Code Technology Committee
Area of Study – NIST World Trade Center Recommendations

2007/2008 Cycle
Code changes related to the CTC area of study noted above

FILE 1 OF 2

The following are code changes related to the CTC NIST World Trade Center Recommendations Area of Study that will be considered at the 2007/2008 Code Development Hearings in Palm Springs, California. **Note that this is File 1 of 2 and includes IBC – Structural and IFC. File 2 includes IBC - Fire safety, General and Egress.**

S59–07/08
1604.11, 1605 (New)

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

Add new text follows:

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

SECTION 1605
DISPROPORTIONATE COLLAPSE

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

1605.2 DEFINITIONS.

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

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

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

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

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

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

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

TABLE 1605.3
BUILDING CLASS

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Where:
A = Horizontal cross sectional area of the column or wall including piers, but excluding the non-load-bearing width, if any of an external wall for cavity construction (ft).
ha = Clear height of a column or wall between restraining surfaces (ft).
t = Wall thickness or column dimension (ft).

TABLE 1605.6.1.6.1
MINIMUM PROPERTIES FOR MASONRY WALLS WITH VERTICAL TIES

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

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

TABLE 1605.6.1.6.3
REMOVAL OF DEFICIENT MASONRY VERTICAL TIE ELEMENTS

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

		For both wall types: clear height between lateral restraints.

a. Lateral supports shall be provided by the following:

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

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

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

TABLE 1605.6.1.8
ACCEPTABILITY CRITERIA AND SUBSEQUENT ACTION FOR MASONRY

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

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

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

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

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

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

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

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

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

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

TABLE 1605.6.1.8.1.8
DEFORMATION LIMITS FOR MASONRY

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

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

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

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

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

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

Where:

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

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

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

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

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

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

TABLE 1605.6.2.2
OVER-STRENGTH FACTORS FOR STRUCTURAL STEEL

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

TABLE 1605.6.2.4.1
ACCEPTABILITY CRITERIA AND SUBSEQUENT ACTION FOR STRUCTURAL STEEL

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

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

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

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

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

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

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

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

1. When the controlling action for the element is force controlled, evaluate the strength of the element using the interaction equations in Chapter H of AISC 360, incorporating the appropriate strength reduction factors Φ and the over-strength factor Ω. Remove the element from the model when the acceptability criteria is violated and redistribute the loads associated with the element in accordance with Section 1605.6.2.4.6.
2. When the controlling action for the element is deformation controlled, the element can be modeled for inelastic action using the modeling parameters for nonlinear procedures in Table 5-6 in FEMA 356. In linear analyses, take the force deformation characteristics of the elements as bilinear (elastic – perfectly plastic), ignoring the degrading portion of the relationship specified in FEMA 356. The modeling of plastic hinges for beam-columns in linear static analyses must include a reduction in the moment capacity due to the effect of

the axial force (in accordance with FEMA 356 Equation 5-4). For nonlinear analysis, the modeling of elements, panel zones, or connections must follow the guidelines in FEMA 356. Nonlinear analyses must utilize coupled (P-M-M) hinges that yield based on the interaction of axial force and bending moment. In no cases shall the deformation limits established in FEMA 356 exceed the deformation limits established in Table 1605.6.2.5(1).

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

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

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

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

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

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

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

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

TABLE 1605.6.2.5(1)
DEFORMATION LIMITS FOR STRUCTURAL STEEL

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

a. As defined in AISC 341.

TABLE 1605.6.2.5(2)
STEEL MOMENT FRAME CONNECTION TYPES

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

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

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

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

TABLE 1605.6.3.1
ACCEPTABILITY CRITERIA AND SUBSEQUENT ACTION FOR REINFORCED CONCRETE

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

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

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

TABLE 1605.6.3.1.1
OVER-STRENGTH FACTORS FOR REINFORCED CONCRETE

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

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

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

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

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

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

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

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

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

TABLE 1605.6.3.1.6
DEFORMATION LIMITS FOR REINFORCED CONCRETE

	CLASS 2 & 3 BUILDINGS		**CLASS 4 BUILDINGS**	
Component	**Ductility** υ	**Rotation,** Degrees θ	**Ductility** υ	**Rotation,** Degrees θ
Slab and Beam Without Tension Membrane[a]				
Single-Reinforced or Double-Reinforced without Shear Reinforcing[b]	-	3	-	2
Double-Reinforced with Shear Reinforcing[c]	-	6	-	4
Slab and Beam with Tension Membrane[a]				
Normal Proportions ($L/h \geq 5$)	-	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. The following procedure shall be met for Nonlinear Dynamic, and Linear or Nonlinear Static Analysis.

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

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

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

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

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

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

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

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

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

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

The need for such standards has been debated for years in the technical community. That debate has resulted in little but inaction.

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

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

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

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

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

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

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

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

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

Bibliography:
American Concrete Institute (ACI) Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05). American Concrete Institute: Farmington Hills, Michigan, 2005.
American Society of Civil Engineers (ASCE) 7-2005, Minimum Design Loads for Buildings and Other Structures. American Society of Civil of Engineers: Danvers, Massachusetts, 2005.
General Services Administration. Facilities Standards for the Public Building Service, Chapter 8, Security Design. 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 code change proposal will increase the cost of construction.

++

S81-07/08

1609.1.1, 1609.1.1.2, Chapter 35 (New)

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

1. Revise as follows:

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

Exceptions:

1. Subject to the limitations of Section 1609.1.1.1, the provisions of SBCCI SSTD 10 shall be permitted for applicable Group R-2 and R-3 buildings.
2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of the AF&PA WFCM.
3. Designs using NAAMM FP 1001.
4. Designs using TIA/EIA-222 for antenna-supporting structures and antennas.
5. Wind tunnel tests in accordance with Section 6.6 of ASCE 7, subject to the limitations in Section 1609.1.1.2.
6. Wind tunnel tests in accordance with ASCE/SEI 49, subject to the limitations in Section 1609.1.1.2.

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

2. Add standard to Chapter 35 as follows:

American Society of Civil Engineers/Structural Engineering Institute
ASCE/SEI 49-07 Wind Tunnel Testing for Buildings and Other Structures

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

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

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

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

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

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

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

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

References:

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

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

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

Analysis: A review of the standard(s) proposed for inclusion in the code, ASCE/SEI 49, for compliance with ICC criteria for referenced standards given in Section 3.6 of Council Policy #CP 28 will be posted on the ICC website on or before January 15, 2008.

++

S101 –07/08

1614 (New)

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

Add new section as follows:

SECTION 1614
STRUCTURAL INTEGRITY

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

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

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

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

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

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

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

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

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

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

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

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

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

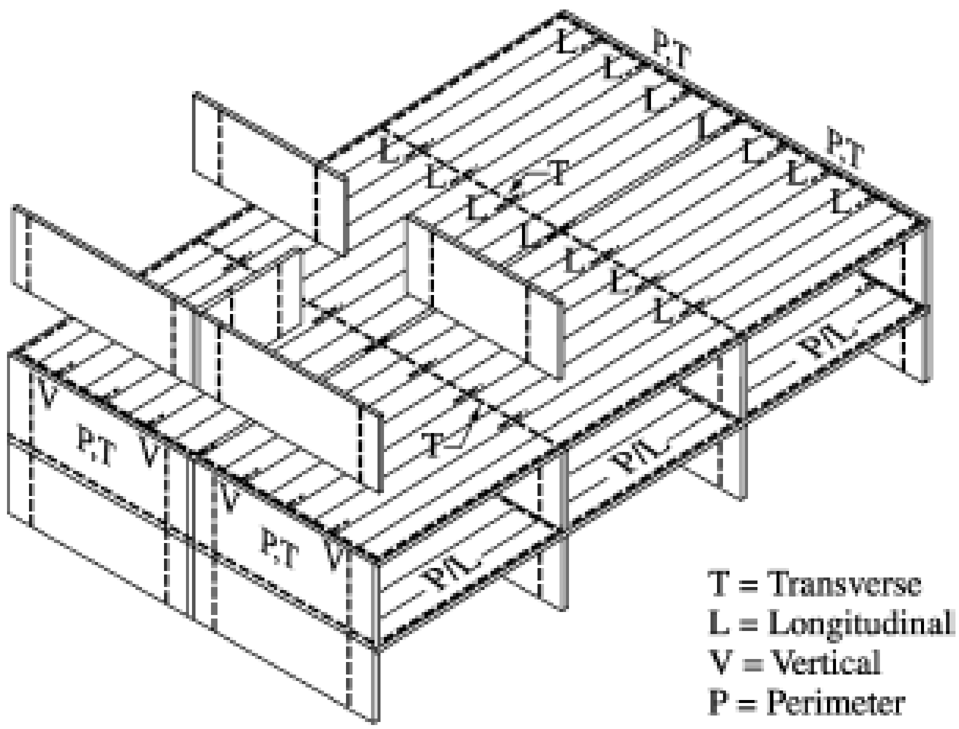

FIGURE 1614.4
LONGITUDINAL, PERIMETER, TRANSVERSE AND VERTICAL TIES

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

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

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

$$T_T = wLs \le \alpha_T s$$ **(Equation 16-45)**

where:

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

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

S = the spacing between ties, ft (m)

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

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

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

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

For SI:

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

where

w = as defined in Section 1614.4.2.1

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

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

Reason: This proposal was developed by a broad industry coalition that includes participation by the National Council of Structural Engineers Associations, the Structural Engineering Institute of the American Society of Civil Engineers, the American Institute of Architects, the American Concrete Institute, the American Forest & Paper Association, the American Iron and Steel Institute, the American Institute of Steel Construction, the Masonry Alliance for Codes and Standards, The Masonry Society, the Portland Cement Association, the Steel Joist Institute, the Precast/Prestressed Concrete Institute. Corresponding members included the International Code Council and the National Fire Protection Association. In addition, there was nonvoting participation by the National Institute of Building Sciences and the National Institute of Standards and Technology.

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

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

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

++

F84–07/08

509.1 (IBC [F] 911.1)

Proponent: Ken Kraus, Fire Department, Los Angeles, CA

Revise as follows:

509.1 (IBC [F] 911.1) (Supp) Features. Where required by other sections of this code and in all buildings classified as high-rise buildings by the *International Building Code*, a fire command center for fire department operations shall be provided. The location and accessibility of the fire command center shall be approved by the fire department. The fire command center shall be separated from the remainder of the building by not less than a 1-hour fire barrier constructed in accordance with Section 706 of the *International Building Code* or horizontal assembly constructed in accordance with Section 711 of the *International Building Code*, or both. The room shall be a minimum of ~~96~~ <u>250</u> square feet (~~9~~ <u>23</u> m^2) with a minimum dimension of ~~8~~ <u>10</u> feet (~~2438~~ <u>3048</u> mm). A layout of the fire command center and all features required by this section to be contained therein shall be submitted for approval prior to installation. The fire command center shall comply with NFPA 72 and shall contain the following features:

1. The emergency voice/alarm communication system unit.
2. The fire department communications system.
3. Fire-detection and alarm system annunciator system.
4. Annunciator <u>unit</u> visually indicating the location of the elevators and whether they are operational.
5. Status indicators and controls for air-handling systems.
6. The fire-fighters control panel required by Section 909.16 for smoke control systems installed in the building.
7. Controls for unlocking stairway doors simultaneously.
8. Sprinkler valve and water-flow detector display panels.
9. Emergency and standby power status indicators.
10. A telephone for fire department use with controlled access to the public telephone system.
11. Fire pump status indicators.
12. Schematic building plans indicating the typical floor plan and detailing the building core, means of egress, fire protection systems, fire-fighting equipment and fire department access.
13. Work table.
14. Generator supervision devices, manual start and transfer features.
15. Public address system, where specifically required by other sections of this code.
16. Elevator fire recall switch in accordance with ASME A17.1.
17. Elevator emergency or standby power selector switch(es), where emergency or standby power is provided.

Reason: This proposal is intended to increase the minimum size of the Fire Command Center to a size and configuration that is conducive to effective use of the facility by emergency responders.

The current minimum requirement for the size of a Fire Command Center is impractical. Fire Command Centers (FCC) not only need to be designed to accommodate a significant number of emergency responders wearing full personel protective equipment. FCC's are also used to review building emergency plans during incidents, co-locate decision makers within the Incident Command System (ICS) and interpret fire protection system information. Given the multiple uses of the FCC, it is extremely likely that the limitations of a 10'by 10' room would serve to compromise the effectiveness of Incident management.

The current minimum size has proven in both exercise and emergency incident scenarios to be too small and confining.

A minimum size of 250 square feet allows for the necessary personnel to effectively perform the required tasks associated with a Fire Command Center.

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

++

F85–07/08
509.1 (IBC [F] 911.1)

Proponent: Lawrence G. Perry, AIA, representing Building Owners and Managers Association International (BOMA)

Revise as follows:

509.1 (IBC [F] 911.1) (Supp) Features. Where required by other sections of this code and in all buildings classified as high-rise buildings by the *International Building Code*, a fire command center for fire department operations shall be provided. The location and accessibility of the fire command center shall be approved by the fire department. The fire command center shall be separated from the remainder of the building by not less than a 1-hour fire barrier constructed in accordance with Section 706 of the *International Building Code* or horizontal assembly constructed in accordance with Section 711 of the *International Building Code*, or both. The room shall be a minimum of 96 square feet (9 m2) with a minimum dimension of 8 feet (2438 mm). A layout of the fire command center and all features required by this section to be contained therein shall be submitted for approval prior to installation. The fire command center shall comply with NFPA72 and shall contain the following features:

1. The emergency voice/alarm communication system unit.
2. The fire department communications system.
3. Fire-detection and alarm system annunciator system.
4. Annunciator visually indicating the location of the elevators and whether they are operational.
5. Status indicators and controls for air-handling systems.
6. The fire-fighter=s control panel required by Section 909.16 for smoke control systems installed in the building.
7. Controls for unlocking stairway doors simultaneously.
8. Sprinkler valve and water-flow detector display panels.
9. Emergency and standby power status indicators.
10. A telephone for fire department use with controlled access to the public telephone system.
11. Fire pump status indicators.
12. Schematic building plans indicating the typical floor plan and detailing the building core, means of egress, fire protection systems, fire-fighting equipment and fire department access<u>, and the location of fire walls, fire barriers, fire partitions, smoke barriers and smoke partitions</u>.
13. Work table.
14. Generator supervision devices, manual start and transfer features.
15. Public address system, where specifically required by other sections of this code.
16. Elevator fire recall switch in accordance with ASME A17.1.
17. Elevator emergency or standby power selector switch(es), where emergency or standby power is provided.

Reason: This proposal will add additional information to first responders in buildings having fire command centers. It will require that the schematic building plans, which are already required, include the location of fire walls, fire barriers, fire partitions, smoke barriers, and smoke partitions. BOMA believes this is a far better method of providing this information to fire inspectors and responding fire fighters than providing stencils or stickers on walls throughout the building. BOMA has submitted a separate proposal to delete the requirement for marking of rated walls (newly added to Section 703.6 of the IBC).

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

F86–07/08
509.1, 509.2 (New) [IBC [F] 911.1, [F] 911.2 (New)]

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

1. Revise as follows:

509.1 (IBC [F] 911.1) (Supp) Features. Where required by other sections of this code and in all buildings classified as high-rise buildings by the *International Building Code*, ~~a fire~~ <u>an emergency</u> command center for ~~fire department~~ <u>emergency</u> operations shall be provided. The location and accessibility of the ~~fire~~ <u>emergency</u> command center shall be approved by the fire department. The ~~fire~~ emergency command center shall be separated from the remainder of the building by not less than a 1-hour fire barrier constructed in accordance with Section 706 of the *International Building Code* or horizontal assembly constructed in accordance with Section 711 of the *International Building Code*, or both. In buildings that are more than 420 feet (128 m) in height, the emergency command center shall be separated from the remainder of the building by not less than a 2-hour fire-resistance-rated fire barrier constructed in accordance with Section 706 of the *International Building Code* or 2-hour horizontal assembly constructed in accordance with Section 711 of the *International Building Code,* or both. The room shall be a minimum of 96 square feet (9 m^2) with a minimum dimension of 8 feet (2438 mm). A layout of the ~~fire~~ emergency command center and all features required by this section to be contained therein shall be submitted for approval prior to installation. The ~~fire~~ emergency command center shall comply with NFPA 72 and shall contain the following features:

1. The emergency voice/alarm communication system unit.
2. The fire department communications system.
3. Fire-detection and alarm system annunciator system.
4. Annunciator visually indicating the location of the elevators and whether they are operational.
5. Status indicators and controls for air-handling systems.
6. The fire-fighter=s control panel required by Section 909.16 for smoke control systems installed in the building.
7. Controls for unlocking stairway doors simultaneously.
8. Sprinkler valve and water-flow detector display panels.
9. Emergency and standby power status indicators.
10. A telephone for fire department use with controlled access to the public telephone system.
11. Fire pump status indicators.
12. ~~Schematic~~ Building Emergency resource manual approved by the fire department that includes emergency operation instructions and building plans indicating the typical floor plan and detailing the building core, means of egress, as well as the layout and operating instructions for the emergency aspects of fire protection systems, HVAC systems, elevator controls, communication systems, utilities, fire-fighting equipment and fire department access.
13. Work table.
14. Generator supervision devices, manual start and transfer features.
15. Public address system, where specifically required by other sections of this code.
16. Elevator fire recall switch in accordance with ASME A17.1.
17. Elevator emergency or standby power selector switch(es), where emergency or standby power is provided.

2. Add new text as follows:

509.2 (IBC [F] 911.2) Location. The emergency command center shall be located at least 25 feet from uncontrolled building entrances and loading docks, shall not be visible from the street, and shall be at a location approved by the fire chief.

Reason: The purpose of this change is to increase the ability of firefighters, and other emergency responders, to develop a clear picture of conditions throughout the building which will enable them to better manage evacuation, fire suppression and other emergency response activities. It will also enhance the safety of emergency responders, in buildings greater than 420 feet in height, by requiring a two-hour fire resistance rated enclosure for the emergency command center, the same as is required for the exit stair enclosure.

The value of the fire control center already required by the Code is enhanced by a strengthened "Emergency Resource Manual" which will now include operating instructions for emergency systems as well as information on the emergency aspects of HVAC systems, elevator controls, communication systems and utilities. The center is re-titled the emergency command center to reflect its role in managing emergencies other than fire emergencies.

New Section 509.2 will establish a minimum distance the command center must be located from any uncontrolled building entrance or loading dock, thus reducing the possibility of access or damage to the command center from outside influences.

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

++

F87–07/08

511 (New), 907.2.12.2 (IBC [F] 907.2.12.2), Appendix I (New)

Proponent: Tom Lariviere, Fire Department, Madison, MS, representing Joint Fire Service Review Committee

1. Add new text as follows:

SECTION 511
EMERGENCY RESPONDER RADIO COVERAGE

511.1 Emergency responder radio coverage in new buildings. All new buildings shall have approved radio coverage for emergency responders within the building.

511.2 Emergency responder radio coverage in existing buildings. Existing buildings that do not have approved radio coverage for emergency responders within the building shall be equipped with such coverage within 18 months of receiving notice of such deficiency from the fire code official.

2. Revise as follows:

907.2.12.2 (IBC [F] 907.2.12.2) (Supp) Fire department communication system. An approved ~~two-way, fire department communication~~ emergency responder radio coverage system ~~designed and installed in accordance with NFPA 72~~ shall be provided for fire department use. It shall operate between a fire command center complying with Section 509 and elevators, elevator lobbies, emergency and standby power rooms, fire pump rooms, areas of refuge and inside enclosed exit stairways. ~~The fire department communication device shall be provided at each floor level within the enclosed exit stairway.~~

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

3. Add new appendix as follows:

APPENDIX I
EMERGENCY RESPONDER RADIO COVERAGE

SECTION I101
GENERAL

I101 Scope. Systems, components, and equipment required to provide emergency responder radio coverage shall be in accordance with this appendix.

I101.1 Permit. A construction permit is required for installation of or modification to emergency responder radio coverage systems and related equipment. Maintenance performed in accordance with this code is not considered a modification and does not require a permit.

SECTION I102
DEFINITIONS

I102.1 Definitions. For the purpose of this appendix, certain terms are defined as follows:

AGENCY. Any emergency responder department within the jurisdiction that utilizes radio frequencies for communication. This could include, but not be limited to, various public safety agencies such as fire department, emergency medical services and law enforcement.

SECTION I103
TECHNICAL REQUIREMENTS

I103.1 Radio signal strength. The building shall be considered to have acceptable emergency responder radio coverage when signal strength measurements in 90 percent of all areas on each floor of the building meet the signal strength requirements in Sections I103.1.1 and I103.1.2.

I103.1.1 Minimum signal strength into the building. A minimum signal strength of -95 dBm shall be receivable within the building.

I103.1.2 Minimum signal strength out of the building. A minimum signal strength of -100 dBm shall be received by the agency's radio system when transmitted from within the building.

I103.2 System design. The emergency responder radio coverage system shall be designed in accordance with Sections I103.2.1 through I103.2.5.

I103.2.1 Amplification Systems Allowed. Buildings and structures which cannot support the required level of radio coverage shall be equipped with a radiating cable system, a distributed antenna system with FCC certified signal boosters, or other system approved by the fire code official in order to achieve the required adequate radio coverage.

I103.2.2 Technical criteria. The fire code official shall maintain a document providing the specific technical information and requirements for the emergency responder radio coverage system. This document shall contain, but not be limited to, the various frequencies required, the location of radio sites, effective radiated power of radio sites, and other supporting technical information.

I103.2.3 Secondary power. The emergency responder radio coverage system shall be equipped with a secondary source of power. The secondary source of power shall be either a battery system or an emergency generator. The secondary power supply shall supply power automatically when the primary power source is lost. The secondary source of power shall be capable of operating the emergency responder radio coverage system for a period of at least twelve hours.

I103.2.3.1 Battery Systems. The active components of the installed system or systems shall be capable of operating on an independent battery system for a period of at least twelve hours without external power input. The battery system shall automatically charge in the presence of external power input.

I103.2.4 Signal Booster requirements. If used, signal boosters shall meet the following requirements:

1. All signal booster components shall be contained in a NEMA4 type water proof cabinet.
2. The battery system shall be contained in a NEMA4 type water proof cabinet.
3. The system shall include automatic alarming of malfunctions of the signal booster and battery system. Any resulting trouble alarm shall be automatically transmitted to an approved central station or proprietary supervising station as defined in NFPA 72 or, when approved by the fire code official, shall sound an audible signal at a constantly attended location.
4. Equipment shall have FCC Certification prior to installation.

I103.2.5 Additional frequencies and change of frequencies. The emergency responder radio coverage system shall be capable of modification or expansion in the event frequency changes are required by the FCC or additional frequencies are made available by the FCC.
I103.3 Installation requirements. The installation of the public safety radio coverage system shall be in accordance with Sections I103.3.1 through I103.3.5.

I103.3.1 Approval prior to installation. No amplification system capable of operating on frequencies licensed to any public safety agency by the FCC shall be installed without prior coordination and approval of the fire code official.

I103.3.2 Permit required. A construction permit as required by Section 105.7.11 shall be obtained prior to the installation of the emergency responder radio coverage system.

I103.3.3 Minimum qualifications of personnel. The minimum qualifications of the system designer and lead installation personnel shall include:

1. A Valid FCC issued General Radio Operators License, and
2. Certification of in-building system training issued by a nationally recognized organization, school or a certificate issued by the manufacturer of the equipment being installed.

The agency may waive these requirements upon successful demonstration of adequate skills and experience satisfactory to the fire code official.

I103.3.4 Acceptance test procedure. When an emergency responder radio coverage system is required, and upon completion of installation, the building owner shall have the radio system tested to ensure that two-way coverage on each floor of the building is a minimum of 90 percent. The test procedure shall be conducted as follows:

1. Each floor of the building shall be divided into a grid of 20 approximately equal areas.
2. The test shall be conducted using a calibrated portable radio of the latest brand and model used by the agency talking through the agency's radio communications system.
3. A maximum of two nonadjacent areas will be allowed to fail the test.
4. In the event that three of the areas fail the test, in order to be more statistically accurate, the floor may be divided into 40 equal areas. A maximum of four nonadjacent areas will be allowed to fail the test. If the system fails the 40-area test, the system shall be altered to meet the 90 percent coverage requirement.
5. A test location approximately in the center of each grid area will be selected for the test, then the radio will be enabled to verify two-way communications to and from the outside of the building through the public agency's radio communications system. Once the test location has been selected, that location shall represent the entire area. If the test fails in the selected test location, that grid area shall fail, and prospecting for a better spot within the grid area will not be allowed.
6. The gain values of all amplifiers shall be measured and the test measurement results shall be kept on file with the building owner so that the measurements can be verified during annual tests. In the event that the measurement results become lost, the building owner will be required to rerun the acceptance test to reestablish the gain values.
7. As part of the installation a spectrum analyzer or other suitable test equipment shall be utilized to insure spurious oscillations are not being generated by the subject signal booster. This test will be conducted at time of installation and subsequent annual inspections.

I103.3.5 FCC compliance. The emergency responder radio coverage system installation and components shall also comply with all applicable Federal regulations, including but not limited to, Federal Communications Rules (47 CFR 90.219).

I103.4 Maintenance. The emergency responder radio coverage system shall be maintained in accordance with Sections I103.4.1 through I103.4.5.

I103.4.1 Maintenance. The public radio coverage system shall be maintained operational at all times.

I103.4.2 Permit required. A permit as required by Section 105.7.4 shall be obtained prior to the modification or alteration of the emergency responder radio coverage system.

I103.4.3 Testing and proof of compliance. The emergency responder radio coverage system shall be inspected and tested annually or whenever structural changes occur including additions or remodels that could materially change the original field performance tests. Testing shall consist of the following:

1. In-building coverage test as described in Section I103.3.4.
2. Signal boosters shall be tested to ensure that the gain is the same as it was upon initial installation and acceptance.
3. Backup batteries and power supplies shall be tested under load of a period of one hour to verify that they will properly operate during an actual power outage. If within the one hour test period the battery exhibits symptoms of failure, the test shall be extended for additional one hour periods until the integrity of the battery can be determined.
4. All other active components shall be checked to verify operation within the manufacturer's specifications.
5. At the conclusion of the testing a report shall be submitted to the fire code official which shall verify compliance with Section I103.3.4.

I103.4.4 Additional frequencies. The building owner shall modify or expand the emergency responder radio coverage system at their expense in the event frequency changes are required by the FCC or additional frequencies are made available by the FCC. Prior approval of a public safety radio coverage system on previous frequencies does not exempt this section.

I103.4.5 Field testing. Agency personnel shall have the right to enter onto the property at any reasonable time to

<u>conduct field-testing to verify the required level of radio coverage.</u>

Reason: Large buildings have historically provided barriers to radio communications within them. This is the reason high rise buildings are required to install hard wired, two-way communications systems. The typical system has phone jacks strategically located throughout the building (in stairways, elevator lobbies, and inside elevators), with hand sets available to emergency responders in the lobby or the fire control room.
However, problems with this solution include:

- Handset availability – even if they don't get stolen or misplaced, the typical building will only have five handsets, far too few for the dozens to hundreds of firefighters required to successfully bring a high rise fire under control
- Lack of training for responders – while some fire departments routinely train on these systems, each one is different, presenting problems remembering the special considerations necessary to operate successfully in each high rise building; other responders (law enforcement, EMS) don't train on these systems at all, and many times don't even know they exist
- Buildings other than high-rise interfere with routine radio communications, but aren't required to provide an alternative.

When this requirement was implemented, it was the best alternative available. Now, technology has progressed to a point where there are multiple solutions with multiple technologies to address virtually any situation. These solutions support emergency responders' radio systems so that no additional training is required by the responders; the same communication system that they use every day can be used in any building in a jurisdiction.

Emergency response agencies use radio communications routinely and lives depend on the adequacy of the radio communication system. Communications must be able to go both into and out of the buildings in times of emergency. Whether it be someone inside the building requesting assistance, or even worse calling May Day, or the Incident Commander outside the building trying to obtain a status report to make a determination on deployment of additional resources, communications is critical.

Some will complain of the cost of these systems, which range from the relatively inexpensive to very expensive, depending upon the solution chosen by the building owner or developer (one estimate is from $.40/ft to $1.25/ft). The fact is that tax payers have invested billions of dollars in their public safety communications systems. It isn't unusual for a mid-size jurisdiction to spend millions of dollars to equip emergency responders with communications systems, only to have a developer construct a building that defeats the entire system inside their facility. Good public policy dictates that these owners/developers bear the cost of upgrading their facilities to allow emergency responders to utilize the tools that tax payers have provided. This is in keeping with the philosophy inherent in the I-Codes that, when a facility grows too large or complex for effective fire response, that fire protection features be provided within the building at the owner's expense.

This proposal provides that an adequate level of communication is available within the building. Once a deficiency is noted in a building, the installation and technical criteria in Appendix I can be utilized to design and install a system to enhance the radio communications. There are several types of systems that can be utilized to enhance radio traffic and under this proposal any of these systems can be used.

This proposal also includes existing buildings in Section 511.2. While modeling and other techniques may provide a good prediction as to whether a building will interfere with radio communications, the reality is that it is unknown if a building will need to install any type of radio system enhancements until after the building is constructed. These issues are dependent on the construction type, shadows of other buildings, size of structure, etc. This proposal includes existing structures so that once the building is built, the system can be installed at any time, when and if it becomes necessary; it also provides a reasonable amount of time for existing buildings to come into conformance (18 months after notification).

The proposed Appendix I includes design, construction, maintenance and testing criteria. This provides guidance to the code official and ensures that the emergency responder radio coverage system will be operational throughout the life of the building.

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

++

F95–07/08

607.3 (New)

Proponent: Ed Donoghue, Edward Donoghue Associates, Inc.

Add new text as follows:

<u>**607.3 Fire service access elevator lobbies.** Where fire service access elevators are required by Section 3007 of the *International Building Code,* fire service access elevator lobbies shall be maintained free of storage and furnishings.</u>

Reason: In this specific proposal the focus is upon storage and furnishings within the fire service access elevator lobby. The fire service access elevator in high rise buildings over 120 feet above fire department vehicle access is a tool for fire fighters to enhance their ability to gain access to and undertake necessary staging activities in. Therefore, any obstructions located in lobbies associated with such elevators in the form of storage or furnishings, whether combustible or non-combustible, could hamper their ability to fully use such features. Prohibiting storage and furnishings in fire service access elevators also eliminates potential fire loads in such areas.

Background: As a result of the September 11, 2001 attacks on the World Trade Center, code provisions for emergency egress from tall buildings are being re-examined. There is renewed interest in the use of elevators for both occupant egress and fire fighters access. Therefore a Workshop on the Use of Elevators in Fires and Other Emergencies was held March 2-4, 2004, in Atlanta, Georgia. The workshop was cosponsored by American Society of Mechanical Engineers (ASME International), National Institute of Standards and Technology (NIST), International Code Council (ICC), National Fire Protection Association (NFPA), U.S. Access Board, and the International Association of Fire Fighters (IAFF).

The workshop focused on two general topics:

1. Use of Elevators by Fire fighters and
2. Use of Elevators by Occupants during Emergencies

To follow up on the ideas generated at the workshop, 2 task groups were formed; one for each topic. Their goals are:

- Review the suggestions from the Workshop on the Use of Elevators in Fires and other Emergencies.
- Develop a prioritized list of issues.
- Conduct a hazard analysis of the prioritized list of issues to see if there are any residual hazards.
- Draft code revisions for those issues that survive the process and the task group members still want addressed.

The membership of these task groups is broad and includes representatives from the elevator industry and manufacturers of devices such as fire alarms, the fire service, model codes and standards development organizations, and the accessibility community as well as fire protection engineers, architects and specialists in human factors and behavior. Since February 2005 the groups have each been conducting a hazard analysis on their assigned topic. The results of the hazard analysis focused upon the fire fighter needs is nearing completion.

The task group studied 16 different cases. In these cases a particular hazard followed by a cause/trigger was reviewed. The result of the hazard interacting with cause/trigger events may create a particular incident/effect. To address possible incident/effects corrective actions are proposed. Such corrective actions are then reviewed to see if they create any residual hazards. The hazard analysis then carries out each of the residual hazards with additional corrective actions until the hazard is mitigated. It is strictly a hazard analysis (i.e. not probabilistic) and certain assumptions were made such as a single fire start in a high rise building.

The code changes generated by this analysis are related both to the summary of corrective actions resulting from the hazard analysis and the existing language related to fire service access elevators placed into the 2007 supplement.

These proposals will work with the 2007 supplement requirements for fire service access elevators to address these concerns. It should be noted that the hazard analysis assumed a lobby to be directly connected with the fire service access elevator thus making the result of the analysis consistent with the philosophical approach found in the 2007 Supplement.

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

++

F171–07/08

907.2.12.2, 907.2.12.2.1 (New) [IBC [F] 907.2.12.2, [F] 907.2.12.2.1 (New)]; 907.3.5 (New), 907.3.5.1 (New)

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

1. Revise as follows:

907.2.12.2 (IBC [F] 907.2.12.2) (Supp) Fire department communication system. An approved two-way, fire department communication system designed and installed in accordance with ~~NFPA 72~~ this section shall be provided for fire department use in buildings classified as high rise buildings or underground buildings, in accordance with Section 403.1 or Section 405.1 respectively, of the *International Building Code* and in buildings that contain Group A occupancies with an occupant load of more than 1,000. ~~It shall operate between a fire command center complying with Section 509 and elevators, elevator lobbies, emergency and standby power rooms, fire pump rooms, areas of refuge and inside enclosed exit stairways. The fire department communication device shall be provided at each floor level within the enclosed exit stairway.~~

Exceptions: ~~Fire department radio systems where approved by the fire department.~~

1. Where approved by the building code official and the fire code official, a wired communication system shall be permitted to be installed or maintained in lieu of a bi-directional amplifier system. Wired fire department communications systems shall be designed and installed in accordance with NFPA 72 and shall operate between a fire command center complying with Section 509, elevators, elevator lobbies, emergency and standby power rooms, fire pump rooms, areas of refuge and inside enclosed exit stairways. The fire department communication device shall be provided at each floor level within the enclosed exit stairway.

2. Where it is determined by the fire code official that amplification of the fire department communication systems is not needed.

2. Add new text as follows:

907.2.12.2.1 (IBC [F] 907.2.12.2.1) System operation. The fire department communication system shall operate between the exterior of the building, a fire command center complying with Section 509 where required or provided, and internal areas of the building. The bi-directional amplifier system shall be compatible with the fire department radio communication system and shall be designed to provide 95% radio coverage based on a 3 watt portable radio nominally located at waist level of a standing or walking adult.

907.3.5 Fire department communications in existing buildings. An approved two-way, fire department communication system designed in accordance with Section 907.2.12.2 shall be provided for fire department use in existing buildings classified as high rise buildings or underground buildings, in accordance with Section 403.1 or Section 405.1 respectively, of the *International Building Code* and in existing buildings that contain Group A occupancies with an occupant load of more than 1,000.

Exceptions:

1. Where approved by the building code official and the fire code official, a wired communication system shall be permitted to be installed or maintained in lieu of a bi-directional amplifier system. Wired fire department communications systems shall be designed and installed in accordance with NFPA 72 and shall operate between a fire command center complying with Section 509, elevators, elevator lobbies, emergency and standby power rooms, fire pump rooms, areas of refuge and inside enclosed exit stairways. The fire department communication device shall be provided at each floor level within the enclosed exit stairway.
2. Where it is determined by the fire code official that amplification of the fire department communication systems is not needed.

907.3.5.1 Timing of Installation. Wherever existing wired communication systems cannot be repaired, they shall be replaced with a compliant bi-directional amplifier system. All existing high-rise buildings shall be made compliant within a time frame established by the adopting authority.

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

This proposed change is a 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 24. NIST Recommendation 24 recommends the establishment and implementation of codes and protocols for ensuring effective operation of the command and control system for large-scale building emergencies.

Due to firefighter safety concerns, fire departments are distributing radios to a larger percentage of emergency response personnel and more are using trunked radio systems which allow better control of radio frequency usage and better monitoring of emergency radio traffic. Wired on-site communications systems cannot be monitored by communications and dispatch personnel and have the potential for missed emergency communications and other essential communications between incident command personnel and on-scene operations personnel. The use of radio amplification systems will allow emergency services personnel to communicate properly throughout the building during an emergency. The revised exception gives authority to any jurisdiction when there is a need to retain wired communications systems or when the code officials believe the use of bi-directional amplifiers is not in the best interest of the emergency service departments serving the jurisdiction.

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

F204–07/08
915 (New), 902.1 (New) [IBC [F] 913 (New), 902.1 (New)]

Proponent: Scott L. Poster, Fire Department, Los Angeles County, CA

1. Add new section as follows:

SECTION 915 (IBC [F] 913)
PUBLIC SAFETY RADIO AMPLIFICATION SYSTEMS

915.1 General. The purpose of this section is to provide a regulatory frame work for the purpose of providing effective radio systems coverage for police and fire emergency services. No person shall maintain, own, erect, or construct, any building or structure or any part thereof, or cause the same to be done which fails to support adequate radio coverage for emergency service workers, including but not limited to firefighters, sheriffs, and local police officers.

Exceptions. This section shall not apply to the following:

1. Existing structures
2. Elevators
3. Group R occupancies
4. Structures that are three stories or less in height without subterranean storage or parking
5. Residential structures of Type V construction four stories or less in height without subterranean storage or parking

915.2. Public safety radio amplification systems. New buildings and structures of all occupancy groups that cannot support the required level of radio coverage shall be equipped with either of the following public safety radio amplification systems in order to achieve the required level of radio coverage:

1. A radiating cable system, or
2. An internal antenna system with or without FCC accepted bi-directional UHF or VHF amplifiers.

Where any part of the installed system or systems contains an electrically powered component, the system shall be capable of operating on an independent battery or generator system for a period of not less than least twelve (12)
hours without external power input. The battery system shall automatically charge in the presence of an external power input.

Exception: Where buildings three stories or less in height include subterranean storage or parking, the provisions of this section shall apply only to the subterranean areas.

915.3 Testing procedures. Tests of radio coverage systems shall be conducted pursuant to the specifications in the local agency Public Safety Radio System Coverage Specifications and in accordance with Sections 915.3.1 and 915.3.2.
915.3.1 Initial tests. Initial tests shall be performed by FCC-certified technicians in accordance with test standards listed in the local agency Public Safety Radio System Coverage Specifications.

915.3.2 Annual Tests. Annual tests shall be performed by the local agency having jurisdiction, the local fire department personnel, or their agent in accordance with the test standards listed in the local agency Public Safety Radio System Coverage Specifications.

2. Add new definitions as follows:

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

PUBLIC SAFETY RADIO SYSTEM COVERAGE SPECIFICATIONS. Specifications designed to provide optimum coverage and radio effectiveness within buildings and structures.

FCC-CERTIFIED TECHNICIAN. An individual who is qualified to review design plans and perform tests in affected structures to measure the Public Safety Radio System Coverage Specifications.

PUBLIC SAFETY RADIO AMPLIFICATION SYSTEM. A device that receives an incoming signal, amplifies it and retransmits it on the same frequency. Such devices are used to improve communications in locations within the normal coverage area of a radio system where the signal is blocked or shielded due to natural terrain or man-made obstacles.

Reason: Bi-directional radio amplifier systems will improve fire & life safety protection for building occupants and firefighter personnel. Jurisdictional Fire and Police services use portable radios as their primary communications device, with systems coverage being expected wherever handheld radios may be carried (i.e. basements, high rise buildings, etc.) Reliable wireless communications is an absolute necessity today. Radios are lifelines to first-responders. The public expects first responders to provide their services no matter where the citizen may be; on public and private property alike. The use of these amplification or signal booster systems is critical to the welfare of public safety personnel in the performance of their duties. The installation of such booster signal systems could potentially increase the value of the properties, improving property attractiveness to potential tenants, and can provide insurance savings to the owners.

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

+++

F211–07/08

1027.22 (New)

Proponent: Gary Lewis, City of Summit, NJ, representing ICC Ad-Hoc Committee on Terrorism Resistant Buildings

Add new text as follows:

1027.22 Exit path markings. Existing buildings of Group A ,B, E, I, M, and R-1 having occupied floors located more than 75 feet (22 860mm) above the lowest level of fire department vehicle access shall have exit path markings in accordance with Section 1027 (Supp).

Reason: The membership, at the final hearings of the 2006/2007 code development cycle, overturned the committee action on E84-06/07 with a two-thirds majority vote to include requirements in the IBC and the IFC for luminous exit path markings. The TRB Ad Hoc committee was the original proponent to this code change and it was our intent to make these requirements retroactive for existing buildings. Our intent was not clear in the original proposal, so, at this time, the TRB Ad Hoc committee is proposing to make these requirements applicable to existing buildings.

The proposed new section on exit path markings will require luminescent exit path markings be provided in existing buildings. This proposal will facilitate rapid egress and assist in full building evacuation and is drawn from Recommendations 17 and 18 of the National Institute of Standards and Technology's (NIST) report on the World Trade Center tragedy.

Up to this point, code requirements for high rise buildings were written under the assumption that the building would be evacuated floor by floor. In most instances, in a building with a full suppression system, only the floor where the fire is located and the floors immediately above and below would be evacuated. Acts of terrorism and accidental incidents like power failures have made it necessary to consider design for full building evacuation that is as rapid as possible. This may be made necessary in response to an event within the building or an event outside the building. The proposed code change to require exit path markings is intended to facilitate the most rapid possible full building evacuation.

In the City of New York, after the first bombing of the WTC, requirements were instituted to require exit path markings in vertical exit enclosures in new and existing buildings. This proposal is taken directly from those requirements.

Bibliography:

1. Reference Standard 6-1, Photoluminescent exit path markings as required by Local Law 26 of 2004, New York City Building Code, § 27-383(b)
2. 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 proposal will increase the cost of construction however, the life safety benefit is great.

Analysis: This proposal is based on Section 1027 – Means of Egress for Existing Buildings of the 2006 edition, which will be renumbered to be 1028 in the 2009 edition (due to the addition of new Section 1027 - Exit Path Markings in the 2006/2007 cycle). The reference in this proposal to Section 1027 (Supp) will be to the new Section 1027 in the 2009 edition.