthers than the immediate adjacent story.

1605.5.3.4.2 Class 3 key element analysis (prescriptive). 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 (prescriptive). 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 taking into account all the normal hazards that may be reasonably foreseen, together with any abnormal hazard. Critical situations for design shall be selected that reflect the conditions that can reasonably be foreseen as possible during the life of the building.

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 (prescriptive). 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 (prescriptive). 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 (prescriptive). 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. Splices are not to be located near connections or mid-span. Tie reinforcing bars that provide tie forces at right angle to other reinforcing bars shall used 135 degree hooks with six-diameter, but not less than 3 inches, extension. Use the strength reduction factors φ 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 (prescriptive). 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 (prescriptive). For two-way spans the internal ties shall be design to resist a required tie strengths equal to the greater of:

1. $(1.0D + 1.0L)L_aF_t/(8475)$ (Kips/ft)

or

2. $1.0F_t/3.3$ (Kips/ft)

Where:

D = Dead load (psf)
L = Live load (psf)
L_a = 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).
F_t = "Basic Strength" = Lesser of $4.5 + 0.9\ N_s$) or 13.5.
N_s = Number of stories including basement(s)

1605.6.1.3.2 Masonry one-way spans (prescriptive). 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 F_t.

1605.6.1.4 Masonry peripheral ties (prescriptive). Peripheral ties shall have a required tie strength of $1.0F_t$. 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 (prescriptive). 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 a design tie strength equal to:

$2.0F_t$ or $(h/8.2)F_t$, whichever is smaller (kips)

Where:

H	=	Clear story height (ft)
F_t	=	"Basic Strength" = Lesser of $(4.5 + 0.9N_s)$ or 13.5
N_s	=	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 (prescriptive). 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 (prescriptive). 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 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 F_t (kips) per 3.33 feet width.

1605.6.1.6.2 Masonry columns (prescriptive). 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:

$6.2 \times 10^{-4} A(h_a/t)^2$ 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).
h_a	=	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 (prescriptive). Load-bearing 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 F_t in $0.45F_t$ 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 $0.45F_t$ 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 $0.15F_t$ 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 (prescriptive). 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 (prescriptive). 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 1605.6.1.8.3.

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 φ

factors are defined per Chapter 3 of ACI 530.

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

1605.6.1.8.2 Masonry linear static analysis (prescriptive). 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 (prescriptive). 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 (prescriptive). The structural element shall be removed when the required moment exceeds the flexural design strength and shall redistributed 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 (prescriptive). The axial design strength with the applicable strength reduction factor φ 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 φ 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 (prescriptive). The connections design strength with the applicable strength reduction factor φ 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 (prescriptive). 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 90% of F_y for reinforcement.

1605.6.1.8.9 Masonry loads associated with failed elements (prescriptive). 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 (prescriptive). 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 (prescriptive). 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 (prescriptive). 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.

$$2.0[(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S)] + 0.2W$$

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 (prescriptive). 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 (prescriptive). 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 (prescriptive). 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 (prescriptive). 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_v**
Hot-Rolled Structural Shapes and Bars	1.05	
ASTM A36/A36M	1.05	1.5
ASTM A573/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 (prescriptive). 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 (prescriptive). 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 (prescriptive). 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 (prescriptive). 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 (prescriptive). 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. Locate splices away from joints or regions of high stress and shall be staggered.

1605.6.2.3.2.1.2 Hooks (prescriptive). 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 (prescriptive). 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:

$0.5(1.2D + 1.6L)s_t L_l$ but not less than 16.9 kips

Where:

D = Dead load (psf)
L = Live load (psf)
L_l = Span (ft.)

s_t = Mean transverse spacing of the ties adjacent to the ties being checked (ft.)

1605.6.2.3.4 Steel peripheral ties (prescriptive). Peripheral ties shall be capable of resisting the following load:

$0.25(1.2D + 1.6L)s_tL_l$ but not less than 8.4 kips

Where:

D = Dead load (psf)
L = Live load (psf)
L_l = Span (ft.)
S_t = Mean transverse spacing of the ties adjacent to the ties being checked (ft.)

1605.6.2.3.5 Steel tying of external columns (prescriptive). 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 1% 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 (prescriptive). 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 (prescriptive). 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 (prescriptive). 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 Ω_y and Ω_u depending upon the limit state; all Φ factors are defined per AISC 360.

1605.6.2.4.1 Steel flexural resistance of structural steel (prescriptive). 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, M_n, 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 (prescriptive). 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, M_p, 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 (prescriptive). 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 (prescriptive). 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 (prescriptive). 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 bending resistance of structural steel (prescriptive). The response of an element under combined axial force and bending 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 (P_u/P_y where $P_y = A_g F_y$) 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 (prescriptive). The acceptability criteria for shear of structural steel is 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 (prescriptive). 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 (prescriptive)**.** 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 (prescriptive). 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 (prescriptive). 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 (prescriptive). 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 (prescriptive). 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.

$$2.0[(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S)] + 0.2W$$

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

Component	CLASS 2 AND 3 BUILDINGS Ductility μ	CLASS 2 AND 3 BUILDINGS Rotation, Degrees θ	CLASS 4 BUILDINGS Ductility μ	CLASS 4 BUILDINGS 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 (prescriptive). 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 the internal tie requirements of ACI 318, while the steel frame shall comply the other tie requirements (vertical, peripheral, and external column).

1605.6.3.1 Concrete alternate load path method design requirements (prescriptive). 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 subsection.

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 Ω_y and Ω_u 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 (prescriptive). 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 (prescriptive). 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 φ 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 (prescriptive). 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 (prescriptive). 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 (prescriptive). 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 (prescriptive). 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 P_b, 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 P_b, 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 (prescriptive). 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 (prescriptive). The connections design strength with the applicable strength reduction factor φ 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 (prescriptive). When the element or the connections at each end of an element exceed the a 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 Membrame[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 Membrame[a]				
Normal Proportions ($L/h \geq 5$)	-	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 (prescriptive). The following procedure shall be met for Nonlinear Dynamic, and Linear or Nonlinear Static Analysis.

1605.6.3.2.1 Concrete nonlinear dynamic (prescriptive). 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 (prescriptive). 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 (prescriptive). 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.

$$2.0[(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S)] + 0.2W$$

Where:

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

1605.6.4 Key elements analysis (prescriptive). 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 (prescriptive). The following load combinations shall be used in addition to the accidental design loading in the key element analysis:

$$1.2D + A_k + (0.5L \text{ or } 0.2S)$$

$$(0.9 \text{ or } 1.2)D + A_k + 0.2W$$

As per the definition of key element, A_k = 700 psf.

Reason: This code change proposal is one of fourteen proposals being submitted by the International Code Council Ad Hoc Committee on Terrorism Resistant Buildings.

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.

This is the sort of issue that one might expect to be addressed through engineering design standards such as ASCE-7 and others. It is not and there is not, at this writing, any firm plan or timetable to do so. It is the proponents' belief that the time is long past for such a dramatic gap in the public safety requirement for buildings to exist. 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. It is the proponents' hope that the nation's engineering community will take up, soon and with urgency, the challenge of preparing detailed technical standards that will be suitable for reference in future editions of the Code.

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 approach has 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 1605.4 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 1605.3

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 <u>or</u> 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 the Class 2 buildings of masonry (1605.4.2.1), steel (1605.4.2.2), and concrete (1605.4.2.3). Similarly, the requirements for Class 3 buildings and set forth for masonry (1605.4.3.1), steel (1605.4.3.2), and concrete (1605.4.3.3).

1605.5 sets forth the prescription "deemed to comply" design approach. Like Section 1605.4, the requirements for each class of building are set forth separately, for ease of use, and within each class the approach that can be used for masonry, steel, and concrete are each set out in their own subsection. It is here that ACI 318-02 is referenced for the concrete tie force approach.

Bibliography:

<u>American Concrete Institute (ACI) Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05).</u> American Concrete Institute: Farmington Hills, Michigan, 2005.

American Society of Civil Engineers (ASCE) 7-2005, <u>Minimum Design Loads for Buildings and Other Structures.</u> American Society of Civil of Engineers: Danvers, Massachusetts, 2005.

General Services Administration. <u>Facilities Standards for the Public Building Service, Chapter 8, Security Design.</u> Washington,

DC. November 2000.

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

United Kingdom, Office of the Deputy Prime Minister. Building Regulations, 2004 (Structure); Approved Document A. United Kingdom: London, 2004.

United Kingdom, Guidance on Robustness and Provision Against Accidental Actions." Allot and Lomax Proposal on the Current Application of Requirements A3 of the Building Regulations 1991. United Kingdom: London.

United Kingdom, Proposed Revised Guidance on Meeting Compliance with the Requirements of Building Regulation, A3: Revision of Allot and Lomax Proposal. United Kingdom: London. 2000.

United States Department of Defense. Unified Facilities Criteria, Design of Buildings to Resist Progressive Collapse, UFC 4-023-03. Washington, DC., January 25, 2005.

Cost Impact: The proponents believe that actual construction costs will be increased little, if at all. This belief is based on 30 years of British experience. There will be increased design analysis and detailing costs, but those will be modest when viewed as a percentage of total construction costs.

Committee Action: Disapproved

Committee Reason: There are concerns that the application of the proposed disproportionate collapse provisions would result in unintended consequences and that these provisions are arbitrary and unenforceable. The proposal would inappropriately place material requirements in Chapter 16. It is unclear whether a minor addition would trigger compliance for the entire structure. Terms such as "abnormal hazard" and "masonry cross-wall construction" are not clear. The definition of structural frame differs from Table 601 requirements. It is unclear if this difference is intentional or an oversight. The definition of key element contains a requirement for a 700 psf accidental design loading. The correct application of this load to the structure is not apparent. There are potential conflicts in the building class for Group R-3 occupancies. Also any residential building that can't comply with the IRC would need to comply with these provisions. It is unclear how the requirements that apply to specific types of construction would be applied to typical buildings that consist of combinations of various construction types.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

William M. Connolly, State of New Jersey, Department of Community Affairs, Division of Codes and Standards, representing International Code Council Ad Hoc Committee on Terrorism Resistant Buildings, requests Approval as Modified by this public comment.

Modify proposal as follows:

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

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. ~~Design of new buildings in accordance with Section 1605.5 shall be deemed to comply with Section 1605.4.~~

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. ~~Local failure of a member of the structural frame that leads to the collapse of the adjoining structural members, which then leads to additional 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 ~~capable of sustaining an accidental design loading of 700 psf (34.5 kN/m2) applied in the horizontal and vertical directions (in one direction at a time) to the member and any attached components (ie. cladding, etc.)~~ 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. ~~Group S buildings not exceeding 6 stories.~~
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 ~~Performance d~~ Design approach: Design to protect against disproportionate collapse ~~shall be designed in accordance with accepted engineering practice to meet the requirements of this section or~~ 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.

~~**1605.4.1 Class 1 buildings (performance).** Class 1 buildings are not required to comply with this section.~~

~~**1605.4.2 Class 2 buildings (performance).** Class 2 buildings shall be provided with horizontal ties or with anchorage.~~

~~**1605.4.2.1 Class 2 structural use of reinforced and unreinforced masonry (performance).** Design to protect against disproportionate collapse for unreinforced masonry construction shall be in accordance with Section 1605.4.2.1.1 through Section 1605.4.2.1.5.~~

~~**1605.4.2.1.1 Class 2 masonry general (performance).** 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. 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.~~

~~**1605.4.2.1.2 Class 2 masonry tie force design requirements (performance).** Load-bearing walls shall be tied from the lowest to the highest level.~~

~~**1605.4.2.1.3 Class 2 masonry Internal ties (performance).** Internal ties shall be anchored to peripheral ties at each end, or must continue as wall or column ties.~~

~~**1605.4.2.1.4 Class 2 masonry peripheral ties (performance).** Peripheral ties shall be provided at the edge of a floor or roof or in the perimeter wall and anchor at re-entrant corners or changes of construction.~~

~~**1605.4.2.1.5 Class 2 masonry horizontal ties to external columns and walls (performance).** Each external column and external load-bearing wall shall be anchored or tied horizontally into the structure at each floor and roof level.~~

~~**1605.4.2.2 Class 2 structural use of steel (performance).** Design against disproportionate collapse for structural steel shall be in accordance with Section 1605.4.2.2.1 through Section 1605.4.2.2.2.~~

~~**1605.4.2.2.1 Class 2 steel general (performance).** 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 (peripheral and external column) contained in Section 1605.4.2.2.2.~~

~~**1605.4.2.2.2 Class 2 steel tie force requirements (performance).** All buildings shall be tied together at each principal floor level. Each column shall be held in position by means of horizontal ties in two directions at each principal floor level supported by that column. Continuous lines of ties shall be provided at the edges of the floor or roof and to each column line.~~

~~**1605.4.2.3 Class 2 structural use of plain, reinforced and prestressed concrete (performance).** Design to protect against disproportionate collapse for concrete shall be in accordance with ACI 318. 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. 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 (peripheral and external column).~~

~~**1605.4.3 Class 3 buildings (performance)**. Class 3 buildings shall be provided with horizontal ties, anchorage, and vertical ties or shall be designed utilizing alternate load path analysis.~~

~~**1605.4.3.1 Class 3 structural use of reinforced and unreinforced masonry (performance).** Design to protect against disproportionate collapse for unreinforced masonry construction shall be in accordance with Section 1605.4.3.1.1 through Section 1605.4.3.1.7.~~

~~**1605.4.3.1.1 Class 3 masonry general (performance).** 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.~~

~~**1605.4.3.1.2 Class 3 masonry tie force design requirements (performance).** Load-bearing walls shall be tied from the lowest to the highest level.~~

~~**1605.4.3.1.3 Class 3 masonry internal ties (performance).** Internal ties shall be anchored to peripheral ties at each end, or must continue as wall or column ties.~~

~~**1605.4.3.1.4 Class 3 masonry peripheral ties (performance).** Peripheral ties shall be provided at the edge of a floor or roof or in the perimeter wall and anchor at re-entrant corners or changes of construction.~~

~~**1605.4.3.1.5 Class 3 masonry horizontal ties to external columns and walls (performance).** Each external column and external load-bearing wall shall be anchored or tied horizontally into the structure at each floor and roof level.~~

~~**1605.4.3.1.6 Class 3 masonry vertical ties (performance).** Columns and load-bearing walls shall have vertical ties. Vertical ties shall extend from the roof level to the foundation. Vertical ties 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.~~

~~**1605.4.3.1.6.1 Class 3 masonry load-bearing walls and columns with deficient vertical tie forces (performance).** Load-bearing elements that do not comply with the required vertical tie strength, shall be designed in accordance with the alternate load path method.~~

~~**1605.4.3.1.7 Class 3 masonry alternate load path method design requirements (performance).** Alternate load path method is used to verify that the structure can bridge over removed elements.~~

~~**1605.4.3.1.7.1 Class 3 masonry key element analysis (performance).** When applying the alternate load path method design requirements and the removal of columns and lengths of walls results in a disproportionate collapse, then such elements shall be designed as a key element.~~

~~**1605.4.3.2 Class 3 structural use of steel (performance).** Design against disproportionate collapse for structural steel shall be in accordance with Section 1605.4.3.2.1 through Section 1605.3.2.3.~~

~~**1605.4.3.2.1 Class 3 steel general (performance).** 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 the internal tie requirements of ACI 318, while the steel frame shall comply the other tie requirements (vertical, peripheral, and external column) and the alternate load path requirements of this section.~~

~~**1605.4.3.2.2 Class 3 steel tie force requirements (performance).** 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 at each principal floor level supported by that column. Continuous lines of ties shall be provided at the edges of the floor or roof and to each column line.~~

~~**1605.4.3.2.2.1 Class 3 steel vertical ties (performance)**. All columns shall be continuous through each beam-tocolumn connection.~~

~~**1605.4.3.2.2.2 Class 3 steel columns with deficient vertical tie forces (performance)**. The alternate load path method shall be used in each deficient column, where it is not possible to provide the vertical required tie strength.~~

~~**1605.4.3.2.3 Class 3 steel alternate load path method design requirements (performance).** Alternate load path method is used to verify that the structure can bridge over removed elements.~~

~~**1605.4.3.2.3.1 Class 3 steel key element analysis (performance).** When applying the alternate load path method design requirements and the removal of columns and lengths of walls results in a disproportionate collapse, then such elements shall be designed as a key element.~~

~~**1605.4.3.3 Class 3 concrete structural use of plain, reinforced and prestressed concrete (performance).** Design to protect against disproportionate collapse for concrete shall be in accordance with ACI 318. 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. 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 the other tie requirements (vertical, peripheral, and external column).~~

~~**1605.4.3.3.1 Class 3 concrete alternate load path method design requirements (performance).** 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, then compliance shall be in accordance with the alternate load path model subsection.~~

~~**1605.4.3.3.1.1 Class 3 concrete key element analysis (performance).** When applying the alternate load path method design requirements and the removal of columns and lengths of walls results in a disproportionate collapse, then such elements shall be designed as a key element.~~

~~**1605.4.4 Class 4 buildings (performance).** Class 4 buildings shall comply with the requirements for Class 3 buildings and a systematic risk assessment of the building shall be undertaken taking into account all the normal hazards that may be reasonably foreseen, together with any abnormal hazard. A peer review shall be submitted with the risk assessment. Critical situations for design shall be selected that reflect the conditions that can reasonably be foreseen as possible during the life of the building.~~

1605.5 Prescriptive design approach. Design of new buildings to protect against disproportionate collapse shall be in accordance with <u>the requirements specified below for each building class.</u> ~~this section or shall be in accordance with an approved engineering method in accordance with Section 1605.4.~~

1605.5.1 Class 1 buildings ~~(prescriptive)~~. Class 1 buildings are not required to comply with this section.

1605.5.2 Class 2 buildings ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive~~). 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 15% of the area of that story or 750 square feet whichever is ~~smallest~~ <u>smaller</u>, or extends furthers than the immediate adjacent story.

1605.5.3.4.2 Class 3 key element analysis ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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<u>.</u> ~~taking into account all the normal hazards that may be reasonably foreseen, together with any abnormal hazard.~~ Critical situations ~~for design shall be selected that reflect the conditions that can reasonably be foreseen as possible during the life of the building~~ <u>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</u>.

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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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~~es~~, extension. Use the strength reduction factors φ 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 a 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ar the alternate load path Section ~~1605.6.1.8.3~~ 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 φ factors are defined per Chapter 3 of ACI 530.

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

1605.6.1.8.2 Masonry linear static analysis ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. The structural element shall be removed when the required moment exceeds the flexural design strength and shall redistributed 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 ~~(prescriptive)~~. The axial design strength with the applicable strength reduction factor φ 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 φ 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 ~~(prescriptive)~~. The connections design strength with the applicable strength reduction factor φ 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 ~~(prescriptive)~~. 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 90% of Fy for reinforcement.

1605.6.1.8.9 Masonry loads associated with failed elements ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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_v
Hot-Rolled Structural Shapes and Bars	1.05	
ASTM A36/A36M	1.05	1.5
ASTM ~~A573~~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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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. ~~Locate splices away from joints or regions of high stress and shall be staggered~~ Splices shall be located away from joints or regions of high stress and shall be staggered.

1605.6.2.3.2.1.2 Hooks ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 1% 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~bending~~ <u>flexural</u> resistance of structural steel ~~(prescriptive)~~. The response of an element under combined axial force and ~~bending~~ <u>flexural</u> 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 ~~(prescriptive)~~. The acceptability criteria for shear of structural steel is 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 the internal tie requirements of ACI 318, while the steel frame shall comply the other tie requirements (vertical, peripheral, and external column).

1605.6.3.1 Concrete alternate load path method design requirements ~~(prescriptive)~~. 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 ~~subsection~~ 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 φ 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 (prescriptive). 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. The connections design strength with the applicable strength reduction factor φ 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 ~~(prescriptive)~~. 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 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,**<u>DDegrees</u> θ	**Ductility** υ	**Rotation,**<u>Degrees</u> 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 ($L/h \geq 5$)	-	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 ~~(prescriptive)~~. The following procedure shall be met for Nonlinear Dynamic, and Linear or Nonlinear Static Analysis.

1605.6.3.2.1 Concrete nonlinear dynamic ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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 ~~(prescriptive)~~. 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.

$2.0[(0.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S)] + 0.2W$

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

1605.6.4 Key elements analysis ~~(prescriptive)~~. 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 ~~(prescriptive)~~. The following load combinations shall be used in addition to the accidental design loading in the key element analysis:

$1.2D + Ak + (0.5L \text{ or } 0.2S)$
$(0.9 \text{ or } 1.2)D + Ak + 0.2W$

As per the definition of key element, Ak = 700 psf.

Commenter's Reason: This public comment is submitted in response to some of the comments made at the committee hearings held in Orlando in September, 2006. One of the chief, legitimate criticisms of this proposed code change was that the requirements of 1605.4 were unenforceable. This public comment replaces Section 1605.4 with a statement that designs must comply with the prescriptive requirements of Section 1605.5 or with a design alternative. The new design alternative language uses the prescriptive requirements of 1605.5 as a yardstick for measuring whether a particular design is adequate or not. Based on this public comment, the design professional would have to document that the design provides structural robustness that is at least equivalent to that prescribed through the specific requirements of Section 1605.5. The word "prescriptive" has been deleted from headings of the requirements in 1605.5 and 1605.6. Since the performance requirements contained in 1605.4 have been deleted, this distinction is no longer necessary.

The language of Section 1605.5.4 also was questioned by those commenting at the code change hearings. Specifically, the commenters pointed out that a requirement to design for "normal hazards that may be reasonably foreseen, together with any abnormal hazard" is not an enforceable standard. Accordingly, Section 1605.5.4 is modified by this public comment to state that class 4 buildings, those in the highest hazard category, are to be designed for the specific hazards identified in a risk assessment and that a peer review of the risk assessment and of the design is also required.

In Section 1605.2, the definitions of "disproportionate collapse," "key element" and "structural frame" have been modified and definitions of "alternate load path method" and "ties" have been added for clarity.

In Table 1605.3, buildings of Group S not exceeding six stories in height have been moved from class 3 to class 2 as this classification better reflects the risks and need for additional structural protections for such buildings.

This public comment includes grammatical corrections to Sections 1605.5.3.4.1, 1605.6.1.6.1 and 1605.6.2.3.2.1.1. Also, a correction is made to the number of a referenced ASTM standard in Table 1605.6.2.2. A reference to ACI 318 has been added to Section 1605.6.1.2 and minor corrections have been made to this section. Cross references have been corrected in Section 1605.6.1.8 and added in Section 1605.6.3.1.

Final Action: AS AM AMPC____ D

S16-06/07
1609.1.1

Proposed Change as Submitted:

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

1. Revise as follows:

1609.1.1 Determination of wind loads: Wind loads on every building or structure shall be determined in accordance with Chapter 6 of ASCE 7. The type of opening protection required, the basic wind speed and the exposure category for a site is permitted to be determined in accordance with Section 1609 or ASCE 7. Wind shall be assumed to come from any horizontal direction and wind pressures shall be assumed to act normal to the surface considered.

Exceptions:

1. Subject to the limitations of Section 1609.1.1.1, the provisions of SBCCI SSTD 10 Standard for Hurricane Resistant Residential Construction shall be permitted for applicable Group R-2 and R-3 buildings.
2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of the AF&PA WFCM.
3. Designs using NAAMM FP 1001.
4. Designs using TIA/EIA-222 for antenna-supporting structures and antennas.
5. <u>Designs using wind tunnel testing in accordance with Section 1609.1.1.2</u>

1609.1.1.1 Applicability. The provisions of SSTD 10 are applicable only to buildings located within Exposure B or C as defined in Section 1609.4. The provisions of SSTD 10 and the AF&PA Wood Frame construction Manual for One- and Two-Family Dwellings shall not apply to buildings sited on the upper half of an isolated hill, ridge or escarpment meeting the following conditions:

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

<u>**1609.1.1.2 Wind tunnel testing.** Where wind tunnel testing is used to determine design wind loads, such testing shall be in accordance with ASCE xx.</u>

2. Add standard to Chapter 35 as follows:

ASCE xx-yy Wind Tunnel Testing

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

This proposed change is a result of the CTC's investigation of the area of study entitled "Review of NIST WTC Recommendations". The scope of the activity is noted as:

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

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

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

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

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

As of the submission of this proposal, it is CTC's understanding that the wind tunnel test standard is not complete but is under development.

Bibliography:

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

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

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

Analysis: Results of review of the proposed standard(s) will be posted on the ICC website by August 20, 2006.

Note: The following analysis was not in the Code Change Proposal book but was published in the "Errata to the 2006/2007 Proposed Changes to the International Codes and Analysis of Proposed Referenced Standards" provided at the code development hearings:

Analysis: Review of proposed new standard indicated that, in the opinion of ICC staff, the standard did not comply with ICC criteria for referenced standards, Section 3.6.3-1 readily available.

Committee Action: Disapproved

Committee Reason: This proposal was disapproved because it would incorrectly reference the wind tunnel test standard as an exception to the ASCE 7 wind load requirements. Doing so would allow designers to circumvent other applicable wind load requirements by performing a wind tunnel test. In addition the proposed standard is not in compliance with the ICC code development process since it is not yet readily available.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Paul Heilstedt, PE, Chair, Code Technology Committee (CTC)

Gerry Jones/Herman Brice, Co-Chairs, NIBS/MMC Committee for Translating the NIST World Trade Center Investigation Recommendations into Building Codes

Modify proposal as follows:

1609.1.1.2 Wind tunnel testing. Where wind tunnel testing is used to determine design wind loads, such testing shall be in accordance with ASCE/SEI 49. The minimum design wind load shall not be less than the minimum prescribed by ASCE 7. The lower limit on pressures for main wind-force resisting systems and components and cladding shall be in accordance with Sections 1609.1.1.2.1 and 1609.1.1.2.2.

1609.1.1.2.1 Lower limits on main wind-force-resisting system. Base overturning moments determined from wind tunnel testing shall be limited to not less than 80 percent of the design base overturning moment determined in accordance with Section 6.5 of ASCE 7, unless specific testing is performed that demonstrates it is the aerodynamic coefficient of the building, rather than shielding from other structures, that is responsible for the lower values. The 80 percent limit may be adjusted by the ratio of the frame load at critical wind directions as determined from wind tunnel testing without specific adjacent buildings, but including appropriate upwind roughness, to that determined in Section 6.5 of ASCE 7.

1609.1.1.2.2 Lower limits on components and cladding. The design pressures for components and cladding on walls or roofs shall be selected as the greater of the wind tunnel test results or 80 percent or the pressure obtained for Zone 4 for walls and Zone 1 for roofs as determined in Section 6.5 of ASCE 7, unless specific testing is performed that demonstrates it is the aerodynamic coefficient of the building, rather than shielding from nearby structures, that is responsible for the lower values. Alternatively, limited tests at a few wind directions without specific adjacent buildings, but in the presence of an appropriate upwind roughness, shall be permitted to be used to demonstrate that the lower pressures are due to the shape of the building and not to shielding.

Chapter 35:

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

(Portions of the proposal not shown remain unchanged)

Commenter's Reason: The committee correctly noted that ASCE 7 prescribes a minimum design load. This load is 10 psf in accordance with Section 6.1.4.1 of ASCE 7. In code change S17-06/07, the proponent of the change provides the criteria to be used in conjunction with wind tunnel testing. This public comment is merely correlating the text of S16, including the proposed new referenced standard, with the text that was approved as modified in S17 by the IBC Structural committee.

As to availability of the standard, ASCE has indicated that the public comment phase of the standard development has been completed and the standard will be published and available prior to the Final Action Hearing. If the standard is not available, this public comment will be withdrawn.

If this public comment is successful and the action taken on S17 stands, the correlated text between the two code changes should include the text of exception 5 from S16 as this text refers to the new referenced standard.

Final Action: AS AM AMPC____ D

S39-06/07
1704.10

Proposed Change as Submitted:

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

Revise as follows:

1704.10 Sprayed fire-resistant materials. Special inspections for sprayed fire-resistant materials applied to structural elements and decks shall be in accordance with Sections 1704.10.1 through 1704.10.~~5~~ 6 Special inspections shall be based on the fire-resistance design as designated in the approved construction documents. The tests described in this section shall be based on samplings of specific floor, roof and wall assemblies, and structural framing members. Special inspections shall be performed after the rough installation of electrical, sprinkler, mechanical and plumbing systems and suspension for ceiling systems, where applicable.

1704.10.1 Physical and visual tests. The following physical and visual tests are required to demonstrate compliance with the listing and the fire-resistance rating:

1. Condition of substrates.
2. Thickness of application.
3. Density in pounds per cubic foot (kgs per m^3).
4. Bond strength -adhesion/cohesion.
5. Condition of finished application.

~~**1704.10.1**~~ **1704.10.2 Structural member surface conditions.** The surfaces shall be prepared in accordance with the approved fire-resistance design and the approved manufacturer's written instructions. The prepared surface of structural members to be sprayed shall be inspected before the application of the sprayed fire-resistant material.

~~**1704.10.2**~~ **1704.10.3 Application**. The substrate shall have a minimum ambient temperature before and after application as specified in the approved manufacturer's written instructions. The area for application shall be ventilated during and after application as required by the approved manufacturer's written instructions.

~~**1704.10.3**~~ **1704.10.4 Thickness**. The average thickness minus two times the standard deviation of the thickness measurements of the sprayed fire-resistant materials applied to structural elements shall not be less than the thickness required by the approved fire-resistant design. Individual measured thickness, which exceeds the thickness specified in a design by 1/4 inch (6.4 mm) or more, shall be recorded as the thickness specified in the design plus 1/4 inch (6.4 mm). For design thicknesses 1 inch (25 mm) or greater, the minimum allowable individual thickness shall be the design thickness minus 1/4 inch (6.4 mm). For design thicknesses less than 1 inch (25 mm), the minimum allowable individual thickness shall be the design thickness minus 25 percent. Thickness shall be determined in accordance with ASTM E 605. Samples of the sprayed fire-resistant materials shall be selected in accordance with Sections ~~1704.10.3~~ 1704.10.4.1 and ~~1704.10.3.2~~ 1704.10.4.2.

~~**1704.10.3.1**~~ **1704.10.4.1 Floor, roof and wall assemblies**. The thickness of the sprayed fire-resistant material applied to floor, roof and wall assemblies shall be determined in accordance with ASTM E 605, taking the average minus two times the standard deviation of the thickness measurements of not less than four measurements for each 1,000 square feet ($93m^2$) of the sprayed area on each floor or part thereof.

1704.10.4.1.1 Flat decks. Thickness measurements shall be taken from a 12 inches (305 mm) square with a minimum of four measurements, symmetrically.

1704.10.4.1.2 Fluted decks. Thickness measurements shall be taken from a 12 inches (305 mm) square with four random, symmetrical measurements within the square, including one each of the following: valley, crest and sides and report as an average.

~~**1704.10.3.2**~~ **1704.10.4.2 Structural framing members.** The thickness of the sprayed fire-resistant material applied to structural members shall be determined in accordance with ASTM E 605. Thickness testing shall be performed on not less than 25 percent of the structural members on each floor.

1704.10.4.2.1 Beams. Thickness measurements shall be made at nine locations around the beam at each end of a 12 inches. (305 mm) length.

1704.10.4.2.2 Joists and trusses. Thickness measurements shall be made at seven locations around the joist or truss at each end of a 12 inches (305 mm) length.

1704.10.4.2.3 W-shape columns. Thickness measurements shall be made at 12 locations around the column at each end of a 12inches (305 mm) length.

1704.10.4.2.4 Tube and pipe columns. Thickness measurements shall be made at a minimum of four locations around the column at each end of a 12 inches (305 mm) length.

~~**1704.10.4**~~ **1704.10.5 Density**. The density of the sprayed fire-resistant material shall not be less than the density specified in the approved fire-resistant design. Density of the sprayed fire-resistant material shall be determined in accordance with ASTM E 605. The test samples for determining the density of the sprayed fire-resistant materials shall be selected as follows:

1. From each floor, roof and wall assembly at the rate of not less than one sample for every 2,500 square feet (232 m^2) or part thereof of the sprayed area in each story.
2. From beams, girders, joists, trusses and columns at the rate of not less than one sample for each type of structural framing member for each 2,500 square feet (232 m^2) of floor area or part thereof in each story.

~~**1704.10.5**~~ **1704.10.6 Bond strength.** The cohesive/adhesive bond strength of the cured sprayed fire-resistant material applied to structural elements shall not be less than 150 pounds per square foot (psf) (7.18 kN/m2). The cohesive/adhesive bond strength shall be determined in accordance with the field test specified in ASTM E 736 by testing in-place samples of the sprayed fire-resistant material selected in accordance with Sections ~~1704.10.5.1 and 1704.10.5.2.~~ 1704.10.6.1 through 1704.10.6.3.

~~**1704.10.5.1**~~ **1704.10.6.1 Floor, roof and wall assemblies.** The test samples for determining the cohesive/adhesive bond strength of the sprayed fire-resistant materials shall be selected from each floor, roof and wall assembly at the rate of not less than one sample for every ~~10,000~~ 2,500 square feet (~~929~~ 232 m^2) or part thereof of the sprayed area in each story.

~~1704.10.5.2~~ 1704.10.6.2 Structural framing members. The test samples for determining the cohesive/adhesive bond strength of the sprayed fire-resistant materials shall be selected from beams, girders, joists, trusses and columns at the rate of not less than one sample for each type of structural framing member for each ~~10,000~~ 2,500 square feet (~~929~~ 232 m^2) of floor area or part thereof in each story.

1704.10.6.3 Primer, paint and encapsulant bond tests. Bond tests to qualify a primer, paint or encapsulant shall be conducted only when the fire-resistive coating is applied to a primed, painted or encapsulated surface for which acceptable bond-strength performance between these coatings and the fire resistive material has not been measured. A bonding agent approved by the SFRM manufacturer shall to be applied to a primed, painted or encapsulated surface where the bond strengths are found to be below minimum required values.

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

This proposed change is a result of the CTC's investigation of the area of study entitled "Review of NIST WTC Recommendations". The scope of the activity is noted as:

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

This proposal is intended to address only a portion of NIST recommendation 6. For this specific proposed change, CTC is working in cooperation with the NIBS/MMC Committee to Translate the NIST World Trade Center Investigation Recommendations for the Model Codes. The CTC notes in their investigation that many of the recommendations contained in the NIST report require additional information for the CTC to further investigate. As such, CTC intends to continue to study the other NIST recommendations.

NIST Recommendation 6 recommends the development of criteria, test methods and standards: (1) for the in-service performance of sprayed fire-resistance materials (SFRM, also commonly referred to as fireproofing or insulation) used to protect structural components; and (2) to ensure that these materials, as-installed, conform to conditions in tests used to establish the fire resistance rating of components, assemblies, and systems.

As noted above, this proposed change does not address all aspects of NIST recommendation #6. This proposed change is limited to the necessary inspection parameters for spray applied fire resistant materials after installation and renovation of mechanical, plumbing, electrical and other similar systems.

The proposed revisions are intended to coordinate the text of the IBC with the two standards currently referenced in the code- ASTM 605 and ASTM 736, and also AWCI Technical Manual 12-A Standard Practice for the Testing and Inspection of Filed Applied Sprayed Fire-resistive Materials which is a guide and as such, is not referenced in the code. This proposal also as adds sampling criteria for density measurements (proposed Section 1704.10.5) in addition to the current sampling criteria for bond measurements. However, it is noted that there are two significant differences between this proposal and the standards noted. The first is the determination of thickness in proposed Section 1704.10.4 which is not in the standards. By using the standard deviation method, the test samples must fall within a specified range, otherwise, the combination of very thin samples of spray applied coatings with thick samples may lead to the application passing the test when in reality, the thin sections represent an insufficient amount of fire proofing. The second is the sample size. Currently, ASTM E 605 stipulates the 10,000 square foot sample size that is also in the code. Given the critical nature of spray-applied fire proofing, as noted in the NIST report, this sampling size is viewed as too large, resulting in an increased probability of inadequate protection. This proposal uses a value of 2,500 square feet.

Recommendation #6 also addresses the in-service performance (criteria for performance and durability such as bond strength) of spray applied fire resistance which requires further substantiation.

Bibliography:
Interim Report No. 1 of the CTC, Area of Study – Review of NIST WTC Recommendations, March 9, 2006.
National Institute of Standards and Technology. Final Report of the National Construction Safety Team on the Collapses of the World Trade Center Towers. United States Government Printing Office: Washington, D.C. September 2005.

Cost Impact: The code change proposal will increase the cost of construction due to more frequent sampling of spray applied material.

Committee Action: Approved as Modified

Modify proposal as follows:

1704.10 Sprayed fire-resistant materials. Special inspections for sprayed fire-resistant materials applied to structural elements and decks shall be in accordance with Sections 1704.10.1 through 1704.10.6 Special inspections shall be based on the fire-resistance design as designated in the approved construction documents. The tests described in this section shall be based on samplings of specific floor, roof and wall assemblies, and structural framing members. Special inspections shall be performed after the rough installation of electrical, sprinkler, mechanical and plumbing systems and suspension for ceiling systems, where applicable.

1704.10.1 Physical and visual tests. The following physical and visual tests are required to demonstrate compliance with the listing and the fire-resistance rating:

1. Condition of substrates.

2. Thickness of application.
3. Density in pounds per cubic foot (kgs per m^3).
4. Bond strength -adhesion/cohesion.
5. Condition of finished application.

1704.10.2 Structural member surface conditions. The surfaces shall be prepared in accordance with the approved fire-resistance design and the approved manufacturer's written instructions. The prepared surface of structural members to be sprayed shall be inspected before the application of the sprayed fire-resistant material.

1704.10.3 Application. The substrate shall have a minimum ambient temperature before and after application as specified in the approved manufacturer's written instructions. The area for application shall be ventilated during and after application as required by the approved manufacturer's written instructions.

1704.10.4 Thickness. ~~The average thickness minus two times the standard deviation of the thickness measurements~~ No more than 10 percent of the thickness measurements of the sprayed fire-resistant materials applied to structural elements shall ~~not~~ be less than the thickness required by the approved fire-resistant design but in no case less than the minimum allowable thickness required by Section 1704.10.4.1. ~~Individual measured thickness, which exceeds the thickness specified in a design by 1/4 inch (6.4 mm) or more, shall be recorded as the thickness specified in the design plus 1/4 inch (6.4 mm).~~

1704.10.4.1 Minimum allowable thickness. For design thicknesses 1 inch (25 mm) or greater, the minimum allowable individual thickness shall be the design thickness minus 1/4 inch (6.4 mm). For design thicknesses less than 1 inch (25 mm), the minimum allowable individual thickness shall be the design thickness minus 25 percent. Thickness shall be determined in accordance with ASTM E 605. Samples of the sprayed fire-resistant materials shall be selected in accordance with Sections ~~1704.10.4.1~~ 1704.10.4.2 and ~~1704.10.4.2~~ 1704.10.4.3.

~~1704.10.4.1~~ 1704.10.4.2 Floor, roof and wall assemblies. The thickness of the sprayed fire-resistant material applied to floor, roof and wall assemblies shall be determined in accordance with ASTM E 605, taking ~~the average minus two times the standard deviation of the thickness measurements of~~ not less than four measurements for each 1,000 square feet ($93m^2$) of the sprayed area on each floor or part thereof.

~~1704.10.4.1.1~~ 1704.10.4.2.1 Flat decks. Thickness measurements shall be taken from a 12 inches (305 mm) square with a minimum of four measurements, symmetrically.
~~1704.10.4.1.2~~ 1704.10.4.2.2 Fluted decks. Thickness measurements shall be taken from a 12 inches (305 mm) square with four random, symmetrical measurements within the square, including one each of the following: valley, crest and sides and report as an average.

~~1704.10.4.2~~ 1704.10.4.3 Structural framing members. The thickness of the sprayed fire-resistant material applied to structural members shall be determined in accordance with ASTM E 605. Thickness testing shall be performed on not less than 25 percent of the structural members on each floor.

~~1704.10.4.2.1~~ 1704.10.4.3.1 Beams. Thickness measurements shall be made at nine locations around the beam at each end of a 12 inches (305 mm) length.

~~1704.10.4.2.2~~ 1704.10.4.3.2 Joists and trusses. Thickness measurements shall be made at seven locations around the joist or truss at each end of a 12 inches (305 mm) length.

~~1704.10.4.2.3~~ 1704.10.4.3.3 W-shape columns. Thickness measurements shall be made at 12 locations around the column at each end of a 12 inches (305 mm) length.

~~1704.10.4.2.4~~ 1704.10.4.3.4 Tube and pipe columns. Thickness measurements shall be made at a minimum of four locations around the column at each end of a 12 inches (305 mm) length.

1704.10.5 Density. The density of the sprayed fire-resistant material shall not be less than the density specified in the approved fire-resistant design. Density of the sprayed fire-resistant material shall be determined in accordance with ASTM E 605. The test samples for determining the density of the sprayed fire-resistant materials shall be selected as follows:

1. From each floor, roof and wall assembly at the rate of not less than one sample for every 2,500 square feet (232 m^2) or part thereof of the sprayed area in each story.
2. From beams, girders, joists, trusses and columns at the rate of not less than one sample for each type of structural framing member for each 2,500 square feet (232 m^2) of floor area or part thereof in each story.

1704.10.6 Bond strength. The cohesive/adhesive bond strength of the cured sprayed fire-resistant material applied to structural elements shall not be less than 150 pounds per square foot (psf) (7.18 kN/m^2). The cohesive/adhesive bond strength shall be determined in accordance with the field test specified in ASTM E 736 by testing in-place samples of the sprayed fire-resistant material selected in accordance with Sections 1704.10.6.1 through 1704.10.6.3.

1704.10.6.1 Floor, roof and wall assemblies. The test samples for determining the cohesive/adhesive bond strength of the sprayed fire-resistant materials shall be selected from each floor, roof and wall assembly at the rate of not less than one sample for every 2,500 square feet (232 m^2) or part thereof of the sprayed area in each story.

1704.10.6.2 Structural framing members. The test samples for determining the cohesive/adhesive bond strength of the sprayed fire-resistant materials shall be selected from beams, girders, joists, trusses and columns at the rate of not less than one sample for each type of structural framing member for each 2,500 square feet (232 m^2) of floor area or part thereof in each story.

1704.10.6.3 Primer, paint and encapsulant bond tests. Bond tests to qualify a primer, paint or encapsulant shall be conducted only when the fire-resistive coating is applied to a primed, painted or encapsulated surface for which acceptable bond-strength performance between these coatings and the fire resistive material has not been measured. A bonding agent approved by the SFRM manufacturer shall to be applied to a primed, painted or encapsulated surface where the bond strengths are found to be below minimum required values.

Committee Reason: This proposal provides the details to allow for verification that the sprayed fire-resistant material is properly installed. Given the actions the committee has previously taken to assure that the materials are appropriately applied (FS100-06/07) and that the conditions during the application are appropriate (G68-06/07), the inspection is important to verify installation and to help assure proper performance. The modifications deleted the requirements that the acceptance of the inspection measurements be based upon the "standard deviation." Since this is intended as a means of field inspection, the connection to "standard deviation" was deleted and replaced by the 10 percent limitation. The intent of both the original and this revised text is to provide a 95 percent confidence level that the installed material exceeds the requirements. The committee did note that Section 1704.10.6 of the proposal does refer to the bond strength of 150 pounds. Based on the action taken with code change G68-06/07 a public comment which directs code users to the new Table 403.15 is needed for the high-rise buildings which require a greater bond strength.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

Philip Brazil, P.E., Reid Middleton, Inc, representing himself, requests Approval as Modified by this public comment.

Modify proposal as follows:

1704.10 Sprayed fire-resistant materials. Special inspections for sprayed fire-resistant materials applied to floor, roof and wall assemblies and structural ~~elements~~ members ~~and decks~~ shall be in accordance with Sections 1704.10.1 through 1704.10.6. Special inspections shall be based on the fire-resistance design as designated in the approved construction documents. The tests ~~described~~ set forth in this section shall be based on samplings ~~of~~ from specific floor, roof and wall assemblies, and structural ~~framing~~ members. Special inspections shall be performed after the rough installation of electrical, automatic sprinkler, mechanical and plumbing systems and suspension systems for ceilings ~~systems~~, where applicable.

1704.10.1 Physical and visual tests. The special inspections shall include the following ~~physical and visual~~ tests and observations ~~are required~~ to demonstrate compliance with the listing and the fire-resistance rating:

1. Condition of substrates.
2. Thickness of application.
3. Density in pounds per cubic foot (~~kgs per m^3~~ kg/m^3).
4. Bond strength-adhesion/cohesion.
5. Condition of finished application.

1704.10.2 Structural member surface conditions. The surfaces shall be prepared in accordance with the approved fire-resistance design and the ~~approved manufacturer's~~ written instructions of approved manufacturers. The prepared surface of structural members to be sprayed shall be inspected before the application of the sprayed fire-resistant material.

1704.10.3 Application. The substrate shall have a minimum ambient temperature before and after application as specified in the ~~approved manufacturer's~~ written instructions of approved manufacturers. The area for application shall be ventilated during and after application as required by the ~~approved manufacturer's~~ written instructions of approved manufacturers.

1704.10.4 Thickness. No more than 10 percent of the thickness measurements of the sprayed fire-resistant materials applied to floor, roof and wall assemblies and structural ~~elements~~ members shall be less than the thickness required by the approved fire~~-resistant~~ resistance design but in no case less than the minimum allowable thickness required by Section 1704.10.4.1.

1704.10.4.1 Minimum allowable thickness. For design thicknesses 1 inch (25 mm) or greater, the minimum allowable individual thickness shall be the design thickness minus 1/4 inch (6.4 mm). For design thicknesses less than 1 inch (25 mm), the minimum allowable individual thickness shall be the design thickness minus 25 percent. Thickness shall be determined in accordance with ASTM E 605. Samples of the sprayed fire-resistant materials shall be selected in accordance with Sections 1704.10.4.2 and 1704.10.4.3.

1704.10.4.2 Floor, roof and wall assemblies. The thickness of the sprayed fire-resistant material applied to floor, roof and wall assemblies shall be determined in accordance with ASTM E 605, ~~taking~~ making not less than four measurements for each 1,000 square feet (93 m^2) of the sprayed area ~~on each floor~~ in each story or ~~part~~ portion thereof.

1704.10.4.2.1 ~~Flat~~ Cellular decks. Thickness measurements shall be ~~taken~~ selected from a square area, 12 inches (305 mm) ~~square with a~~ by 12 inches (305 mm) in size. A minimum of four measurements shall be made, located symmetrically within the square area.

1704.10.4.2.2 Fluted decks. Thickness measurements shall be ~~taken~~ selected from a square area, 12 inches (305 mm) ~~square with~~ by 12 inches (305 mm) in size. A minimum of four ~~random, symmetrical~~ measurements shall be made, located symmetrically within the square area, including one each of the following: valley, crest and sides and report as an average.

1704.10.4.3 Structural ~~framing~~ members. The thickness of the sprayed fire-resistant material applied to structural members shall be determined in accordance with ASTM E 605. Thickness testing shall be performed on not less than 25 percent of the structural members on each floor.

1704.10.4.3.1 Beams and girders. At beams and girders, thickness measurements shall be made at nine locations around the beam or girder at each end of a ~~12 inches~~ 12-inch (305 mm) length.

1704.10.4.3.2 Joists and trusses. At joists and trusses, thickness measurements shall be made at seven locations around the joist or truss at each end of a ~~12 inches~~ 12-inch (305 mm) length.

1704.10.4.3.3 ~~W-shape~~ Wide-flanged columns. At wide-flanged columns, thickness measurements shall be made at 12 locations around the column at each end of a ~~12 inches~~ 12-inch (305 mm) length.

1704.10.4.3.4 ~~Tube~~ Hollow structural section and pipe columns. At hollow structural section and pipe columns, thickness measurements shall be made at a minimum of four locations around the column at each end of a ~~12 inches~~ 12-inch (305 mm) length.

1704.10.5 Density. The density of the sprayed fire-resistant material shall not be less than the density specified in the approved fire-~~resistant~~ resistance design. Density of the sprayed fire-resistant material shall be determined in accordance with ASTM E 605. The test samples for determining the density of the sprayed fire-resistant materials shall be selected as follows:

1. From each floor, roof and wall assembly at the rate of not less than one sample for every 2,500 square feet (232 m^2) or ~~part~~ portion thereof of the sprayed area in each story.
2. From beams, girders, ~~joists,~~ trusses and columns at the rate of not less than one sample for each type of structural ~~framing~~ member for each 2,500 square feet (232 m^2) of floor area or ~~part~~ portion thereof in each story.

1704.10.6 Bond strength. The cohesive/adhesive bond strength of the cured sprayed fire-resistant material applied to floor, roof and wall assemblies and structural ~~elements~~ members shall not be less than 150 pounds per square foot (psf) (7.18 kN/m^2). The cohesive/adhesive bond strength shall be determined in accordance with the field test specified in ASTM E 736 by testing in-place samples of the sprayed fire-resistant material selected in accordance with Sections 1704.10.6.1 through 1704.10.6.3.

1704.10.6.1 Floor, roof and wall assemblies. The test samples for determining the cohesive/adhesive bond strength of the sprayed fire-resistant materials shall be selected from each floor, roof and wall assembly at the rate of not less than one sample for every 2,500 square feet (232 m^2) ~~or part thereof~~ of the sprayed area in each story or portion thereof.

1704.10.6.2 Structural ~~framing~~ members. The test samples for determining the cohesive/adhesive bond strength of the sprayed fire-resistant materials shall be selected from beams, girders, ~~joists,~~ trusses, ~~and~~ columns and other structural members at the rate of not less than one sample for each type of structural ~~framing~~ member for each 2,500 square feet (232 m^2) of floor area or ~~part~~ portion thereof in each story.

1704.10.6.3 Primer, paint and encapsulant bond tests. Bond tests to qualify a primer, paint or encapsulant shall be conducted ~~only~~ when the ~~fire-resistive coating~~ sprayed fire-resistant material is applied to a primed, painted or encapsulated surface for which acceptable bond-strength performance between these coatings and the fire ~~resistive~~ resistant material has not been ~~measured~~ determined. A bonding agent approved by the SFRM manufacturer shall ~~to~~ be applied to a primed, painted or encapsulated surface where the bond strengths are found to be ~~below minimum~~ less than required values.

Commenter's Reason: The purpose for this public comment is to make editorial revisions to Proposal S38. "Structural elements and decks" in Section 1704.10 and "structural elements" in Sections 1704.10.4 and 1704.10.6 are changed to "floor, roof and wall assemblies and structural members" for consistency with Section 714 on the fire-resistance rating of structural members and for consistency with Sections 1704.10, 1704.10.4.2, 1704.10.3 and 1704.10.6.1 on "floor, roof and wall assemblies." "Structural framing members" are changed to "structural members" in Sections 1704.10 and 1704.10.6.2 for the same reason. Replacing "decks" with "floor, roof and wall assemblies" is also more comprehensive and takes into account special steel plate shear walls (i.e., Table 12.2-1 of ASCE 7-05).

Several other changes to Section 1704.10 are proposed. "Described" is changed to "set forth" to avoid non-mandatory language. "Sprinkler" is changed to "automatic sprinkler" for consistency with the terminology in Section 903 on automatic sprinkler systems. "Suspension for ceiling systems" is changed to "suspension systems for ceilings" for consistency with ASTM C 635 and C 636 on suspension systems for acoustical tile and lay-in panel ceilings, which are referenced in Section 803.9.1.1.

In Section 1704.10.1, "physical and visual tests" are replaced with "tests and observations" because Items #1 and #5 of Section 1704.10.1 are not accomplished by tests, but by the observations of the special inspector. In Section 1704.10.4, "fire-resistant design" is changed to "fire-resistance design" for consistency with Item #1 of Section 703.3, "fire-resistance designs documented in approved sources." In Section 1704.10.4.2, "on each floor" is changed to "in each story" for consistency with Section 1704.10.6.1. In Section 1704.10.4.3.1, "beams" is changed to "beams and girders" for consistency with the use of "beams" and "girders" in Section 1704.10.6.2. In Sections 1704.10.4.2, 1704.10.6.1 and 1704.10.6.2, "part" is changed to "portion" because area, not a structural element, is typically referenced.

The proposed revisions to Sections 1704.10.4.2.1, 1704.10.4.2.2 and 1704.10.4.3.1 through 1704.10.4.3.4 are intended to bring technical soundness to the provisions and to employ terms more commonly used by the structural engineering profession for the design of structural steel and by nationally recognized testing laboratories in their listings of fire-resistance-rated designs containing sprayed fire-resistant materials. "Flat deck" is changed to "cellular deck" for consistency with the same term used for fluted steel decks with steel sheet added to form flat bottom surfaces. "Random, symmetrical" is replaced by "symmetrical" because symmetrical measurements are not random, they are intentional. References to the subject of Sections 1704.10.4.3.1 through 1704.10.4.3.4 are added to the text because these sections, as written, rely on the title of each section for their charging language. "Tube columns" are changed to "hollow structural section columns" for consistency with current AISC terminology. The language is revised to consistently "select" measurements from specific areas and "make," not "take," the measurements in these areas. "Taking measurements implies sampling whereas measurements (i.e., thickness) are typically nondestructive.

In Section 1704.10.4.3.3, "W-shape columns" are changed to "wide-flanged columns" for consistency with Section 714.8.3.2 in Proposal FS100-06/07. Note that W-shaped, M-shaped, S-shaped and HP-shaped" structural steel columns are manufactured. The current AISC Specification (AISC 360-05) typically refers to "I-shaped members" (i.e., Chapter F). The 2005 AISC Steel Construction Manual, however, typically refers to W-shapes, M-shapes, S-shapes and HP-shapes, which are described collectively as "H-shaped" and "I-shaped" (i.e., Scope). Because of this level of detail, relying on "W-shape columns" in Section 1704.10.4.3.3 can lead to considerable confusion that can be avoided by use of the more generic "wide-flanged columns."

Several other changes to Section 1704.10.6.3 are proposed. "Fire-resistive coating" is changed to "sprayed fire-resistant material" for consistency with the other provisions in Section 1704.10. Note that special inspection of mastic and intumescent fire-resistant coatings is specified in Section 1704.11. "Fire-resistive material" is changed to "fire-resistant material," also for consistency with the other provisions in Section 1704.10. "Measured" is changed to "determined" because it refers to "acceptable bond strength performance," not the results of tests.

Final Action: AS AM AMPC____ D

E2-06/07

1001.4

Proposed Change as Submitted:

Proponent: Dave Frable, U.S. General Services Administration

Add new text as follows:

<u>**1001.4 Emergency planning.** Emergency planning and preparedness provisions shall be provided for all occupancies and buildings as required by Chapter 4 of the *International Fire Code*.</u>

Reason: The purpose of this code change proposal is to provide consistent requirements for jurisdictions regarding emergency planning and preparedness. Many jurisdictions across the country currently have adopted the IBC, however many of these same jurisdictions have not adopted the IFC. Hence, this proposed code change will provide consistent requirements for emergency planning and preparedness in all jurisdictions that adopt the IBC. Effectively, the IBC will adopt all of the emergency planning and preparedness provisions in the IFC.

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

Committee Action: **Disapproved**

Committee Reason: The intent is well founded, however, locating emergency planning requirements in Chapter 10 is inappropriate. Emergency planning is not a construction issues. Emergency planning is the purview of the IFC and the maintenance part of these plans are enforced by the fire officials.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment:

Paul K. Heilstedt, PE, Chair, ICC Code Technology Committee (CTC) and Dave Frable, U.S. General Service Administration request Approval as Modified by this public comment.

Replace proposal with the following:

<u>**1001.4 Fire safety and evacuation plans:** Fire safety and evacuation plans shall be provided for all occupancies and buildings where required by the International Fire Code. Such fire safety and evacuation plans shall comply with the applicable provisions of the International Fire Code.</u>

Commenter's Reason: The committee action notes a concern over the location of the text. While emergency planning is not a construction issue, it is clearly an issue which needs to at least be referenced in the building code in order for the designer to be aware that after the building is constructed, there are provisions in the IFC that will be applied on the day the building is occupied. Further, not all jurisdictions adopt the IFC. This reference will ensure that at least the fire safety and evacuation plans of the IFC are adopted by reference. Enforcement of the provisions is not an issue. The provisions are clearly within the scope of the IFC.

Final Action: AS AM AMPC ____ D

E137-06/07
1020.1 (IFC [B] 1020.1)

Proposed Change as Submitted:

Proponent: William M. Connolly, State of New Jersey, Department of Community Affairs, Division of Codes and Standards, representing International Code Council Ad Hoc Committee on Terrorism Resistant Buildings

Revise as follows:

1020.1 Enclosures required. Interior exit stairways and interior exit ramps shall be enclosed with fire barriers constructed in accordance with Section 706 or horizontal assemblies constructed in accordance with Section 711, or both. Exit enclosures shall have a fire-resistance rating of not less than 2 hours where connecting four stories or more and not less than 1 hour where connecting less than four stories. The number of stories connected by the shaft enclosure shall include any basements but not any mezzanines. An exit enclosure shall not be used for any purpose other than means of egress. <u>Exit stair enclosures shall be continuous from the highest story served by the enclosure to the level of exit discharge and shall not include horizontal transfer corridors other than at the level of exit discharge in accordance with Section 1024.</u>

Exceptions:

1. In all occupancies, other than GroupHand I occupancies, a stairway is not required to be enclosed when the stairway serves an occupant load of less than 10 and the stairway complies with either Item 1.1 or 1.2. In all cases, the maximum number of connecting open stories shall not exceed two.
   - 1.1. The stairway is open to not more than one story above the story at the level of exit discharge; or
   - 1.2. The stairway is open to not more than one story below the story at the level of exit discharge.
2. Exits in buildings of Group A-5 where all portions of the means of egress are essentially open to the outside need not be enclosed.
3. Stairways serving and contained within a single residential dwelling unit or sleeping unit in Group R-1, R-2 or R-3 occupancies are not required to be enclosed.
4. Stairways that are not a required means of egress element are not required to be enclosed where such stairways comply with Section 707.2.
5. Stairways in open parking structures that serve only the parking structure are not required to be enclosed.
6. Stairways in Group I-3 occupancies, as provided for in Section 408.3.6, are not required to be enclosed.
7. Means of egress stairways as required by Section 410.5.3 are not required to be enclosed.
8. In other than Group H and I occupancies, a maximum of 50 percent of egress stairways serving one adjacent floor are not required to be enclosed, provided at least two means of egress are provided from both floors served by the unenclosed stairways. Any two such interconnected floors shall not be open to other floors. Unenclosed exit stairways shall be remotely located as required in Section 1015.2.
9. In other than Group H and I occupancies, interior egress stairways serving only the first and second stories of a building equipped throughout with an automatic sprinkler system in accordance with Section 903.3.1.1 are not required to be enclosed, provided at least two means of egress are provided from both floors served by the unenclosed stairways. Such interconnected stories shall not be open to other stories. Unenclosed exit stairways shall be remotely located as required in Section 1015.2.

Reason: This code change proposal is one of fourteen proposals being submitted by the International Code Council Ad Hoc Committee on Terrorism Resistant Buildings.

The purpose of this proposal is to eliminate the use of horizontal transfer corridors on upper floors thereby requiring an exit stair shaft to descend directly to the level of approved exit discharge.

The National Institute of Standards and Technology (NIST) World Trade Center (WTC) Report pointed out that horizontal transfers from one shaft to another caused occupant confusion and thereby slowed egress time. The WTC Report also recommended that Codes be revised to address the need for full building evacuation in the shortest possible time.

This proposal amends Section 1020.1 of the Code to require that exit stair enclosures be continuous from the top to the level of exit discharge. This will promote prompt evacuations. Some would argue that occupants can be trained to accept the counterintuitive horizontal transfers. Given the impracticality of full drills in high rise buildings, this training will be paper or lecture-based. At any given time, the building will have occupants who have not been trained. The proponents believe it is better to eliminate the unnatural rather than trying to train building occupants, who will be highly stressed, to expect and accept it.

Some will argue that this provision will put constraints on design. Of course it will. All safety requirements put constraints on design. It may take a little extra effort on the part of designers, but good buildings can incorporate this type of feature if designers put safety first.

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

Cost Impact: The code change proposal will not increase the cost of construction. It can be met with careful design alone.

Committee Action: Disapproved

Committee Reason: This proposal, by eliminating the option of horizontal transfers in the exit enclosure, places severe limitations on building design. Horizontal movement may be necessary for adequate dispersion of exits in buildings with setbacks or to move around equipment floors. The proposed text uses the term 'corridor' instead of 'exit passageway'. The NIST report did mention delays at transfer floors, but most evacuation drills had not include actual travel down the stairways. This concern could have been partially addressed by fire drills/training.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment 1:

Edmund C. Domian, West Valley City, Utah, requests Approval as Submitted.

Commenter's Reason: It is important that exit stair enclosures be designed as the most direct route to the building exterior from any point in a building, regardless of the floor plan, the occupant load, or the number of stories between the occupant and the level of exit discharge. Expecting occupants to traverse between stair enclosures down corridors along a complicated exit path in an emergency just adds to the panic and confusion and costs valuable travel time. The proponent's reason was on target. This change would not prohibit any occupant from entering any floor from the stairway for business as usual. It just prohibits ambiguous detours created by some architectural vision or cost cutting plan.

Public Comment 2:

William M. Connolly, State of New Jersey, Department of Community Affairs, Division of Codes and Standards, representing International Code Council Ad Hoc Committee on Terrorism Resistant Buildings, requests Approval as Modified by this public comment.

Staff Note: The proposal includes movement of the proposed requirement from Section 1020 *Vertical Exit Enclosures* to Section 1021 *Exit Passageways*.

Modify proposal as follows:

1020.1 Enclosures required. Interior exit stairways and interior exit ramps shall be enclosed with fire barriers constructed in accordance with Section 706 or horizontal assemblies constructed in accordance with Section 711, or both. Exit enclosures shall have a fire-resistance rating of not less than 2 hours where connecting four stories or more and not less than 1 hour where connecting less than four stories. The number of stories connected by the shaft enclosure shall include any basements but not any mezzanines. An exit enclosure shall not be used for any purpose other than means of egress. ~~Exit stair enclosures shall be continuous from the highest story served by the enclosure to the level of exit discharge and shall not include horizontal transfer corridors other than at the level of exit discharge in accordance with Section 1024.~~

Exceptions:

1. In all occupancies, other than Group Hand I occupancies, a stairway is not required to be enclosed when the stairway serves an occupant load of less than 10 and the stairway complies with either Item 1.1 or 1.2. In all cases, the maximum number of connecting open stories shall not exceed two.
   1.1. The stairway is open to not more than one story above the story at the level of exit discharge; or
   1.2. The stairway is open to not more than one story below the story at the level of exit discharge.
2. Exits in buildings of Group A-5 where all portions of the means of egress are essentially open to the outside need not be enclosed.
3. Stairways serving and contained within a single residential dwelling unit or sleeping unit in Group R-1, R-2 or R-3 occupancies are not required to be enclosed.
4. Stairways that are not a required means of egress element are not required to be enclosed where such stairways comply with Section 707.2.
5. Stairways in open parking structures that serve only the parking structure are not required to be enclosed.
6. Stairways in Group I-3 occupancies, as provided for in Section 408.3.6, are not required to be enclosed.
7. Means of egress stairways as required by Section 410.5.3 are not required to be enclosed.
8. In other than Group H and I occupancies, a maximum of 50 percent of egress stairways serving one adjacent floor are not required to be enclosed, provided at least two means of egress are provided from both floors served by the unenclosed

stairways. Any two such interconnected floors shall not be open to other floors. Unenclosed exit stairways shall be remotely located as required in Section 1015.2.

9. In other than Group H and I occupancies, interior egress stairways serving only the first and second stories of a building equipped throughout with an automatic sprinkler system in accordance with Section 903.3.1.1 are not required to be enclosed, provided at least two means of egress are provided from both floors served by the unenclosed stairways. Such interconnected stories shall not be open to other stories. Unenclosed exit stairways shall be remotely located as required in Section 1015.2.

Add new sections as follows:

1021.3 Length. In buildings with an occupied floor located more than 75 feet (22860 mm) above the lowest level of fire department vehicle access, exit passageways used to connect vertical exit enclosures on floors, other than the level of exit discharge, shall be no more than 50 feet (15 240 mm) in length.

1021.3.1 Signage. Exit passageways, that change direction at other than the level of exit discharge, shall be provided with directional signage. The words "Exit continues this way" and directional arrows shall comply with Section 1011.5.1.

(Renumber current 1021.3 through 1021.5)

Commenter's Reason: The main reason for the committee's disapproval of the change was "eliminating the option of horizontal transfers in the exit enclosure places severe limitations of building design". The committee did acknowledge that the NIST report on the World Trade Center did mention delays at transfer floors. This public comment eliminates the restriction on horizontal transfers; however, it does places a restriction on the horizontal length of travel that an occupant is required to traverse. The 50 feet limit on the exit passageway that being used for horizontal transfer is consistent with the code's limit on dead end corridors. The code currently requires horizontal transfer in exit enclosures to comply with Section 1021; this public comment merely places a restriction on the transfer length as well as provides for directional signage within the exit passageway.

Final Action: AS AM AMPC ____ D

G63-06/07

403.10 (New), 3007 (New), Chapter 35 (New)

Proposed Change as Submitted:

Proponents: Dave Frable, U.S. General Services Administration; Gerry Jones/Herman Brice, Co-Chairs, NIBS/MMC Committee for Translating the NIST World Trade Center Investigation Recommendations into Building Codes

1. Add new text as follows:

403.10 Fire service access elevator. In buildings with an occupied floor more than 120 feet above the lowest level of fire department vehicle access, a minimum of one fire service access elevator shall be provided in accordance with Section 3007.

(Renumber subsequent sections)

SECTION 3007
FIRE SERVICE ACCESS ELEVATOR

3007.1 General. Where required by Section 403.10, every floor of the building shall be served by a fire service access elevator. Except as modified in this section, the fire service access elevator shall be installed in accordance with this chapter and ASME A17.1.

3007.2 Hoistway enclosures protection. The fire service access elevator shall be located in a shaft enclosure complying with Section 707.

3007.3 Fire service access elevator lobby. The fire service access elevator shall have a door opening into a fire service access elevator lobby complying with Sections 3007.3.1 through 3007.3.3.

Exception: Where fire service access elevators have multiple door openings on a floor, additional door openings shall be permitted to open to lobbies protected in accordance with Section 707.14.1.

3007.3.1 Access. The fire service access elevator lobby shall have direct access to a building stair.

3007.3.2 Lobby enclosure. The fire service access elevator lobby enclosure shall have a minimum 1-hour fire resistance rating.

3007.3.3 Lobby fire door assemblies. Each fire service access elevator lobby fire door shall have a fire protection rating of not less than 1 hour and shall be self closing or automatic closing.

3007.4 Standpipe hose connection. Each building exit stair having direct access to the fire service access elevator lobby shall be provided with a standpipe hose connection in accordance with Section 905.

3007.5 Two-way fire department communication system. The fire service access elevator and every associated fire service access elevator lobby shall be provided with an approved two-way fire department communication system. It shall operate between a fire command center complying with Section 911 and the fire service access elevator and every associated fire service access elevator lobby. The two-way fire department communication system shall be designed and installed in accordance with NFPA 72.

Exception: Fire department radio systems where approved by the fire department.

3007.6 Elevator car size. The elevator car size shall be in accordance with Section 3002.4.

3007.7 Elevator system monitoring. Conditions necessary for the continued safe operation of the fire service access elevator shall be continuously monitored at the fire command center by a *Standard Fire Service Interface* system meeting the requirements of NFPA 72 and NEMA SB30.

3007.8 Electrical power. The following features associated with fire service access elevators shall be supplied by both normal power and Type 60/Class 2/Level 1 standby power:

1. Elevator equipment
2. Elevator machine room ventilation and cooling equipment
3. Elevator controller cooling equipment

3007.8.1 Control wiring The normal and standby power control wiring supplying the fire service access elevators shall be protected by construction having a minimum 1 hour fire resistive rating.

3007.9 Standby power. The fire service access elevator shall be provided with standby power in accordance with Sections 2702 and 3003.

3007.10 Elevator machine rooms and machinery spaces. Automatic fire sprinklers shall not be installed in fire service access elevator machine rooms and machinery spaces.

2. Add new standards organization and standard to Chapter 35 as follows:

National Electrical Manufacturer's Association (NEMA)
1300 N. 17th Street
Suite 1847
Rosslyn, VA 22209

SB30-05 Fire Service Annunciator and Interface

Reason: Following the events of September 11, 2001, the U.S. General Services Administration (GSA) undertook a research initiative for the development of performance requirements for the use of elevators for occupant egress and fire service access in buildings. This research initiative is currently being conducted by the National Institute of Standards and Technology (NIST). The proposed code change is a by-product of the research currently being conducted by NIST as well preliminary information provided by a task group of ASME A17.1 for determining the required system features necessary for safe operation by trained firefighters during a fire emergency.

We feel that the requirements included in this proposal provide a reasonable degree of safety for firefighters operating the fire service access elevator to a location for staging firefighters and equipment one or two floors below the fire. The staging location will have access to a stair and standpipe that will allow for firefighting operations to be conducted from just above the staging area.

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

Analysis: Results of review of the proposed standard will be posted on the ICC Website by August 20, 2006.

Note: The following analysis was not in the Code Change Proposal book but was published in the "Errata to the 2006/2007 Proposed Changes to the International Codes and Analysis of Proposed Referenced Standards" provided at the code development hearings:

Analysis: Review of the proposed new standard indicated that, in the opinion of ICC Staff, the standard did not comply with ICC standards criteria, Section 3.6.2.1 for mandatory language.

Committee Action: **Disapproved**

Committee Reason: Generally the committee was in favor of the proposal but disapproved the code change based upon a variety of issues that needed to be addressed. For instance there was concern with terminology in proposed section 3007.3.1 which currently references a "building stair" instead of an "exit enclosure." Other concerns related to the standard reference to NEMA SB30 and the size of the elevator lobby.

Assembly Action: **Approved as Submitted**

Individual Consideration Agenda

This item is on the agenda for individual consideration because an assembly action was successful and public comments were submitted.

Public Comment 1:

Dave Frable, U.S. General Services Administration requests Approval as Modified by this public comment.

1. Modify proposal as follows:

403.10 Fire service access elevator. In buildings with an occupied floor more than 120 feet above the lowest level of fire department vehicle access, a minimum of one fire service access elevator shall be provided in accordance with Section 3007.

(Renumber subsequent sections)

SECTION 3007
FIRE SERVICE ACCESS ELEVATOR

3007.1 General. Where required by Section 403.10, every floor of the building shall be served by a fire service access elevator. Except as modified in this section, the fire service access elevator shall be installed in accordance with this chapter and ASME A17.1.

3007.2 Hoistway enclosures protection. The fire service access elevator shall be located in a shaft enclosure complying with Section 707.

3007.3 Fire service access elevator lobby. The fire service access elevator shall ~~have a door opening~~ open into a fire service access elevator lobby ~~complying~~ in accordance with Sections 3007.3.1 through 3007.3.3.

> **Exception:** Where a fire service access elevator~~s have multiple~~ has two entrances ~~door openings~~ onto a floor, the second ~~additional~~ entrance ~~door openings~~ shall be permitted to open into an elevator lobby ~~lobbies protected~~ in accordance with Section 707.14.1.

3007.3.1 Access. The fire service access elevator lobby shall have direct access to an exit enclosure ~~building stair.~~

3007.3.2 Lobby enclosure. The fire service access elevator lobby ~~enclosure~~ shall be enclosed with a smoke barrier having ~~have~~ a minimum 1-hour fire resistance rating, except that lobby doorways shall comply with Section 3007.3.3.

> **Exception:** Enclosed fire service access elevator lobbies are not required at the street floor.

3007.3.3 Lobby ~~fire door assemblies~~ doorways. Each fire service access elevator lobby ~~fire door~~ shall ~~have a fire protection rating of not less than 1-hour and shall be self closing or automatic closing~~ be provided with a doorway that is protected with a ¾ hour fire door assembly complying with Section 715.4.

3007.4 Standpipe hose connection. ~~Each building exit stair having direct access to the fire service access elevator lobby shall be provided with a~~ A Class I standpipe hose connection in accordance with Section 905 shall be provided in the exit enclosure having direct access from the fire service access elevator lobby.

~~**3007.5 Two-way fire department communication system.** The fire service access elevator and every associated fire service access elevator lobby shall be provided with an approved two-way fire department communication system. It shall operate between a fire command center complying with Section 911 and the fire service access elevator and every associated fire service access elevator lobby. The two-way fire department communication system shall be designed and installed in accordance with NFPA 72.~~

> ~~**Exception**: Fire department radio systems where approved by the fire department.~~

~~**3007.6 Elevator car size.** The elevator car size shall be in accordance with Section 3002.4.~~

3007.7 Elevator system monitoring. The ~~Conditions necessary for the continued safe operation of the~~ fire service access elevator shall be continuously monitored at the fire command center by a *Standard ~~Fire~~ Emergency Service Interface* system meeting the requirements of NFPA 72 ~~and NEMA SB30~~.

3007.8 Electrical power. The following features ~~associated with~~ serving each fire service access elevator~~s~~ shall be supplied by both normal power and Type 60/Class 2/Level 1 standby power:

1. Elevator equipment
2. Elevator machine room ventilation and cooling equipment
3. Elevator controller cooling equipment

3007.8.1 Protection of wiring or cables ~~Control wiring~~. ~~The normal and standby power control wiring supplying the fire service access elevators shall be protected by construction having a minimum 1 hour fire resistive rating.~~ Wires or cables that provide normal and standby power, control signals, communication with the car, lighting, heating, air conditioning, ventilation, and fire detecting systems to fire service access elevators shall be protected by construction having a minimum 1 hour fire resistance rating or shall be circuit integrity cable having a minimum 1 hour fire resistance rating.

~~**3007.9 Standby power**. The fire service access elevator shall be provided with standby power in accordance with Sections 2702 and 3003~~.

~~**3007.10 Elevator machine rooms and machinery spaces.** Automatic fire sprinklers shall not be installed in fire service access elevator machine rooms and machinery spaces.~~

2. Revise as follows:

903.3.1.1 NFPA 13 sprinkler systems. Where the provisions of this code require that a building or portion thereof be equipped throughout with an automatic sprinkler system in accordance with this section, sprinklers shall be installed throughout in accordance with NFPA 13 except as provided in Section 903.3.1.1.1.

903.3.1.1.1 Exempt locations. Automatic sprinklers shall not be required in the following rooms or areas where such rooms or areas are protected with an approved automatic fire detection system in accordance with Section 907.2 that will respond to visible or invisible particles of combustion. Sprinklers shall not be omitted from any room merely because it is damp, of fire-resistance rated construction or contains electrical equipment.

1. Any room where the application of water, or flame and water, constitutes a serious life or fire hazard.
2. Any room or space where sprinklers are considered undesirable because of the nature of the contents, when approved by the fire code official.
3. Generator and transformer rooms separated from the remainder of the building by walls and floor/ceiling or roof/ceiling assemblies having a fire-resistance rating of not less than 2 hours.
4. In rooms or areas that are of noncombustible construction with wholly noncombustible contents.
5. In fire service access elevator machine rooms and machinery spaces.

Commenter's Reason: As the proponent of the original code change proposal, I submit this comment to support the successful Assembly Action in Lake Buena Vista that recommended approval of this code change. The proposed code change is a by-product of research currently being conducted by the National Institute of Standards and Technology (NIST) and funded by the U.S. General Services Administration. Overall, the General Code Committee stated they were in favor of the code change proposal but disapproved the code change proposal based on a number of issues. The purpose of this modified code change is to address the issues raised by the General Code Committee.

1. **3007.3 Fire service access elevator lobby.** This paragraph was revised for clarification purposes only. The exception also clarifies that if the fire service access elevator has two entrances, the second entrance is permitted to open into an enclosed or otherwise protected elevator lobby in accordance with 707.14.1.
2. **3007.3.1 Access.** This paragraph was revised for clarification purposes only. This change addresses the concerns of the Code Committee regarding the term "building stair".
3. **3007.3.2 Lobby enclosure.** This paragraph was revised for clarification purposes only. This change addresses concerns that this paragraph was non-specific with regard to the enclosure requirements for the lobby. A smoke barrier is the appropriate reference since it is designed to resist fire and smoke spread and is intended to create an area of refuge. The new exception addresses the need for not requiring an enclosed lobby on the street floor.
4. **3007.3.3 Lobby doorways.** The title and content of this paragraph was revised for clarification purposes only. A horizontal separation is allowed to be protected with a ¾ hour fire door assembly. Clearly a ¾ hour fire rated door provides more protection then a 20 minute fire door typically required by a smoke barrier.
5. **3007.4 Standpipe hose connection.** This paragraph was revised for clarification purposes only. It was felt that since 905.4 is limited to "required" exit stairways; this paragraph will still require a standpipe hose connection in non-required or additional exit stairways.
6. **3007.5 Two-way fire department communication system.** This paragraph was deleted since it is redundant and currently covered in the IBC.
7. **3007.6 Elevator car size.** This paragraph was deleted since it is redundant and currently covered in the IBC.
8. **3007.7 Elevator system monitoring**. This paragraph was revised to delete superfluous text, correct the title of the interface system, and to delete the reference to NEMA SB30.
9. **3007.8 Electrical power**. This paragraph was revised for clarification purposes only.
10. **3007.8.1 Protection of wiring or cables.** The title and content of this paragraph was revised for clarification purposes only.
11. **3007.9 Standby power**. This paragraph was deleted since it is redundant and currently covered in the IBC.
12. **3007.10 Elevator machine rooms and machinery spaces.** This paragraph was relocated to Chapter 9 based on a recommendation from the General Code Committee. See new exemption number 5, 903.3.1.1.1.

903.3.1.1.1 Exempt locations (new exemption 5). Added new exemption number 5. Relocated material from 3007.10. Provides an exemption for providing automatic sprinklers in fire service access elevator machine rooms and machinery spaces. The need for providing sprinkler protection in these areas is questionable based on the lack of fire loss statistics for elevator machine room and machinery space fires and the concerns from ASME regarding disconnecting the main power line to the affected elevators prior to the application of water from the sprinklers.

Public Comment 2:

Paul K. Heilstedt, PE, Chair, ICC Code Technology Committee (CTC), requests Approval as Modified by this public comment.

Modify proposal as follows:

3007.7 Elevator system monitoring. Conditions necessary for the continued safe operation of the fire service access elevator shall be continuously monitored at the fire command center by a *Standard Fire Service Interface* system meeting the requirements of NFPA 72 ~~and NEMA SB30~~.

(Portions of proposal not shown remain unchanged)

Commenter's reason: This proposal was disapproved by the committee, noting that that there were concerns due to some of the terminology and the reference to a proposed standard that does not meet ICC standard's criteria. It is further noted that this proposal gained the support of the assembled body in Orlando as evidenced by the successful assembly action.

This public comment addresses the issue of the non-compliant standard by deleting the proposed standard. This standard is not needed for purposes of the monitoring of the elevator system as the provisions of Section 7.10 of NFPA 72 include standard fire service interface provisions. Further, while the committee notes that some of the language is in need of possible clarification, this concept is very relevant and important in terms of fire fighting operations and occupant egress safety. The concept of a fire service elevator in tall buildings (more than 120 feet) is long overdue to be considered in the building code.

Public Comment 3:

Tim Pate, City and County of Broomfield, Colorado, representing himself requests Approval as Modified by this public comment.

Modify proposal as follows:

3007.3.1 Access. The fire service access elevator lobby shall have direct access to a ~~building stair~~ <u>exit enclosure</u> .

(Portions of proposal not shown remain unchanged)

Commenter's Reason: It was pointed out during testimony from the Committee that the building stair does not have any specific requirements to actually extend to the exit discharge. This modification would require the elevator lobby to have direct access to a vertical exit enclosure which of course leads to the exit discharge.

Final Action: AS AM AMPC _______ D

G72-06/07

403.15 (New)

Proposed Change as Submitted:

Proponent: William M. Connolly, State of New Jersey, Dept. of Community Affairs, Division of Codes and Standards, representing the International Code Council Ad Hoc Committee on Terrorism Resistant Buildings

Add new text as follows:

<u>**403.15 Remoteness of exit stairway enclosures.** Exit stairway enclosures shall be located in different structural bays. The nearest wall of separate required exit stairway enclosures shall be placed a distance apart equal to not less than one-half of the length of the maximum overall diagonal dimension of the building or area to be served measured in a straight line between the nearest portion of the stairway enclosure. In buildings with three or more exit stairway enclosures, the exit stairway enclosures shall be placed a distance apart equal to not less than one-third of the length of the maximum overall diagonal dimension of the building or area to be served measured in a straight line between the nearest portion of the exit stairway enclosure. Scissor stairs shall be counted as one exit stairway.</u>

Reason: This code change proposal is one of fourteen proposals being submitted by the International Code Council Ad Hoc Committee on Terrorism Resistant Buildings.

The purpose of this change is to add a new Section 403.15 that will require stair shafts to meet remoteness criteria, in addition to the separation distance requirements for exit access doorways of Section 1015.2.

The Code has long contained requirements designed to ensure that all the exit access doors on a floor are not grouped closely together. Grouping exit access doors too closely defeats the whole point of multiple exits.

The National Institute of Standards and Technology's (NIST) report on the World Trade Center (WTC) tragedy recommends a new remoteness criterion for stair shafts (Recommendation 18). The report pointed out that, at some locations, stairs that met the exit access distance requirements were, nonetheless, very closely grouped. Their shafts were very close together and all three were destroyed by the airplane impact, thereby dooming all above. It is not the proponents' intent to make stair shafts immune to airplane attacks but the re-examination of our basic criteria that was prompted by the attack and the WTC Report suggests that far less dramatic events could render more than one stair shaft unusable. The cause need not be an act of terror either. There are other explosive hazards in high rise buildings. It is only prudent to separate the stair shafts themselves as well as the exit access doors.

It is possible that, in some high rise office buildings, this provision will result in one or more stairs being across the hall from the core rather than in the core. No additional floor area will be required for the sum total of core and stairs. If a stair is outside the traditional core, then the core itself will be smaller. Some might suggest that such a stair location might inhibit design flexibility in tenant spaces. This is simply not true. The architect might have to work a little harder to develop layouts but, with a little skill, any constraint can be incorporated into an acceptable design.

The proposal actually introduces two remoteness criteria. The first is a traditional standard based upon diagonal distances. The second requires that two stairs not be located in the same bay. This requirement correlates with two other changes submitted by the proponents. The proposed disproportionate collapse provisions of the proposed new Section 1605 and the proposed burnout without excessive collapse provisions of proposed new Section 403.15 both work to limit the extent of collapse. The structural bay aspect of this proposal is intended to exclude the possibility that two shafts might be in the same collapse zone.

The proposal requires the nearest points of two stair enclosures to be separated by a distance exceeding one-half the maximum overall diagonal dimension (one third in the case of buildings having three or more required stairs). The proposal also requires that multiple stair shafts not be located in the same bay for the reasons described above.

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

Cost Impact: The code change proposal will not increase construction costs. It merely deals with the location of building elements that are already required by the Code.

Committee Action: Disapproved

Committee Reason: The committee felt that review of the NIST report was not yet complete, therefore this proposal was premature. The term 'structural bay' was not defined. The standard 'structural bay' is not used in high rise construction. Justification was not provided for the significant change for the additional separation of exits, especially if the additional stairway in G71 is also required. The ½ of the diagonal dimension, in a standard plan with 3 or more stairways, would force the stairway enclosure out of the building footprint. An analysis of the architectural and engineering impact of this change must be performed.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

William M. Connolly, State of New Jersey, Dept. of Community Affairs, Division of Codes and Standards, representing the International Code Council Ad Hoc Committee on Terrorism Resistant Buildings, requests Approval as Modified by this public comment.

Modify proposal as follows:

403.15 Remoteness of exit stairway enclosures. ~~Exit stairway enclosures shall be located in different structural bays.~~ The nearest wall of separate required exit stairway enclosures shall be placed a distance apart equal to not less than one-half of the length of the maximum overall diagonal dimension of the building or area to be served measured in a straight line between the nearest portion of the stairway enclosure. In buildings with three or more exit stairway enclosures, <u>at least two of</u> the exit stairway enclosures shall be placed a distance apart equal to not less than ~~one-third~~ <u>one-half</u> of the length of the maximum overall diagonal dimension of the building or area to be served measured in a straight line between the nearest portion of the exit stairway enclosure. Scissor stairs shall be counted as one exit stairway.

Commenter's Reason: In the 2006 Report of the Public Hearing, it was stated that the committee disapproved this proposal for several reasons. This public comment satisfies these concerns. Firstly, the committee could not support the change because the term "structural bay" is not defined. This public comment deletes this requirement. Additionally, the committee felt that "in a standard plan with 3 or more stairways, this would force the stair enclosure out of the building footprint". To reduce the impact on buildings with three exits, this public comment eliminates the requirement for all exits to be remote and merely contains a requirement similar to Section 1015.2.2 and requires two of the exits to be remote.

Final Action: AS AM AMPC _______ D

G73-06/07
403.15 (New)

Proposed Change as Submitted:

Proponent: William M. Connolly, State of New Jersey, Dept. of Community Affairs, Division of Codes and Standards, representing the International Code Council Ad Hoc Committee on Terrorism Resistant Buildings

Add new text as follows:

<u>**403.15 Structural integrity of exit stairway enclosures.** For all buildings that are more than 420 feet (128 m) in height, exit stairway enclosure wall surfaces, from the top of the floor to the underside of the floor or roof above and connections to supporting members, shall be capable of resisting a static load expressed as a uniform pressure of not less than 2 pounds per square inch (psi) applied perpendicular to the exterior of the enclosure. This load need not be assumed to act concurrently with the loads specified in Chapter 16.</u>

Reason: This code change proposal is one of fourteen proposals being submitted by the International Code Council Ad Hoc Committee on Terrorism Resistant Buildings.

The purpose of this change is to establish a standard for the structural robustness of exit stairway enclosures. It implements Recommendation 18 of the National Institute of Standards and Technology (NIST) report on the World Trade Center (WTC) tragedy.

The Code has traditionally looked upon a stair enclosure as a place of relative safety. There are any number of carefully crafted code provisions designed to ensure that goal, but they are based upon only one hazard – fire. The enclosures of these stairs are made fire resistive through the traditional rating and listing system, but the Code does not establish a criterion for structural robustness. The proponents do not believe that the existing “hose stream” test addresses this issue. The hose stream does not and cannot represent the real world impact of blast loads that a stair shaft might encounter. Neither does the ongoing industry work designed to develop an impact resistance test standard. That work relates to durability rather than safety. The proponents believe that a structural standard is needed.

The stair enclosures of the WTC were destroyed by an aircraft impact. Far lesser events, such as a gas explosion or a vehicle impact (on lower floors) can destroy a stair enclosure, especially when one considers that the Code contains no structural criteria at all. Any structural robustness that existing stair shaft enclosures have is a by-product of the fire rating process; a process that was never intended to provide structural integrity.

A new criterion is needed for exit stair enclosures – a structural one.

The NIST WTC Report suggests a standard based upon resistance to over-pressure. This approach has two real advantages. It reflects one possible damage scenario and can represent others as well. Secondly, it is a performance standard. All materials can be analyzed and engineered to comply.

Compliance with this standard is determined by engineering analysis, not a test. This is a simple and direct approach that can be implemented immediately.

The requirement is expressed as a simple static load of 2 psi acting perpendicular to the shaft. The criterion is very similar to that already established for guardrails. It is expressed in the same way as the existing guardrail structural requirement so that the manner in which it is to be applied is clear. The proponents believe that traditional forms of enclosure, such as 8” full mortar bedded and reinforced CMU walls, will meet the requirement. There is no question that less traditional and more lightweight systems can be designed to meet it as well.

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

Cost Impact: The code change proposal will increase the cost of construction but the continued absence of structural criteria for exit stairway enclosures is not possible. This is a cost that must be met for safety’s sake.

Committee Action: Disapproved

Committee Reason: Based upon considerable testimony in opposition, indicating that there are many problems with the proposal that need resolution. There is no explanation given that justifies the proposed 2 psi loading on the walls of stair enclosures. It should be clarified whether that load is considered a strength or service level load. Why does the load apply only to the walls and not the supporting floors? This loading would result in much stiffer enclosure walls, that would be treated as shear walls since they can’t be isolated which in turn would adversely impact the design of some seismic force resisting systems.

Assembly Action: None

Individual Consideration Agenda

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

Public Comment:

William M. Connolly, State of New Jersey, Dept. of Community Affairs, Division of Codes and Standards, representing the International Code Council Ad Hoc Committee on Terrorism Resistant Buildings,

requests Approval as Modified by this public comment.

Modify proposal as follows:

403.15 Structural integrity of exit stairway enclosures. For all buildings that are more than 420 feet (128 m) in height, exit stairway enclosure wall surfaces, from the top of ~~the~~ each floor to the underside of the floor or roof above and connections to supporting members, shall be capable of resisting a ~~static~~ stress design load expressed as a uniform pressure of not less than 2 pounds per square inch (psi) applied perpendicular to the exterior of the enclosure. This load need not be assumed to act concurrently with the loads specified in Chapter 16 and shall be applied to one floor at a time.

Commenter's Reason: The committee disapproved this proposal for several reasons. One reason was that no justification was provided for the 2 psi load. The 2 psi load requirement is consistent with the overpressure associated with a gas explosion**.** NIST has performed an analysis to verify this statement. Additionally, the committee suggested that it be made clear whether this load is a strength load or a service load. This public comment modifies the proposal to make clear that this load is a stress design load. This public comment also clarifies that the load is applied to each floor, however, the load is applied one floor level at a time.

Final Action: AS AM AMPC _______ D

FS98-06/07

Table 601, 714.1, 714.1.1 (New), 714.1.2 (New), 714.2, 714.2.1, 714.2.2, 714.3, 714.4

Proposed Change as Submitted:

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

Revise as follows:

TABLE 601
FIRE-RESISTANCE RATING REQUIREMENTS FOR BUILDING ELEMENTS (hours)

BUILDING ELEMENT	TYPE I		TYPE II		TYPE III		TYPE IV	TYPE V	
	A	B	A[d]	B	A[d]	B	HT	A[d]	B
~~Structural~~ Primary structural frame[a] See Section 714.1.1 Including columns, girders, trusses	3[b]	2[b]	1	0	1	0	HT	1	0
Bearing walls Exterior[f] Interior	3 3[b]	2 2[b]	1 1	0 0	2 1	2 0	2 1/HT	1 1	0 0
Nonbearing walls and partitions Exterior	See Table 602								
Nonbearing walls and partitions Interior[e]	0	0	0	0	0	0	See Section 602.4.6	0	0
Floor construction Including supporting beams and joists	2	2	1	0	1	0	HT	1	0
Roof construction Including supporting beams and joists	1½[c]	1[c,d]	1[c,d]	0[c,d]	1[c,d]	0[c,d]	HT	1[c,d]	0

For SI: 1 foot = 304.8 mm.

~~a. The structural frame shall be considered to be the columns and the girders, beams, trusses and spandrels having direct connections to the columns and bracing members designed to carry gravity loads. The members of floor or roof panels which have no connection to the columns shall be considered secondary members and not a part of the structural frame.~~

b. through g. (No change to current text – re-letter to become a. through f.)

714.1 Requirements. The fire-resistance rating of structural members and assemblies shall comply with this section and the requirements for the type of construction as specified in Table 601 and shall not be less than the rating required for the fire-resistance-rated assemblies supported by the structural members.

Exception: Fire barriers, fire partitions and smoke barriers as provided in Sections 706.5, 708.4 and 709.4, respectively.

~~**714.2 Protection of structural members.** Protection of columns, girders, trusses, beams, lintels or other structural members that are required to have a fire-resistance rating shall comply with this section.~~

714.1.1 Primary structural frame. The primary structural frame shall be 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.

714.1.2 Secondary members. The members of floor or roof construction which are not connected to the columns shall be considered secondary members and not part of the primary structural frame

~~**714.2.1**~~ **714.2 Individual encasement protection.** ~~Columns, g~~Girders, trusses, beams, lintels or other structural members that are required to have a fire-resistance rating and that support more than two floors or one floor and roof, or support a load-bearing wall or a nonload-bearing wall more than two stories high, shall be individually protected on all sides for the full length, including connections to other structural members, with materials having the required fire-resistance rating.

714.2.1 Alternative protection. The structural members that are required to have a fire-resistance rating and are not required to be provided individual encasement protection according to Section 714.2 ~~Other structural members required to have a fire-resistance rating~~ shall be protected by individual encasement protection, by a membrane or ceiling protection as specified in Section 711, or by a combination of both. ~~Columns shall also comply with Section 714.2.2.~~

~~**714.2.1.**~~**1** **714.3 Membrane protection.** King studs and boundary elements that are integral elements in load-bearing walls of light-framed construction shall be permitted to have required fire-resistance ratings provided by the membrane protection provided for the load-bearing wall.

~~**714.2.2**~~ **714.4 Column protection** ~~**above ceilings.**~~ Where columns are required ~~a~~ to be fire-resistance ~~rating~~ rated, the entire column, including its connections to beams or girders, shall be ~~protected~~ provided individual encasement protection on all sides for the full column length. Where the column extends through a ceiling, the fire resistance rating of the column shall be continuous from the top of the foundation or floor/ceiling assembly below through the ceiling space to the top of the column.

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

This proposed change is a result of the CTC's investigation of the area of study entitled "Review of NIST WTC Recommendations". The scope of the activity is noted as:

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

This proposal is intended to address NIST recommendation 7. For this specific proposed change, CTC is working in cooperation with the NIBS/MMC Committee to Translate the NIST World Trade Center Investigation Recommendations for the Model Codes. The CTC notes in their investigation that many of the recommendations contained in the NIST report require additional information for the CTC to further investigate. As such, CTC intends to continue to study the other NIST recommendations.

NIST Recommendation #7 is summarized as "NIST recommends the adoption and use of the structural frame approach to fire resistance ratings." While the IBC currently contains this approach, the NIST team recommends that the concept be reinforced by incorporating text similar to that contained in Footnote a to Table 601 into the pertinent code text for a higher visibility and understanding by code users.

The proposed modification to line 1, column 1 of Table 601 is not intended to revise the intent but to incorporate the revised term. In lieu of a footnote, reliance on the reference to the specific code text of Section 714.1.1 enables a better understanding of the requirements for the pertinent building elements.

The modifications to the subsections of Section 714 are intended to retain the current intent. The assemblies for floors and roofs are not consistently referred to as "panels" and the apparent intent is to deal with "floor and roof construction".

The modifications to the several subsections of Section 714 are intended to work in concert with the reference from Table 601 and consolidate text into a more efficient format without a change in intent.

714.1 – The section is revised by incorporating the requirement that the fire-resistance rating of structural members is to comply with "this section" and "Table 601".

714.1.1 – Existing section 714.2 is not necessary and contains no particular requirements which are not contained in Section 714.1. The text of Section 714.1.1 was revised to more closely resemble the current terminology in line 1 and footnote a of Table 601 which is "structural frame". The incorporation of "other structural members" in Section 714.1.1 is to place reliance on the function of the member to determine its inclusion in the primary structural frame although a laundry list of commonly understood members is retained for understanding of the intent. The structural members named in the existing laundry list are included in the subsections which apply to such members. It should be noted that this section, as does the current footnote, does not consider the lateral load resisting system as part of the structural frame within the context of fire resistance ratings.

714.1.2 – This text is based on the second sentence of existing Footnote a to Table 601.

714.2 – The proposal utilizes the text and concept contained in existing Section 714.2.1. The inclusion of "encasement" in the section title is to enhance the focus of the section's intent. The proposed deletion of "columns" from the laundry list is to eliminate the implication that columns are not required to be individually protected to their full height when protected by Section 711 - Horizontal Assemblies. Individual protection for columns is required by existing Section 714.2.2. This is addressed in proposed Section 714.4. The connections of these elements to other structural members are required to be protected for the continuity of protection.

714.2.1 – The proposal is based on the text in the second sentence of the existing Section 714.2.1 and is addressing those structural members which are not required to be individually protected according to proposed Section 714.2. The last sentence of existing Section 714.2.1 is not needed as proposed Section 714.4 exclusively deals with columns.

714.2.2 – The proposal requires columns to be individually protected for the full column length and columns are not permitted to be protected by membrane protection.

Bibliography:
Interim Report No. 1 of the CTC, Area of Study – Review of NIST WTC Recommendations, March 9, 2006.
National Institute of Standards and Technology. Final Report of the National Construction Safety Team on the Collapses of the World Trade Center Towers. United States Government Printing Office: Washington, D.C. September 2005.

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

Committee Action: **Approved as Submitted**

Committee Reason: This helps to address a couple of concerns which were raised by the NIST report on issues related to the World Trade Center. This item was considered to help with the concerns that the structural frame be better defined and addressed so that the level of fire protection is easier to determine. Having these elements better defined helps to clarify the fire protection required for the structural frame and secondary members. It also helps to clarify that the floor is not considered as being a part of the structural frame. This proposal does not contain any technical changes to the requirements but appropriately moves the definition for structural frame from the table footnote into the proposed sections 714.1.1 and 714.1.2.

Assembly Action: **None**

Individual Consideration Agenda

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

Public Comment 1:

Philip Brazil, P.E., Reid Middleton, Inc., representing himself, requests Approval as Modified by this public comment.

Modify proposal as follows:

TABLE 601
FIRE-RESISTANCE RATING REQUIREMENTS FOR BUILDING ELEMENTS (hours)

BUILDING ELEMENT	TYPE I		TYPE II		TYPE III		TYPE IV	TYPE V	
	A	B	A[d]	B	A[d]	B	HT	A[d]	B
Primary structural frame[a] See Section 714.1.1 ~~Including columns, girders, trusses~~	3[b]	2[b]	1	0	1	0	HT	1	0

(Portions of proposed changes to table and footnotes not shown remain unchanged)

714.1 Requirements. The fire-resistance ratings of structural members and assemblies shall comply with this section and the requirements for the type of construction as specified in Table 601 ~~and~~. The fire-resistance ratings shall not be less than ~~the rating~~ required for the fire-resistance-rated assemblies supported by the structural members.

Exception: Fire barriers, fire partitions and smoke barriers as provided in Sections 706.5, 708.4 and 709.4, respectively.

714.1.1 Primary structural frame. The primary structural frame shall ~~be~~ consist of the following:

1. The columns ~~and other~~
2. Structural members ~~including the girders, beams, trusses and spandrels~~ having direct connections to the columns, including girders, beams, trusses, lintels and spandrels, and
3. Bracing members designed to carry gravity loads.

714.1.2 Secondary members. ~~The~~ Members of the floor ~~or~~ construction and roof construction ~~which~~ that are not connected to the columns, including structural members not having direction connections to the columns and bracing members not designed to carry gravity loads, shall be considered secondary members and not part of the primary structural frame.

714.2 Individual ~~encasement~~ protection. Girders, trusses, beams, lintels, spandrels and ~~or~~ other structural members ~~that are~~ required to have a fire-resistance rating and that support more than two floors or one floor and roof, or support a load-bearing wall or a nonload-bearing wall more than two stories high, shall be individually protected on all sides for the full length, including connections to other structural members, with materials having the required fire-resistance rating.

714.2.1 Alternative protection. The structural members that are required to have a fire-resistance rating and are not required to be ~~provided individual encasement protection according to~~ individually protected in accordance with Section 714.2 shall be protected ~~by individual encasement protection~~ in accordance with Section 714.2, by ~~a~~ the membrane or ceiling ~~protection as specified in~~ of a horizontal assembly in accordance with Section 711, or by a combination of both.

714.3 Membrane protection. King studs and boundary elements that are integral elements in load-bearing walls of light-framed construction shall be permitted to have required fire-resistance ratings provided by the membrane protection provided for the load-bearing wall.

714.4 Column protection. Where columns are required to be fire-resistance-rated, the entire column, including ~~its~~ connections to ~~beams or girders~~ other structural members, shall be ~~provided individual encasement protection~~ individually protected on all sides for the full column length. Where the column extends through a ceiling, the ~~fire resistance rating~~ individual protection of the column shall be continuous from the top of the foundation or floor/ceiling assembly below through the ceiling space to the top of the column.

Commenter's Reason: The purpose for this public comment is to make the proposal more technically sound. Reference to "columns, girders and trusses" in the item under "Building Element" for primary structural frame" at Table 601 is deleted because it is effectively replaced by the reference to Section 714.1.1 and conflicts with the references in Section 714.1.1 to columns, girders, beams, trusses and spandrels.

The revision to Section 714.1 is editorial. The revision to Section 714.1.1 may appear editorial but it is being done to make it clear which components of the structure are part of the structural frame. The current language implies that only girders, beams, trusses and spandrels having direct connections to the columns are part of the structural frame when the intent is that all structural members having direct connections to the columns are part of the structural frame. The listing of girders, beams, trusses and spandrels in Section 714.1.1 should be viewed as examples of such structural members. Note that "structural member" is not currently defined in the IBC.

Section 714.1.1 currently lists girders, beams, trusses and spandrels, but not lintels, as examples of structural members. Section 714.2 currently lists girders, beams, trusses and lintels, but not spandrels, as examples of structural members. The proposal correlates the lists for consistency. The listing of spandrels and lintels is, to a certain extent, superfluous but they should remain at least until a definition of "structural member" is added to the IBC by a future action of the membership.

Section 714.1.2 is revised because the current language does not make it clear whether structural members not having direction connections to the columns and bracing members not designed to carry gravity loads are members of the floor or roof construction such that they are considered secondary members. Note that horizontal bracing members typically are, but vertical bracing members typically are not, part of the floor or roof construction.

References to individual encasement protection in the remainder of the proposal are replaced with references to individual protection. Sections 714.2.1 and 714.4 reference individual encasement protection but the proposal does not contain technical provisions for it. The title of Section 714.2 is "individual encasement protection" but the provisions in the section do not mention it. Instead, individual protection on all sides of the structural member for its full length, including connections to other structural members, is specified. Referencing individual encasement protection without technical provisions for it amounts to referencing nothing at all. The language in Section 714.2 is technically sound and provides clear and understandable performance language to achieve effective fire-resistance-rated protection for structural members. Labeling it "individual encasement protection" would do nothing more than add a label to a requirement that is not in need of one.

Public Comment 2:

Maureen Traxler, City of Seattle Department of Planning and Development, requests Approval as Modified by this public comment.

Modify proposal as follows:

TABLE 601
FIRE-RESISTANCE RATING REQUIREMENTS FOR BUILDING ELEMENTS (hours)

	TYPE I		**TYPE II**		**TYPE III**		**TYPE IV**	**TYPE V**	
BUILDING ELEMENT	**A**	**B**	**A[d]**	**B**	**A[d]**	**B**	**HT**	**A[d]**	**B**
~~Primary~~ structural frame[a] See Section 714.1.1 Including columns, girders, trusses	3[b]	2[b]	1	0	1	0	HT	1	0

(Portions of proposed changes to table and footnotes not shown remain unchanged)

Add definition of "structural frame" to Section 702.

702 **~~714.1.1~~** **~~Primary~~** **structural frame.** The ~~primary~~ structural frame ~~shall be~~ is 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.

Delete "definitions" from Section 714.1.1 and 714.1.2.

~~**714.1.1 Primary structural frame.** The primary structural frame shall be is 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.~~

~~**714.1.2 Secondary members.** The members of floor or roof construction which are not connected to the columns shall be considered secondary members and not part of the primary structural frame.~~

(Portions of proposal not shown remain unchanged)

Commenter's Reason: Proposed section 714.1.1 contains no substantive requirements; it is a definition only and belongs in Section 702. The definition of "secondary members" is unnecessary because the term is not used in the code, and there is no longer a reason to distinguish between primary and secondary structural frame. The definition of secondary members is also superfluous because it merely says that members that don't fall within the definition of primary members are secondary.

Final Action: AS AM AMPC____ D