2006/2007 PROPOSED CHANGES TO THE INTERNATIONAL BUILDING CODE — STRUCTURAL

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TENTATIVE ORDER OF DISCUSSION

2006-2007 PROPOSED CHANGES TO THE INTERNATIONAL BUILDING CODE

STRUCTURAL

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

Proposed change numbers that are indented are those which are being heard out of numerical order. Indentation **does not** necessarily indicate that one change is related to another. Proposed changes may be grouped for purposes of discussion at the hearing at the discretion of the chair. Note that some “IBC-S” code change proposals are not included on this list, as they are being heard by other committees. Please consult the Cross Index of Proposed Changes.

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S1–06/07
Table 1604.5

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

Revise table as follows:

TABLE 1604.5
OCCUPANCY CATGORY OF BUILDINGS AND OTHER STRUCTURES

OCCUPANCY CATEGORY	NATURE OF OCCUPANCY
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to: • Agricultural facilities. • Certain temporary facilities. • Minor storage facilities.
II	Buildings and other structures except those listed in Occupancy Categories I, III and IV
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: • Covered structures whose primary occupancy is public assembly with an occupant load greater than 300. • Buildings and other structures ~~with~~ containing elementary school, secondary school or day care facilities with an occupant load greater than 250. • Buildings and other structures ~~with an occupant load greater than 500 for~~ containing adult education facilities, such as colleges ~~or adult education facilities~~ and universities, with an occupant load greater than 500. • ~~Health care facilities~~ Group I-2 occupancies with an occupant load of 50 or more resident patients, but not having surgery or emergency treatment facilities. • ~~Jails and detention facilities~~ Group I-3 occupancies. • Any other occupancy with an occupant load greater than 5,000. • Power-generating stations, water treatment for potable water, waste water treatment facilities and other public utility facilities not included in Occupancy Category IV. • Buildings and other structures not included in Occupancy Category IV containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released.
IV	Buildings and other structures designated as essential facilities, including but not limited to: • ~~Hospitals and other health care facilities~~ Group I-2 occupancies having surgery or emergency treatment facilities. • Fire, rescue and police stations and emergency vehicle garages. • Designated earthquake, hurricane or other emergency shelters. • Designated emergency preparedness, communications, and operations centers and other facilities required for emergency response. • Power-generating stations and other public utility facilities required as emergency backup facilities for Occupancy Category IV structures. • Structures containing highly toxic materials as defined by Section 307 where the quantity of the material exceeds the maximum allowable quantities of Table 307.1(2). • Aviation control towers, air traffic control centers and emergency aircraft hangars. • Buildings and other structures having critical national defense functions. • Water treatment facilities required to maintain water pressure for fire suppression.

Reason: The purpose of this proposal is to align the structural occupancy categories in Table 1604.5 more closely with the nonstructural occupancy classifications elsewhere in the IBC. Under Occupancy Category III, jails and detention facilities are currently listed. Section 308.4 for Group I-3 occupancies, however, also lists prisons, reformatories, correctional centers and prerelease centers as Group I-3 occupancies.

Also, under Occupancy Category III, health care facilities with an occupant load of 50 or more resident patients but not having surgery or emergency treatment facilities are currently listed. Instead of health care facilities, Section 308.3 for Group I-2 occupancies lists hospitals, nursing homes, mental hospitals and detoxification facilities as Group I-2 occupancies. It is conceivable that any of these facilities could provide services for 50 or more resident patients without having surgery or emergency treatment facilities. Similarly under Occupancy Category IV, hospitals and health care facilities having surgery or emergency treatment facilities are currently listed.

In all the cases illustrated above, the absence from Table 1604.5 of the uses listed in the occupancy classifications of Sections 308.3 and 308.4 for Groups I-2 and I-3 occupancies, respectively, may lead code users to conclude that such uses are exempt from the requirements for a higher occupancy category.

The change from health care facilities to Group I-2 occupancies is also intended to avoid classification of a building or structure as Occupancy Category III where it is not warranted. The higher classification is intended to apply to buildings and other structures that represent a substantial hazard to human life in the event of a failure. This is the case for buildings where large numbers of children or adults congregate in one area (e.g., assembly rooms, day care facilities, elementary and secondary schools, etc.). It is also the case for Group I-2 occupancies with resident patients receiving treatment other than surgery or emergency treatment (see Occupancy Category IV).

A health care facility with resident patients, however, could be perceived by some as applying to Group I-1 occupancies. These occupancies provide personal care (i.e., not health care) services to residents (i.e., not patients) in a supervised residential environment. The residents seek the services of a Group I-1 occupancy because of age, mental disability and other reasons but they are assumed to not require chronic or convalescent medical or nursing care. They are also assumed to be capable of responding to an emergency situation without physical assistance from staff.

These occupancies do not represent a substantial hazard to human life.

A change from “colleges or adult education facilities” with an occupant load greater than 500 to “adult education facilities, including colleges and universities” is intended to clarify that the higher level of structural performance associated with Occupancy Category III is warranted at facilities for adult education with high occupant loads. Such facilities can be located at universities as well as colleges, and at facilities not traditionally referred to universities or colleges. The revision will also reduce the possibility of a code user concluding that Occupancy Category III is required at buildings on college and university campuses that do not contain facilities for adult education with high occupant loads, which is not the intent.

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

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

S2–06/07
Table 1604.5

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

Revise table as follows:

TABLE 1604.5
OCCUPANCY CATEGORY OF BUILDINGS AND OTHER STRUCTURES

OCCUPANCY CATEGORY	NATURE OF OCCUPANCY
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to: • Agricultural facilities. • Certain temporary facilities. • Minor storage facilities.
II	Buildings and other structures ~~except those listed in~~ <u>not assigned to</u> Occupancy Categor<u>y</u> ~~ies~~ I, III ~~and~~ <u>or</u> IV
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: • Covered structures whose primary occupancy is public assembly with an occupant load greater than 300. • Buildings and other structures with elementary school, secondary school or day care facilities with an occupant load greater than 250. • Buildings and other structures with an occupant load greater than 500 for colleges or adult education facilities. • Health care facilities with an occupant load of 50 or more resident patients, but not having surgery or emergency treatment facilities. • Jails and detention facilities. • Any other occupancy with an occupant load greater than 5,000. • Power-generating stations, water treatment for potable water, waste water treatment facilities and other public utility facilities not included in Occupancy Category IV. • Buildings and other structures ~~not included in Occupancy Category IV~~ containing ~~sufficient quantities of toxic or explosive substances to be dangerous to the public if released.~~ <u>explosives where the quantity of material exceeds the maximum allowable quantities of Table 307.1(1).</u> <u>• Buildings and other structures containing toxic materials where the quantity of the material exceeds the maximum allowable quantities of Table 307.1(2).</u>
IV	Buildings and other structures designated as essential facilities, including but not limited to: • Hospitals and other health care facilities having surgery or emergency treatment facilities. • Fire, rescue and police stations and emergency vehicle garages. • Designated earthquake, hurricane or other emergency shelters. • Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response. • Power-generating stations and other public utility facilities required as emergency backup facilities for Occupancy Category IV structures. • <u>Building and other</u> structures containing highly toxic materials ~~as defined by Section 307~~ where the quantity of the material exceeds the maximum allowable quantities of Table 307.1.(2). • Aviation control towers, air traffic control centers and emergency aircraft hangars. • Buildings and other structures having critical national defense functions. • Water treatment facilities required to maintain water pressure for fire suppression.

Reason: The purpose of the proposal is to eliminate vague and unenforceable language. Currently, a building or structure is classified as having an occupancy category of III when there are sufficient quantities of toxic or explosive substances to be dangerous to the public if released. This can lead to a wide variance in the quantities of toxic or explosive substances permitted in a building or structure before it is required to be designed for the higher design loads resulting from an occupancy category of III. The proposal will establish objective thresholds for the determination of minimum quantities.

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

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

S3–06/07
Table 1604.5

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

Revise table as follows:

TABLE 1604.5
OCCUPANCY CATEGORY OF BUILDINGS AND OTHER STRUCTURES

OCCUPANCY CATEGORY	NATURE OF OCCUPANCY
I	Buildings and other structures that represent a low hazard to human life in the event of failure, including but not limited to: • Agricultural facilities. • Certain temporary facilities. • Minor storage facilities.
II	Buildings and other structures except those listed in Occupancy Categories I, III and IV
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: • ~~Covered~~ Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300. • Buildings and other structures with elementary school, secondary school or day care facilities with an occupant load greater than 250. • Buildings and other structures with an occupant load greater than 500 for colleges or adult education facilities. • Health care facilities with an occupant load of 50 or more resident patients, but not having surgery or emergency treatment facilities. • Jails and detention facilities. • Any other occupancy with an occupant load greater than 5,000. • Power-generating stations, water treatment for potable water, waste water treatment facilities and other public utility facilities not included in Occupancy Category IV. • Buildings and other structures not included in Occupancy Category IV containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released.
IV	Buildings and other structures designated as essential facilities, including but not limited to: • Hospitals and other health care facilities having surgery or emergency treatment facilities. • Fire, rescue, ambulance and police stations and emergency vehicle garages. • Designated earthquake, hurricane or other emergency shelters. • Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response. • Power-generating stations and other public utility facilities required as emergency backup facilities for Occupancy Category IV structures. • Structures containing highly toxic materials as defined by Section 307 where the quantity of the material exceeds the maximum allowable quantities of Table 307.1.(2). • Aviation control towers, air traffic control centers and emergency aircraft hangars. • Buildings and other structures having critical national defense functions. • Water ~~treatment~~ storage facilities and pump structures required to maintain water pressure for fire suppression.

Reason: Substitute revised material for current provision of the Code.
The purpose of the proposal is to align IBC Table 1604.5 more closely with corresponding Table 1-1 of ASCE 7-05, which contains certain terms not included in Table 1604.5. Their absence from Table 1604.5 may lead code users to conclude that the uses stipulated in Table 1-1 of ASCE 7-05 for a higher occupancy category are exempt from the same requirement in the IBC due to their absence in Table 1604.5.

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

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

S4–06/07
Table 1604.5

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

1. Revise table as follows:

TABLE 1604.5
OCCUPANCY CATEGORY OF BUILDINGS AND OTHER STRUCTURES

OCCUPANCY CATEGORY	NATURE OF OCCUPANCY
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure including, but not limited to: • ~~Covered~~ Buildings and other structures whose primary occupancy is ~~public assembly~~ Group A1, A2, A3, or A4 with an occupant load greater than 300 • Buildings and other structures ~~with~~ containing Group E occupancies ~~elementary school, secondary school or day care facilities~~ with an occupant load of greater than 250 • Buildings and other structures containing Group B educational facilities with an occupant load of greater than 500 ~~for colleges or adult education~~ • Buildings and other structures containing Group I-2 ~~H~~healthcare facilities which provide care on a 24 hour basis for ~~more than with an occupant load of~~ 50 or more ~~resident~~ patients but which do not ~~having~~ contain surgery or emergency treatment facilities • Buildings and other structures containing Group I-3 Jails and detention facilities • Buildings and other structures containing an occupancy, other than those listed above, ~~Any other occupancy~~ with an occupant load of greater than 5000 • Power-generating stations, water treatment for potable water, waste water treatment facilities and other public utility facilities not included in Category IV • Buildings and other structures not included in Category IV containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released

(Portions of table not shown do not change)

2. TABLE 1604.5
OCCUPANCY CATEGORY OF BUILDINGS AND OTHER STRUCTURES

OCCUPANCY CATEGORY	NATURE OF OCCUPANCY
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure including, but not limited to: • Any other occupancy with an occupant load of greater than 5000[a]

(Portions of table not shown do not change)

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

Reason: The intent of the code change is two fold: improve clarity and specificity of the Category III terms, and to provide some reasonable adjustment in the threshold relating to an occupant load of 5000 in any one occupancy.

In order to determine occupant loads, the user is forced to use methods outlined in Section 1004. There is no clear rational that connects occupant loads used to calculate minimum means of egress standards to risks associated with structural design standards. This is particularly the case for the 5000 threshold trigger in multi-story high-rise buildings.

Chapter 10 sets forth standards that provide a reasonably conservative number of occupants for all spaces, and while actual loads are commonly less than the design amount, it is not unusual in the life of a space in a building to have periods when high actual occupant loads exist. From a whole building perspective in multistory building, Chapter 10 does not require the occupant load of the whole building to be determined; rather the egress design is determined on a floor to floor basis with the floor containing the largest design occupant load controlling the design from that floor to grade.

Table 1604.5 requires that the total occupant load of an occupancy be calculated in a building – if the occupancy is spread over 30 stories, then all 30 stories are added. Based on Chapter 10, this assumes a maximum occupant load on every floor, and may result in an excessive assumption.

It seems that some method similar to live load reductions would be more reasonable. In the interim, the proposed footnote is suggesting a code based method that would provide a more reasonable approach for occupancies such as office, mercantile, and residential that are required to base occupant load on gross area – an area that includes corridors, stairways, elevators, closets, accessory areas, structural walls and columns, etc.

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

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

S5–06/07
1604.11 (New), 1605 (New)

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

Add new text as follows:

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

1605 DISPROPORTIONATE COLLAPSE

1605.1 General. The building, structure or portion thereof shall be constructed so the building will not suffer collapse as the result of an accident or incident to an extent disproportionate to the cause. Buildings shall be designed for sufficient robustness to sustain a limited extent of damage or failure, depending on the class of the building, without collapse. Design of new buildings in accordance with Section 1605.5 shall be deemed to comply with Section 1605.4.

1605.2 DEFINITIONS.

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

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

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

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

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

TABLE 1605.3
BUILDING CLASS

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

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

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

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

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

1605.4.2.1.1 Class 2 masonry general (performance). For composite construction, such as masonry load-bearing walls with other materials for the floor and roof systems, the application of both the requirements of this section and those provided for the other materials are required. Peripheral, internal, and column or wall ties shall be provided at each floor level and at roof level, except where the roof is of lightweight construction, no such ties need be provided at that level. Horizontal ties shall be provided by structural members or by reinforcement that is provided for other purposes.

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

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

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

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

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

1605.4.2.2.1 Class 2 steel general (performance). For composite construction, such as concrete deck slabs on steel beams, sheet steel decking with an integral slab, and columns reinforced with structural steel shapes, the application of both the requirements of this section and those provided for reinforced concrete in ACI 318 are required. For a concrete deck slab on steel beam in which the slab is used to provide internal tie capacity, the floor system and roof system shall comply with the internal tie requirements of ACI 318, while the steel frame shall comply with the other tie requirements (peripheral and external column) contained in Section 1605.4.2.2.2.

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

1605.4.2.3 Class 2 structural use of plain, reinforced and prestressed concrete (performance). Design to protect against disproportionate collapse for concrete shall be in accordance with ACI 318. For composite construction, such as concrete deck slabs on steel beams, sheet steel decking with an integral slab, and columns reinforced with structural steel shapes, the application of both the requirements of this section and those provided for structural steel. For a concrete deck slab on steel beam in which the slab is used to provide internal tie capacity, the floor system and roof system shall comply with the internal tie requirements of ACI 318, while the steel frame shall comply with the other tie requirements (peripheral and external column).

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

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

1605.4.3.1.1 Class 3 masonry general (performance). For composite construction, such as masonry load-bearing walls with other materials for the floor and roof systems, the application of both the requirements of this section and those provided for the other materials are required. The masonry walls shall comply with the tie (vertical, peripheral, and wall) requirements or alternate load path requirements. Peripheral, internal, and column or wall ties shall be provided at each floor level and at roof level, except where the roof is of lightweight construction, no such ties need be provided at that level. Horizontal ties shall be provided by structural members or by reinforcement that is provided for other purposes.

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

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

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

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

1605.4.3.1.6 Class 3 masonry vertical ties (performance). Columns and load-bearing walls shall have vertical ties. Vertical ties shall extend from the roof level to the foundation. Vertical ties fully anchored at each end and at each floor level. All joints shall be design to transmit the required tensile forces. The wall shall be constrained between concrete surfaces or other similar construction capable of providing resistance to lateral movement and rotation across the full width of the wall.

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

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

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

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

1605.4.3.2.1 Class 3 steel general (performance). For composite construction, such as concrete deck slabs on steel beams, sheet steel decking with an integral slab, and columns reinforced with structural steel shapes, the application of both the requirements of this section and those provided for reinforced concrete in ACI 318 are required. For a concrete deck slab on steel beam in which the slab is used to provide internal tie capacity, the floor system and roof system shall comply the internal tie requirements of ACI 318, while the steel frame shall comply the other tie requirements (vertical, peripheral, and external column) and the alternate load path requirements of this section.

1605.4.3.2.2 Class 3 steel tie force requirements (performance). All buildings shall be effectively tied together at each principal floor level. Each column shall be effectively held in position by means of horizontal ties in two directions at each principal floor level supported by that column. Continuous lines of ties shall be provided at the edges of the floor or roof and to each column line.

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

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

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

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

1605.4.3.3 Class 3 concrete structural use of plain, reinforced and prestressed concrete (performance). Design to protect against disproportionate collapse for concrete shall be in accordance with ACI 318. For composite construction, such as concrete deck slabs on steel beams, sheet steel decking with an integral slab, and columns reinforced with structural steel shapes, the application of both the requirements of this section and those provided for structural steel. For a concrete deck slab on steel beam in which the slab is used to provide internal tie capacity, the floor system and roof system shall comply with the internal tie requirements of ACI 318, while the steel frame shall comply the other tie requirements (vertical, peripheral, and external column).

1605.4.3.3.1 Class 3 concrete alternate load path method design requirements (performance). Alternate load path method is used to verify that the structure can bridge over removed elements. The design strengths shall be determined in accordance with ACI 318. If the design strengths are less, then compliance shall be in accordance with the alternate load path model subsection.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

1605.5.4 Class 4 buildings (prescriptive). Class 4 buildings shall comply with the requirements for Class 3 buildings in accordance with Section 1605.5.3 and a systematic risk assessment of the building shall be undertaken taking into account all the normal hazards that may be reasonably foreseen, together with any abnormal hazard. Critical situations for design shall be selected that reflect the conditions that can reasonably be foreseen as possible during the life of the building.

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

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

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

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

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

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

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

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

or

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

Where:

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

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

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

1605.6.1.5 Masonry horizontal ties to external columns and walls (prescriptive). Each external column and every 3.33 feet length of external load-bearing wall shall be anchored or tied horizontally into the structure at each floor and roof level with a design tie strength equal to:

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

Where:

H = Clear story height (ft)
F_t = "Basic Strength" = Lesser of $(4.5 + 0.9N_s)$ or 13.5
N_s = Number of stories including basement(s)

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

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

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

1605.6.1.6.2 Masonry columns (prescriptive). A column or every 3.33 feet length of a load-bearing wall that complies with the minimum requirements of Table 1605.6.1.6.1, shall provide a required tie strength equal to:

$6.2 \times 10^{-4} A(h_a/t)^2$ or 22.5 whichever is larger. (kips)

Where:

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

TABLE 1605.6.1.6.1
MINIMUM PROPERTIES FOR MASONRY WALLS WITH VERTICAL TIES

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

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

TABLE 1605.6.1.6.3
REMOVAL OF DEFICIENT MASONRY VERTICAL TIE ELEMENTS

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

a. Lateral supports shall be provided by the following:

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

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

1605.6.1.8 Masonry alternate load path method design requirements (prescriptive). Alternate load path method is used to verify that the structure can bridge over removed elements. The design strengths shall be determined from ACI 530. If the design strengths are less than those in Table 1605.6.1.8, then compliance shall be in accordance with the alternate load path Section 1605.6.1.8.3.

TABLE 1605.6.1.8
ACCEPTABILITY CRITERIA AND SUBSEQUENT ACTION FOR MASONRY

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

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

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

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

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

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

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

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

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

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

TABLE 1605.6.1.8.1.8
DEFORMATION LIMITS FOR MASONRY

	CLASS 2 AND 3 BUILDINGS		CLASS 4 BUILDINGS	
Component	**Ductility** υ	**Rotation, Degrees** θ	**Ductility** υ	**Rotation, Degrees** θ
Unreinforced Masonry[a]	-	2	-	1
Reinforced Masonry[b]	-	7	-	2

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

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

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

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

1605.6.1.8.9.3 Masonry linear and nonlinear static analysis load case (prescriptive). Linear and nonlinear static analysis shall have a factored load combination applied to the immediate adjacent bays and at all the floors above the removed element, using the following formula.

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

Where:

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

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

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

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

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

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

TABLE 1605.6.2.2
OVER-STRENGTH FACTORS FOR STRUCTURAL STEEL

STRUCTURAL STEEL	ULTIMATE OVER-STRENGTH FACTOR, Ω_u	YIELD OVER-STRENGTH FACTOR, Ω_y
Hot-Rolled Structural Shapes and Bars	1.05	
ASTM A36/A36M	1.05	1.5
ASTM A573/A572M Grade 42	1.05	1.3
ASTM A992/A992M	1.05	1.1
All grades	1.05	1.1
Hollow Structural Sections	1.05	
ASTM A500, A501, A618, and A847	1.05	1.3
Steel Pipes	1.05	
ASTM A53/A53M	1.05	1.4
Plates	1.05	1.1
All other products	1.05	1.1

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

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

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

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

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

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

1605.6.2.3.3 Steel internal ties (prescriptive). Design steel members acting as internal ties and their end connections shall be capable of resisting the following required tie strength, which need not be considered as additive to other loads.

The required tie strength is calculated as follows:

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

Where:

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

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

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

Where:

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

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

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

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

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

TABLE 1605.6.2.4.1
ACCEPTABILITY CRITERIA AND SUBSEQUENT ACTION FOR STRUCTURAL STEEL

STRUCTURAL BEHAVIOR	ACCEPTABILITY CRITERIA	SUBSEQUENT ACTION FOR VIOLATION OF CRITERIA
Element Flexure	φM_n[a]	Section 1605.6.2.4.1
Element Combined Axial and Bending	AISC LRFD Chapter H Interaction Equations[a]	Section 1605.6.2.4.2
Element Shear	φV_n[a]	Section 1605.6.2.4.3
Connections	Connection Design Strength[a]	Section 1605.6.2.4.4
Deformation	Deformation Limits, defined in Table 1605.6.2.5(1)	Section 1605.6.2.4.5

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

1605.6.2.4.1 Steel flexural resistance of structural steel (prescriptive). A flexural member can fail by reaching its full plastic moment capacity, or it can fail by lateral-torsional buckling (LTB), flange local buckling (FLB), or web local buckling (WLB). Calculate nominal moment strength, M_n, in accordance with AISC 360. If a flexural member's capacity is governed by a buckling mode of failure, remove the element when the internal moment reaches the nominal moment strength. Distribute the loads associated with the element in accordance with Section 1605.6.2.4.1.1. If the member strength is not governed by buckling, the strength will be governed by plastification of the cross-section and it may be possible for a plastic hinge to form.

Deformation of primary members shall not cause premature failure in secondary members, due to geometric interference. Torsional rotation of a girder shall not cause excessive deformation and stresses in any beam that frames into the girder with a simple shear tab connection.

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

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

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

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

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

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

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

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

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

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

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

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

1605.6.2.4.6.3 Steel linear and nonlinear static analysis load case (prescriptive). Linear and nonlinear static analysis shall have a factored load combination applied to the immediate adjacent bays and at all the floors above the removed element, using the following formula.

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

Where:

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

TABLE 1605.6.2.5(1)
DEFORMATION LIMITS FOR STRUCTURAL STEEL

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

a. As defined in AISC 341.

TABLE 1605.6.2.5(2)
STEEL MOMENT FRAME CONNECTION TYPES

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

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

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

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

TABLE 1605.6.3.1
ACCEPTABILITY CRITERIA AND SUBSEQUENT ACTION FOR REINFORCED CONCRETE

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

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

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

TABLE 1605.6.3.1.1
OVER-STRENGTH FACTORS FOR REINFORCED CONCRETE

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

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

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

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

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

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

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

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

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

TABLE 1605.6.3.1.6
DEFORMATION LIMITS FOR REINFORCED CONCRETE

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

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

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

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

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

1605.6.3.2.3 Concrete linear and nonlinear static analysis load case (prescriptive). Linear and nonlinear static analysis shall have a factored load combination applied to the immediate adjacent bays and at all the floors above the removed element, using the following formula.

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

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

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

1605.6.4.1 Load combinations (prescriptive). The following load combinations shall be used in addition to the accidental design loading in the key element analysis:

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

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

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

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

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

This proposal is intended to implement the very first recommendation of the National Institute of Standards and Technology's (NIST) report on the World Trade Center (WTC) tragedy. It is very important to understand that neither the NIST Report nor the proponents of this change seek to make buildings immune to attack by airliners. Rather, the WTC event resulted in a detailed examination of the adequacy of our codes in connection with a wide variety of much less dramatic damage scenarios, including now, for the first time, some that might be willful and deliberate.

The Code and the many standards that it references deal comprehensively and thoroughly with the live and dead loads that buildings routinely encounter, including exceptional but predictable extreme loads such as wind and seismic. The Code does not deal at all with damage, accidental or deliberate. The possibility of deliberate damage was brought home by the WTC tragedy but it has always existed. The same is true with accidental damage. Whether a bomb, a gas explosion, or a vehicle accidentally taking out a ground level column, it is simply unacceptable that the current code would permit structural systems that are prone to total progressive collapse following a relatively minor initiating event.

This is the sort of issue that one might expect to be addressed through engineering design standards such as ASCE-7 and others. It is not and there is not, at this writing, any firm plan or timetable to do so. It is the proponents' belief that the time is long past for such a dramatic gap in the public safety requirement for buildings to exist. The proponents believe that the Code should establish a strong public policy against disproportionate damage and progressive collapse. This proposal also includes detailed technical requirements. Those would be better included as standards that could be referenced. The near complete absence of detailed technical design requirements from American standards means that they have to be included here. Only ACI 318-02 contains any technical requirements, and those are only applicable to the "tie forces" approach in concrete design. That standard is referenced by this proposal and detailed technical requirements for that subject are not included in the proposal. It is the proponents' hope that the nation's engineering community will take up, soon and with urgency, the challenge of preparing detailed technical standards that will be suitable for reference in future editions of the Code.

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

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

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

1604.11 – establishes the basic requirement that structures be designed to resist disproportionate collapse.

1605.1 – sets forth the basic standard that the Code will require be met

1605.2 – provides definitions needed to understand and apply the Sections.

1605.3 – establishes a four level classification system for all buildings by size and by occupancy group.

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

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

1605.4 – sets forth the criteria for the performance design approach.
Different requirements are set forth for each of the four (4) classes established by Section 1605.3

Class 1 buildings are not required to comply.
Class 2 buildings are required to have effective horizontal ties.
Class 3 buildings are required to have effective horizontal and vertical ties 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 the Class 2 buildings of masonry (1605.4.2.1), steel (1605.4.2.2), and concrete (1605.4.2.3). Similarly, the requirements for Class 3 buildings and set forth for masonry (1605.4.3.1), steel (1605.4.3.2), and concrete (1605.4.3.3).

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

Bibliography:

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

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General Services Administration. <ins>Facilities Standards for the Public Building Service, Chapter 8, Security Design.</ins> Washington, DC. November 2000.

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

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

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

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

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

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

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

S6–06/07

1605.3.1.1

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

Revise as follows:

1605.3.1.1 Stress increases. Increases in allowable stresses specified in the appropriate material chapter of this code or referenced standard shall not be used with the load combinations of Section 1605.3.1 except that ~~a duration of load~~ increase<ins>s due to adjustment factors in accordance with AF&PA NDS</ins> shall be permitted ~~in accordance with Chapter 23~~.

Reason: Several adjustment factors for the design of wood construction can result in increases to reference design values. The flat use factor, repetitive member factor, buckling stiffness factor and bearing area factor are examples of this. These factors are material dependent in much the same manner as load duration factor. The prohibition on stress increases when using the basic allowable stress design load combinations is intended to prevent increases in stresses already accounted for by the decreases in multiple variable loads incorporated into the load combinations. The adjustment factors listed above, however, are not accounted for because they are not related to design loads.

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

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

S7–06/07

1605.3.1.1

Proponent: Jeffrey B. Stone, American Forest & Paper Association

Revise as follows:

1605.3.1.1 Stress increases. Increases in allowable stresses specified in the appropriate material chapter or the referenced standards shall not be used with the load combinations of Section 1605.3.1, except that ~~a duration of load~~ increase<ins>s</ins> shall be permitted in accordance with Chapter 23.

Reason: The change will eliminate confusion. A literal interpretation of current language would imply prohibition on the use of other increases such as the repetitive member factor, flat-use factors, size factors, etc.

Cost Impact: The code change proposal will not increase the cost of construction. This change merely clarifies the use of applicable adjustment factors contained in the NDS.

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

S8–06/07
1605.1, 1605.4

Proponent: W. Lee Shoemaker, Metal Building Manufacturers Association, Inc. (MBMA)

Revise as follows:

1605.1 General. Buildings and other structures and portions thereof shall be designed to resist the load combinations specified in Sections 1605.2 or 1605.3 and Chapters 18 through 23, and the special seismic load combinations of ~~Section 1605.4~~ <u>Section 12.4.3.2 of ASCE 7</u> where required by Section 12.3.3.3 or 12.10.2.1 of ASCE 7. Applicable loads shall be considered, including both earthquake and wind, in accordance with the specified load combinations. Each load combination shall also be investigated with one or more of the variable loads set to zero.

Delete without substitution:

~~**1605.4 Special seismic load combinations.** For both allowable stress design and strength design methods where specifically required by Section 1605.1 or by Chapters 18 through 23, elements and components shall be designed to resist the forces calculated using Equation 16-22 when the effects of the seismic ground motion are additive to gravity forces and those calculated using Equation 16-23 when the effects of the seismic ground motion counteract gravity forces.~~

~~$1.2D + f_1L + E_m$~~ ~~**(Equation 16-22)**~~
~~$0.9D + E_m$~~ ~~**(Equation 16-23)**~~

~~where:~~
~~E_m = The maximum effect of horizontal and vertical forces as set forth in Section 12.4.3 of ASCE 7.~~

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

Reason: The purpose of this change is to remove the inconsistencies between ASCE 7 and IBC with regard to the special seismic load combinations.

There needs to be a correct set of special seismic load combinations to be used with Allowable Stress Design. The existing IBC Section 1605.4 is really only correct for strength design methods (even though it says it can be used for both).

The proposed revision invokes ASCE 7 for the special seismic load combinations, because ASCE 7 correctly has two distinct sets of load combinations – one for strength design and one for allowable stress design. Alternatively, IBC could reproduce the load combinations listed in ASCE 7, Section 12.4.3.2, but it seems better to just reference them.

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

Analysis: If approved, the proposal would result in a terminology difference between the IBC and ASCE 7 since that document does not use the term "special seismic load combinations." The IBC also contains several references to Section 1605.4 which is proposed for deletion.

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

S9–06/07
1602, 202, Table 1607.1; IRC R202, Table R 301.5

THIS PROPOSAL IS ON THE AGENDA OF THE IBC STRUCTURAL AND THE IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

Proponent: Jonathan C. Siu, City of Seattle, representing Washington Association of Building Officials

PART I – IBC

1. Delete definitions without substitution:

SECTION 202
DEFINITIONS

~~**BALCONY, EXTERIOR.** See Section 1602.1.~~
~~**DECK.** See Section 1602.1.~~

SECTION 1602
DEFINITIONS AND NOTATIONS

1602.1 Definitions. The following words and terms shall, for the purposes of this chapter, have the meanings shown herein.

~~**BALCONY, EXTERIOR.** An exterior floor projecting from and supported by a structure without additional independent supports.~~

~~**DECK.** An exterior floor supported on at least two opposing sides by an adjacent structure, and/or posts, piers or other independent supports.~~

2. Revise table as follows:

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS AND MINIMUM CONCENTRATED LIVE LOADS

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
4. Assembly areas and theaters Fixed seats (fastened to floor) Follow spot, projections, and control rooms Lobbies Movable seats Stages and platforms <u>Other assembly areas</u>	 60 50 100 100 125 <u>100</u>	____
5. Balconies <u>(exterior) and decks[h]</u> ~~On one- and two-family residences only, and not exceeding 100 sq ft~~	~~100~~ ~~60~~ <u>Same as occupancy served</u>	____
~~9. Decks~~	~~Same as occupancy served[h]~~	____
28. Residential One- and two-family dwellings Uninhabitable attics without storage[i] Uninhabitable attics with storage[i, j, k] Habitable attics and sleeping areas All other areas ~~except balconies and decks~~ Hotels and multifamily dwellings Private rooms and corridors serving them Public rooms and corridors serving them	 10 20 30 40 40 100	____

h. See Section 1604.8.3 for decks attached to exterior walls.

(Portions of table and footnotes not shown do not change)

PART II – IRC

1. Delete definitions without substitution:

SECTION R202

~~**[B] BALCONY, EXTERIOR.** An exterior floor projecting from and supported by a structure without additional independent supports.~~

~~**[B] DECK.** An exterior floor system supported on at least two opposing sides by an adjoining structure and/or posts, piers, or other independent supports.~~

2. Revise table as follows:

TABLE R301.5
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS
(in pounds per square foot)

USE	LIVE LOAD
Balconies (exterior) and decks[e]	40
~~Exterior balconies~~	~~60~~

e. See Section R502.2.1 for decks attached to exterior walls.

(Portions of table and footnotes not shown do not change)

Reason: This proposal is one of four dealing with changing Table 1607.1, Minimum Uniformly Distributed Live Loads and Minimum Concentrated Live Loads. The main intent of all these code change proposals is to remove the illogical distinction between deck and balcony live loads. In order to do that in the code, one must determine the design live loads for these elements. However, for the purposes of these proposals, that is secondary to removing the distinction. Each of the four proposals eliminates the distinction in the same way (delete the definitions and combine the items in Table 1607.1), but each proposes a different live load. While the reasoning below focuses on the proposed changes to the IBC, the same arguments apply to the proposed changes to the IRC.

The supporting information has been broken into two parts. The first part is repeated on all four proposals, and relates to removing the distinction between balconies and decks. The second part is unique to each proposal, as it gives reasons for the particular live load being proposed.

BALCONY VS. DECK

The current situation was set up in the 1996-1997 timeframe, when two of the three legacy organizations adopted definitions for decks and balconies into their codes. The definitions were then carried forward into the IBC. This error has now been propagated from the IBC into the 2005 edition of ASCE 7, which previously did not define the terms and had different live load requirements from the IBC and legacy codes. There are several reasons, explained below, why the original change made 10 years ago was incorrect, and the distinction between balconies and decks should be eliminated.

Technical Justification: There is no engineering justification for having different live loads for different support conditions, if the use is the same. Either the loads are there, or they aren't, and changing how the element is supported doesn't change the loads. If there are inherent problems with a particular type of structure or with a particular structural material, then the solution should be dealt with on the "resistance" side by increasing the required factor of safety or through additional requirements in the materials chapters, rather than by increasing the loads.

Having participated in the debate at one of the organizations' hearings in 1996, we believe no logical or technical justification was presented to make this distinction—only that the "feeling" was that cantilevers are less redundant than supported structures, and thus, should have a higher live load requirement. Again, if this is the case (which is doubtful), then the solution should be to increase the factor of safety, rather than to increase the live load.

Redundancy: Essentially all of the balcony/deck structures we see are either cantilevered or simply-supported structures. Some engineers will argue that a cantilever is less redundant than simply-supported systems. That is, a single failure could lead to collapse. However, from an engineering standpoint as applied to these structures, a simply-supported structure has no added redundancy compared to a cantilever.

Safety Record: The safety record of cantilevers is better than decks. If simply-supported systems are more redundant than cantilevers, one would expect to see increased safety as reflected by fewer collapses. However, in a Google search for "deck/balcony/failure/collapse", we were only able to find one instance of cantilevered balconies that failed, in Australia. In contrast to that single case of a cantilevered balcony failure, there were many reports of deck failures.

With most of the reports of failures, it could not be distinguished whether the structure was cantilevered or not. However, where it could be distinguished that the failed structure was a "deck" or a "balcony" per the definitions in the code, the vast majority were "deck" failures. Usually, the deck failures occurred at the connection of the deck to the building due to incorrect or poor design (e.g., nails in withdrawal, incorrect type of joist hanger) or by deterioration of the connection components. In the reports for some cases, it was questioned whether proper permits had been obtained. In one recent case in the state of Washington, the posts supporting the structure were not connected to anything at the ground level, and they "kicked out". In the one balcony failure case, the concrete balconies apparently developed a crack at the support allowing moisture to rust the rebar. Neither of these causes of failure (poor design or deterioration) can be attributed to a lack of redundancy. It is notable that where the reports discussed loading conditions, it was to state the failures were not caused by overload conditions.

Consistency: The live loads for balconies and decks are inconsistent with all the other loads in the Live Load table (IBC Table, 1607.1, ASCE 7-05 Table 4-1), in that no other loads are based on the structural support conditions. All others are based on occupancy or use (which is the heading in the table). Logically, if cantilevers are inherently dangerous, then all other items in the table should have separate loads for cantilevers versus other support conditions.

Definitions: The definitions were inserted into the two legacy codes because the live load tables required different loads for balconies versus decks, similar to ASCE 7. Once it has been demonstrated there is not a reason to apply different loading conditions to balconies versus decks, there is no need to define the terms.

It is to be noted, however, there is not an exact match between the legacy code definitions and what appears to be the intended application in ASCE 7. Table 4-1 of earlier editions of ASCE 7 has an item for "Balconies (exterior)" (live load = 100 psf, or 60 psf for small residential balconies), and an item for "Decks (patio and roof)" (live load = "same as area served, or for the type of occupancy accommodated"). One legacy code deleted the parenthetical "patio and roof" from the "deck" item. The second retained it, but inserted the same definitions. It appears the definitions inserted into the legacy codes were in error as compared to ASCE 7, because "decks" were supposed to be patios (decks on grade?) or roof decks.

However, even if one were to redefine "balcony" and "deck" to fit with what appears to be the intent of ASCE 7, there does not appear to be justification for having different loads for them, as they will most likely be used similarly.

IBC versus ASCE 7:

Some will argue that IBC and ASCE 7 should not be different, and that it is really the province of ASCE 7 to determine appropriate live loads. In general, we agree with this philosophy, and it is our intent to submit similar proposals to the ASCE 7 process. There are two reasons why we believe ICC should act now:

1. It is our understanding that the primary reason for the deck and balcony modification to the live load table of ASCE 7-05 was so it would match the organization contained in the 2003 IBC. As stated above, this just means that errors made in legacy codes have been propagated now into ASCE 7. Therefore, if ASCE 7 has been changed once to match the IBC, there is no reason why the IBC can't lead the way again.
2. It is our understanding that the next edition of ASCE 7 is not scheduled to come out until 2010. If one assumes that ASCE 7 fixes this problem in their process in that cycle (and there is no guarantee that it will), this means it will not be until the 2012 edition of the IBC that the fix will be included in the code, which will mean it will be 2013 before many jurisdictions actually adopt the code. That is too long to be propagating this error.

DESIGN LIVE LOAD FOR BALCONIES AND DECKS:
Once the premise has been accepted that the loads should not differ based on structural support conditions, the question is, what is the appropriate design live load for these structures?

The premise behind this option is if a deck can be designed to the same load as the occupancy it serves (as the code currently allows), the same should be allowed for balconies. If the balcony/deck serves a one-family dwelling, the minimum live load will be 40 psf. If it serves a private office, the live load is 50 psf. If it is an assembly area such as a roof deck, then it can be argued that it should be designed for 100 psf. The addition proposed to the Assembly item in Table 1607.1 will clarify this requirement, as well as for other assembly areas not currently covered by the table. It is significant to note that where the reports turned up in the Google search discussed loading conditions, it was to state that the decks did not fail due to overload conditions.

The callout for Footnote h in Table 1607.2 has been moved (attached to "decks" instead of the load), since it only applies to decks.

The changes being proposed in Part II for the IRC are for consistency with the terminology used in the IBC and with the live loads in the Part I proposal.

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

PART I – IBC

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

PART II – IRC

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

S10–06/07

1602, 202, Table 1607.1; IRC R202, Table R301.5

THIS PROPOSAL IS ON THE AGENDA OF THE IBC STRUCTURAL AND THE IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

Proponent: Jonathan C. Siu, City of Seattle, representing Washington Association of Building Officials

PART I – IBC

1. Delete definitions without substitution:

SECTION 202
DEFINITIONS

~~**BALCONY, EXTERIOR.** See Section 1602.1.~~
~~**DECK.** See Section 1602.1.~~

SECTION 1602
DEFINITIONS AND NOTATIONS

1602.1 Definitions. The following words and terms shall, for the purposes of this chapter, have the meanings shown herein.

~~**BALCONY, EXTERIOR.** An exterior floor projecting from and supported by a structure without additional independent supports.~~

~~**DECK.** An exterior floor supported on at least two opposing sides by an adjacent structure, and/or posts, piers or other independent supports.~~

2. Revise table as follows:

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS AND
MINIMUM CONCENTRATED LIVE LOADS

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
4. Assembly areas and theaters [unchanged]		
5. Balconies (exterior) <u>and decks[h]</u> On one- and two-family residences only, and not exceeding 100 sq ft	100 60	____
~~9. Decks~~	~~Same as occupancy served[h]~~	____
28. Residential [unchanged]		

h. See Section 1604.8.3 for decks attached to exterior walls.

(Portions of table and footnotes not shown do not change)

PART II – IRC

1. Delete definitions without substitution:

SECTION R202

~~**[B] BALCONY, EXTERIOR.** An exterior floor projecting from and supported by a structure without additional independent supports.~~

~~**[B] DECK.** An exterior floor system supported on at least two opposing sides by an adjoining structure and/or posts, piers, or other independent supports.~~

2. Revise table as follows:

TABLE R301.5
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS
(in pounds per square foot)

USE	LIVE LOAD
~~Decks[e]~~	~~40~~
~~Exterior~~ Balconies <u>(exterior) and decks[e]</u>	60

e. See Section R502.2.1 for decks attached to exterior walls.

(Portions of table and footnotes not shown do not change)

Reason: This proposal is one of four dealing with changing Table 1607.1, Minimum Uniformly Distributed Live Loads and Minimum Concentrated Live Loads. The main intent of all these code change proposals is to remove the illogical distinction between deck and balcony live loads. In order to do that in the code, one must determine the design live loads for these elements. However, for the purposes of these proposals, that is secondary to removing the distinction. Each of the four proposals eliminates the distinction in the same way (delete the definitions and combine the items in Table 1607.1), but each proposes a different live load. While the reasoning below focuses on the proposed changes to the IBC, the same arguments apply to the proposed changes to the IRC.

The supporting information has been broken into two pieces. The first part is repeated on all four proposals, and relates to removing the distinction between balconies and decks. The second part is unique to each proposal, as it gives reasons for the particular live load being proposed.

<u>BALCONY VS. DECK</u>

The current situation was set up in the 1996-1997 timeframe, when two of the three legacy organizations adopted definitions for decks and balconies into their codes. The definitions were then carried forward into the IBC. This error has now been propagated from the IBC into the 2005 edition of ASCE 7, which previously did not define the terms and had different live load requirements from the IBC and legacy codes. There are several reasons, explained below, why the original change made 10 years ago was incorrect, and the distinction between balconies and decks should be eliminated.

Technical Justification: There is no engineering justification for having different live loads for different support conditions, if the use is the same. Either the loads are there, or they aren't, and changing how the element is supported doesn't change the loads. If there are inherent problems with a particular type of structure or with a particular structural material, then the solution should be dealt with on the "resistance" side by increasing the required factor of safety or through additional requirements in the materials chapters, rather than by increasing the loads.

Having participated in the debate at one of the organizations' hearings in 1996, we believe no logical or technical justification was presented to make this distinction—only that the "feeling" was that cantilevers are less redundant than supported structures, and thus, should have a higher live load requirement. Again, if this is the case (which is doubtful), then the solution should be to increase the factor of safety, rather than to increase the live load.

Redundancy: Essentially all of the balcony/deck structures we see are either cantilevered or simply-supported structures. Some engineers will argue that a cantilever is less redundant than simply-supported systems. That is, a single failure could lead to collapse. However, from an engineering standpoint as applied to these structures, a simply-supported structure has no added redundancy compared to a cantilever.

Safety Record: The safety record of cantilevers is better than decks. If simply-supported systems are more redundant than cantilevers, one would expect to see increased safety as reflected by fewer collapses. However, in a Google search for "deck/balcony/failure/collapse", we were only able to find one instance of cantilevered balconies that failed, in Australia. In contrast to that single case of a cantilevered balcony failure, there were many reports of deck failures.

With most of the reports of failures, it could not be distinguished whether the structure was cantilevered or not. However, where it could be distinguished that the failed structure was a "deck" or a "balcony" per the definitions in the code, the vast majority were "deck" failures. Usually, the deck failures occurred at the connection of the deck to the building due to incorrect or poor design (e.g., nails in withdrawal, incorrect type of joist hanger) or by deterioration of the connection components. In the reports for some cases, it was questioned whether proper permits had been obtained. In one recent case in the state of Washington, the posts supporting the structure were not connected to anything at the ground level, and they "kicked out". In the one balcony failure case, the concrete balconies apparently developed a crack at the support allowing moisture to rust the rebar. Neither of these causes of failure (poor design or deterioration) can be attributed to a lack of redundancy. It is notable that where the reports discussed loading conditions, it was to state the failures were not caused by overload conditions.

Consistency: The live loads for balconies and decks are inconsistent with all the other loads in the Live Load table (IBC Table, 1607.1, ASCE 7-05 Table 4-1), in that no other loads are based on the structural support conditions. All others are based on occupancy or use (which is the heading in the table). Logically, if cantilevers are inherently dangerous, then all other items in the table should have separate loads for cantilevers versus other support conditions.

Definitions: The definitions were inserted into the two legacy codes because the live load tables required different loads for balconies versus decks, similar to ASCE 7. Once it has been demonstrated there is not a reason to apply different loading conditions to balconies versus decks, there is no need to define the terms.

It is to be noted, however, there is not an exact match between the legacy code definitions and what appears to be the intended application in ASCE 7. Table 4-1 of earlier editions of ASCE 7 has an item for "Balconies (exterior)" (live load = 100 psf, or 60 psf for small residential balconies), and an item for "Decks (patio and roof)" (live load = "same as area served, or for the type of occupancy accommodated"). One legacy code deleted the parenthetical "patio and roof" from the "deck" item. The second retained it, but inserted the same definitions. It appears the definitions inserted into the legacy codes were in error as compared to ASCE 7, because "decks" were supposed to be patios (decks on grade?) or roof decks. However, even if one were to redefine "balcony" and "deck" to fit with what appears to be the intent of ASCE 7, there does not appear to be justification for having different loads for them, as they will most likely be used similarly.

IBC versus ASCE 7:

Some will argue that IBC and ASCE 7 should not be different, and that it is really the province of ASCE 7 to determine appropriate live loads. In general, we agree with this philosophy, and it is our intent to submit similar proposals to the ASCE 7 process. There are two reasons why we believe ICC should act now:

3. It is our understanding that the primary reason for the deck and balcony modification to the live load table of ASCE 7-05 was so it would match the organization contained in the 2003 IBC. As stated above, this just means that errors made in legacy codes have been propagated now into ASCE 7. Therefore, if ASCE 7 has been changed once to match the IBC, there is no reason why the IBC can't lead the way again.
4. It is our understanding that the next edition of ASCE 7 is not scheduled to come out until 2010. If one assumes that ASCE 7 fixes this problem in their process in that cycle (and there is no guarantee that it will), this means it will not be until the 2012 edition of the IBC that the fix will be included in the code, which will mean it will be 2013 before many jurisdictions actually adopt the code. That is too long to be propagating this error.

<u>DESIGN LIVE LOAD FOR BALCONIES AND DECKS:</u>

Once the premise has been accepted that the loads should not differ based on structural support conditions, the question is, what is the appropriate design live load for these structures?

This option is based on taking the most conservative approach to determine the required design live load. That is, since balconies were required to be designed to 100 psf or 60 psf, decks will be required to be designed to the same load. It is our understanding from previous discussions at code change hearings that the 60 psf is derived from the weight of a stack of firewood. However, it is to be noted that none of the reports turned up in the Google search listed overloading due to stacked firewood as a cause of failure.

The callout for Footnote h in Table 1607.2 has been moved (attached to "decks" instead of the load), since it only applies to decks.

The changes being proposed in Part II for the IRC are for consistency with the terminology used in the IBC and with the live loads in the Part I proposal.

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

PART I – IBC

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

PART II – IRC

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

S11–06/07

1602, 202, Table 1607.1; IRC R202, Table R301.5

THIS PROPOSAL IS ON THE AGENDA OF THE IBC STRUCTURAL AND THE IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

Proponent: Jonathan C. Siu, City of Seattle, representing Washington Association of Building Officials

PART I – IBC

1. Delete definitions without substitution:

SECTION 202
DEFINITIONS

~~**BALCONY, EXTERIOR.** See Section 1602.1.~~
~~**DECK.** See Section 1602.1.~~

SECTION 1602
DEFINITIONS AND NOTATIONS

1602.1 Definitions. The following words and terms shall, for the purposes of this chapter, have the meanings shown herein.

~~**BALCONY, EXTERIOR.** An exterior floor projecting from and supported by a structure without additional independent supports.~~

~~**DECK.** An exterior floor supported on at least two opposing sides by an adjacent structure, and/or posts, piers or other independent supports.~~

2. Revise table as follows:

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS
AND MINIMUM CONCENTRATED LIVE LOADS

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
5. Assembly areas and theaters Fixed seats (fastened to floor) Follow spot, projections, and control rooms Lobbies Movable seats Stages and platforms <u>Other assembly areas</u>	 60 50 100 100 125 <u>100</u>	____
5. Balconies (exterior)<u> and decks</u>[h] ~~On one- and two-family residences only, and not exceeding 100 sq ft~~ <u>Accessory to a single tenant or dwelling unit</u>	100 ~~60~~ <u>Same as occupancy served</u>	____
~~9. Decks~~	~~Same as occupancy served[h]~~	____
28. Residential One- and two-family dwellings Uninhabitable attics without storage[i] Uninhabitable attics with storage[i, j, k] Habitable attics and sleeping areas All other areas ~~except balconies and decks~~ Hotels and multifamily dwellings Private rooms and corridors serving them Public rooms and corridors serving them	 10 20 30 40 40 100	____

h. See Section 1604.8.3 for decks attached to exterior walls.

(Portions of table and footnotes not shown do not change)

PART II – IRC

1. Delete definitions without substitution:

SECTION R202

~~**[B] BALCONY, EXTERIOR.** An exterior floor projecting from and supported by a structure without additional independent supports.~~

~~**[B] DECK.** An exterior floor system supported on at least two opposing sides by an adjoining structure and/or posts, piers, or other independent supports.~~

2. Revise table as follows:

TABLE R301.5
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS
(in pounds per square foot)

USE	LIVE LOAD
<u>Balconies (exterior) and</u>decks[e]	40
~~Exterior balconies~~	~~60~~

e. See Section R502.2.1 for decks attached to exterior walls.

(Portions of table not shown do not change)

Reason: Supporting Information: This proposal is one of four dealing with changing Table 1607.1, Minimum Uniformly Distributed Live Loads and Minimum Concentrated Live Loads. The main intent of all these code change proposals is to remove the illogical distinction between deck and balcony live loads. In order to do that in the code, one must determine the design live loads for these elements. However, for the purposes of these proposals, that is secondary to removing the distinction. Each of the four proposals eliminates the distinction in the same way (delete the definitions and combine the items in Table 1607.1), but each proposes a different live load. While the reasoning below focuses on the proposed changes to the IBC, the same arguments apply to the proposed changes to the IRC.

The supporting information has been broken into two pieces. The first part is repeated on all four proposals, and relates to removing the distinction between balconies and decks. The second part is unique to each proposal, as it gives reasons for the particular live load being proposed.

<u>BALCONY VS. DECK</u>

The current situation was set up in the 1996-1997 timeframe, when two of the three legacy organizations adopted definitions for decks and balconies into their codes. The definitions were then carried forward into the IBC. This error has now been propagated from the IBC into the 2005 edition of ASCE 7, which previously did not define the terms and had different live load requirements from the IBC and legacy codes. There are several reasons, explained below, why the original change made 10 years ago was incorrect, and the distinction between balconies and decks should be eliminated.

Technical Justification: There is no engineering justification for having different live loads for different support conditions, if the use is the same. Either the loads are there, or they aren't, and changing how the element is supported doesn't change the loads. If there are inherent problems with a particular type of structure or with a particular structural material, then the solution should be dealt with on the "resistance" side by increasing the required factor of safety or through additional requirements in the materials chapters, rather than by increasing the loads.

Having participated in the debate at one of the organizations' hearings in 1996, we believe no logical or technical justification was presented to make this distinction—only that the "feeling" was that cantilevers are less redundant than supported structures, and thus, should have a higher live load requirement. Again, if this is the case (which is doubtful), then the solution should be to increase the factor of safety, rather than to increase the live load.

Redundancy: Essentially all of the balcony/deck structures we see are either cantilevered or simply-supported structures. Some engineers will argue that a cantilever is less redundant than simply-supported systems. That is, a single failure could lead to collapse. However, from an engineering standpoint as applied to these structures, a simply-supported structure has no added redundancy compared to a cantilever.

Safety Record: The safety record of cantilevers is better than decks. If simply-supported systems are more redundant than cantilevers, one would expect to see increased safety as reflected by fewer collapses. However, in a Google search for "deck/balcony/failure/collapse", we were only able to find one instance of cantilevered balconies that failed, in Australia. In contrast to that single case of a cantilevered balcony failure, there were many reports of deck failures.

With most of the reports of failures, it could not be distinguished whether the structure was cantilevered or not. However, where it could be distinguished that the failed structure was a "deck" or a "balcony" per the definitions in the code, the vast majority were "deck" failures. Usually, the deck failures occurred at the connection of the deck to the building due to incorrect or poor design (e.g., nails in withdrawal, incorrect type of joist hanger) or by deterioration of the connection components. In the reports for some cases, it was questioned whether proper permits had been obtained. In one recent case in the state of Washington, the posts supporting the structure were not connected to anything at the ground level, and they "kicked out". In the one balcony failure case, the concrete balconies apparently developed a crack at the support allowing moisture to rust the rebar. Neither of these causes of failure (poor design or deterioration) can be attributed to a lack of redundancy. It is notable that where the reports discussed loading conditions, it was to state the failures were not caused by overload conditions.

Consistency: The live loads for balconies and decks are inconsistent with all the other loads in the Live Load table (IBC Table, 1607.1, ASCE 7-05 Table 4-1), in that no other loads are based on the structural support conditions. All others are based on occupancy or use (which is the heading in the table). Logically, if cantilevers are inherently dangerous, then all other items in the table should have separate loads for cantilevers versus other support conditions.

Definitions: The definitions were inserted into the two legacy codes because the live load tables required different loads for balconies versus decks, similar to ASCE 7. Once it has been demonstrated there is not a reason to apply different loading conditions to balconies versus decks, there is no need to define the terms.

It is to be noted, however, there is not an exact match between the legacy code definitions and what appears to be the intended application in ASCE 7. Table 4-1 of earlier editions of ASCE 7 has an item for "Balconies (exterior)" (live load = 100 psf, or 60 psf for small residential balconies), and an item for "Decks (patio and roof)" (live load = "same as area served, or for the type of occupancy accommodated"). One legacy code deleted the parenthetical "patio and roof" from the "deck" item. The second retained it, but inserted the same definitions. It appears the definitions inserted into the legacy codes were in error as compared to ASCE 7, because "decks" were supposed to be patios (decks on grade?) or roof decks. However, even if one were to redefine "balcony" and "deck" to fit with what appears to be the intent of ASCE 7, there does not appear to be justification for having different loads for them, as they will most likely be used similarly.

IBC versus ASCE 7: Some will argue that IBC and ASCE 7 should not be different, and that it is really the province of ASCE 7 to determine appropriate live loads. In general, we agree with this philosophy, and it is our intent to submit similar proposals to the ASCE 7 process. There are two reasons why we believe ICC should act now:

5. It is our understanding that the primary reason for the deck and balcony modification to the live load table of ASCE 7-05 was so it would match the organization contained in the 2003 IBC. As stated above, this just means that errors made in legacy codes have been propagated now into ASCE 7. Therefore, if ASCE 7 has been changed once to match the IBC, there is no reason why the IBC can't lead the way again.
6. It is our understanding that the next edition of ASCE 7 is not scheduled to come out until 2010. If one assumes that ASCE 7 fixes this problem in their process in that cycle (and there is no guarantee that it will), this means it will not be until the 2012 edition of the IBC that the fix will be included in the code, which will mean it will be 2013 before many jurisdictions actually adopt the code. That is too long to be propagating this error.

<u>DESIGN LIVE LOAD FOR BALCONIES AND DECKS:</u>

Once the premise has been accepted that the loads should not differ based on structural support conditions, the question is, what is the appropriate design live load for these structures?

This option takes the approach that there are other types of "private" balconies and decks besides residential that would not be expected to be highly loaded. A balcony or deck accessed from a private office could have the same loading condition as a residential balcony or deck. The second premise behind this option is if a deck is allowed to be designed to the same load as the occupancy it serves, the same should be allowed for balconies. If the balcony/deck serves a one-family dwelling, the minimum live load will be 40 psf. If it serves a private office, the live load is 50 psf. If it is an assembly area such as a roof deck, then it can be argued that it should be designed for 100 psf. The addition proposed to the Assembly item in Table 1607.1 will clarify this requirement, as well as for other assembly areas not currently covered by the table. It is significant to note that where the reports turned up in the Google search discussed loading conditions, it was to state that the decks did not fail due to overload conditions.

The callout for Footnote h in Table 1607.2 has been moved (attached to "decks" instead of the load), since it only applies to decks.

The changes being proposed in Part II for the IRC are for consistency with the terminology used in the IBC and with the live loads in the Part I proposal.

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

PART I – IBC

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

PART II – IRC

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

S12–06/07

1602, 202, Table 1607.1; IRC R202, Table R301.5

THIS PROPOSAL IS ON THE AGENDA OF THE IBC STRUCTURAL AND THE IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

Proponent: Jonathan C. Siu, City of Seattle, representing Washington Association of Building Officials

PART I – IBC

1. Delete definitions without substitution:

SECTION 202
DEFINITIONS

~~**BALCONY, EXTERIOR.** See Section 1602.1.~~
~~**DECK.** See Section 1602.1.~~

SECTION 1602
DEFINITIONS AND NOTATIONS

1602.1 Definitions. The following words and terms shall, for the purposes of this chapter, have the meanings shown herein.

~~**BALCONY, EXTERIOR.** An exterior floor projecting from and supported by a structure without additional independent supports.~~

~~**DECK.** An exterior floor supported on at least two opposing sides by an adjacent structure, and/or posts, piers or other independent supports.~~

2. Revise table as follows:

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS AND MINIMUM CONCENTRATED LIVE LOADS

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
6. Assembly areas and theaters [unchanged]		
5. Balconies (exterior) and decks[h] ~~On one- and two-family residences only, and not exceeding 100 sq ft~~ Accessory to a single tenant or dwelling unit	100 ~~60~~ 60	____
~~9. Decks~~	~~Same as occupancy served~~[h]	____

h. See Section 1604.8.3 for decks attached to exterior walls.

(Portions of table and footnotes not shown do not change)

PART II – IRC

1. Delete definitions without substitution:

SECTION R202

~~**[B] BALCONY, EXTERIOR.** An exterior floor projecting from and supported by a structure without additional independent supports.~~

~~**[B] DECK.** An exterior floor system supported on at least two opposing sides by an adjoining structure and/or posts, piers, or other independent supports.~~

2. Revise table as follows:

TABLE R301.5
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS
(in pounds per square foot)

USE	LIVE LOAD
~~Decks[e]~~	~~40~~
~~Exterior~~ ~~b~~<u>B</u>alconies <u>(exterior) and decks</u>[e]	60

e. See Section R502.2.1 for decks attached to exterior walls.

(Portions of table and footnotes not shown do not change)

Reason: Supporting Information: This proposal is one of four dealing with changing Table 1607.1, Minimum Uniformly Distributed Live Loads and Minimum Concentrated Live Loads. The main intent of all these code change proposals is to remove the illogical distinction between deck and balcony live loads. In order to do that in the code, one must determine the design live loads for these elements. However, for the purposes of these proposals, that is secondary to removing the distinction. Each of the four proposals eliminates the distinction in the same way (delete the definitions and combine the items in Table 1607.1), but each proposes a different live load. While the reasoning below focuses on the proposed changes to the IBC, the same arguments apply to the proposed changes to the IRC.

The supporting information has been broken into two pieces. The first part is repeated on all four proposals, and relates to removing the distinction between balconies and decks. The second part is unique to each proposal, as it gives reasons for the particular live load being proposed.

<u>BALCONY VS. DECK</u>

The current situation was set up in the 1996-1997 timeframe, when two of the three legacy organizations adopted definitions for decks and balconies into their codes. The definitions were then carried forward into the IBC. This error has now been propagated from the IBC into the 2005 edition of ASCE 7, which previously did not define the terms and had different live load requirements from the IBC and legacy codes. There are several reasons, explained below, why the original change made 10 years ago was incorrect, and the distinction between balconies and decks should be eliminated.

Technical Justification: There is no engineering justification for having different live loads for different support conditions, if the use is the same. Either the loads are there, or they aren't, and changing how the element is supported doesn't change the loads. If there are inherent problems with a particular type of structure or with a particular structural material, then the solution should be dealt with on the "resistance" side by increasing the required factor of safety or through additional requirements in the materials chapters, rather than by increasing the loads.

Having participated in the debate at one of the organizations' hearings in 1996, we believe no logical or technical justification was presented to make this distinction—only that the "feeling" was that cantilevers are less redundant than supported structures, and thus, should have a higher live load requirement. Again, if this is the case (which is doubtful), then the solution should be to increase the factor of safety, rather than to increase the live load.

Redundancy: Essentially all of the balcony/deck structures we see are either cantilevered or simply-supported structures. Some engineers will argue that a cantilever is less redundant than simply-supported systems. That is, a single failure could lead to collapse. However, from an engineering standpoint as applied to these structures, a simply-supported structure has no added redundancy compared to a cantilever.

Safety Record: The safety record of cantilevers is better than decks. If simply-supported systems are more redundant than cantilevers, one would expect to see increased safety as reflected by fewer collapses. However, in a Google search for "deck/balcony/failure/collapse", we were only able to find one instance of cantilevered balconies that failed, in Australia. In contrast to that single case of a cantilevered balcony failure, there were many reports of deck failures.

With most of the reports of failures, it could not be distinguished whether the structure was cantilevered or not. However, where it could be distinguished that the failed structure was a "deck" or a "balcony" per the definitions in the code, the vast majority were "deck" failures. Usually, the deck failures occurred at the connection of the deck to the building due to incorrect or poor design (e.g., nails in withdrawal, incorrect type of joist hanger) or by deterioration of the connection components. In the reports for some cases, it was questioned whether proper permits had been obtained. In one recent case in the state of Washington, the posts supporting the structure were not connected to anything at the ground level, and they "kicked out". In the one balcony failure case, the concrete balconies apparently developed a crack at the support allowing moisture to rust the rebar. Neither of these causes of failure (poor design or deterioration) can be attributed to a lack of redundancy. It is notable that where the reports discussed loading conditions, it was to state the failures were not caused by overload conditions.

Consistency: The live loads for balconies and decks are inconsistent with all the other loads in the Live Load table (IBC Table, 1607.1, ASCE 7-05 Table 4-1), in that no other loads are based on the structural support conditions. All others are based on occupancy or use (which is the heading in the table). Logically, if cantilevers are inherently dangerous, then all other items in the table should have separate loads for cantilevers versus other support conditions.

Definitions: The definitions were inserted into the two legacy codes because the live load tables required different loads for balconies versus decks, similar to ASCE 7. Once it has been demonstrated there is not a reason to apply different loading conditions to balconies versus decks, there is no need to define the terms.

It is to be noted, however, there is not an exact match between the legacy code definitions and what appears to be the intended application in ASCE 7. Table 4-1 of earlier editions of ASCE 7 has an item for "Balconies (exterior)" (live load = 100 psf, or 60 psf for small residential balconies), and an item for "Decks (patio and roof)" (live load = "same as area served, or for the type of occupancy accommodated"). One legacy code deleted the parenthetical "patio and roof" from the "deck" item. The second retained it, but inserted the same definitions. It appears the definitions inserted into the legacy codes were in error as compared to ASCE 7, because "decks" were supposed to be patios (decks on grade?) or roof decks. However, even if one were to redefine "balcony" and "deck" to fit with what appears to be the intent of ASCE 7, there does not appear to be justification for having different loads for them, as they will most likely be used similarly.

IBC versus ASCE 7: Some will argue that IBC and ASCE 7 should not be different, and that it is really the province of ASCE 7 to determine appropriate live loads. In general, we agree with this philosophy, and it is our intent to submit similar proposals to the ASCE 7 process. There are two reasons why we believe ICC should act now:

1. It is our understanding that the primary reason for the deck and balcony modification to the live load table of ASCE 7-05 was so it would match the organization contained in the 2003 IBC. As stated above, this just means that errors made in legacy codes have been propagated now into ASCE 7. Therefore, if ASCE 7 has been changed once to match the IBC, there is no reason why the IBC can't lead the way again.
2. It is our understanding that the next edition of ASCE 7 is not scheduled to come out until 2010. If one assumes that ASCE 7 fixes this problem in their process in that cycle (and there is no guarantee that it will), this means it will not be until the 2012 edition of the IBC that the fix will be included in the code, which will mean it will be 2013 before many jurisdictions actually adopt the code. That is too long to be propagating this error.

DESIGN LIVE LOAD FOR BALCONIES AND DECKS:

Once the premise has been accepted that the loads should not differ based on structural support conditions, the question is, what is the appropriate design live load for these structures?

This option takes the approach that there are other types of "private" balconies and decks besides residential that would not be expected to be highly loaded. A balcony or deck accessed from a private office could have the same loading condition as a residential balcony or deck. This option is also based on taking the most conservative approach to determine the required design live load. That is, since balconies were required to be designed to 100 psf or 60 psf, decks will be required to be designed to the same load. It is my understanding from previous discussions at code change hearings that the 60 psf is derived from the weight of a stack of firewood. However, it is to be noted that none of the reports turned up in the Google search listed overloading due to stacked firewood as a cause of failure.

The callout for Footnote h in Table 1607.2 has been moved (attached to "decks" instead of the load), since it only applies to decks.

The changes being proposed in Part II for the IRC are for consistency with the terminology used in the IBC and with the live loads in the Part I proposal.

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

PART I – IBC

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

PART II – IRC

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

S13–06/07
Table 1607.1

Proponent: Arlan Smith, Idaho Division of Building Safety, representing Idaho Association of Building Officials

Revise table as follows:

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS AND MINIMUM CONCENTRATED LIVE LOADS

28. Residential
One- and two-family dwellings
Uninhabitable attics without storage
Uninhabitable attics with limited storage
Habitable attics and sleeping areas
All other areas except balconies ~~and decks~~
Hotels and multiple-family dwellings
Private rooms and corridors serving them
Public rooms and corridors serving them

(Portions of table not shown do not change)

Reason: The "except decks" language sends us to item 9 Decks for the load that in turn sends us back to item 28 with no load determined. It is less confusing with this deletion.

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

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

S14–06/07
1607.7.1.3

Proponents: Edwin T. Huston, Smith & Huston, Inc., representing National Council of Structural Engineering Associations and John V. Loscheider; P.E., Loscheider Engineering Company

Delete without substitution:

~~**1607.7.1.3 Stress increase.** Where handrails and guards are designed in accordance with the provisions for allowable stress design (working stress design) exclusively for the loads specified in Section 1607.7.1, the allowable stress for the members and their attachments are permitted to be increased by one-third.~~

Reason: To delete an outdated provision.

(Loscheider) The structural safety of handrails and guards is predominantly governed by strength. When this provision was created during the drafting of the IBC, strength-based (LRFD) material standards were neither widely used nor readily available for all materials. Furthermore, for the design of steel handrails and guards, allowable stress design (ASD) consistently provided substantially lower unfactored load capacities than LRFD, and AISC had no plans to update its ASD standard correct this situation. When the IBC was drafted, the sole purpose of the allowable stress increase for handrails and guards was to provide nominal design parity between LRFD and the much more widely used ASD.

In recent years, however, there have been several important changes in our structural codes. LRFD standards are now more commonly available, and their adoption by reference in the IBC allows designers to rationally evaluate strength-critical elements such as handrails and guards. Furthermore, AISC has finally issued updated ASD provisions, which have been adopted by reference in the 2006 IBC. AISC 360-2005 is an integrated ASD/LRFD design standard that provides consistent parity between the two design methods, so designers are no longer penalized for using ASD. In fact, for many types of members commonly used for handrails and guards, ASD now actually provides unfactored load capacities that are slightly higher than LRFD, without the use of a one-third increase. For this reason, a one-third increase for ASD is no longer appropriate, and continuing to allow its use may result in unsafe handrails and guards.

(Huston) The stress increase is no longer appropriate given the latest editions of the referenced standards that more properly coordinate allowable stress design with load and resistance factor design through a unified design process. The continued use of the one-third stress increase for handrails could lead to unconservative results.

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

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

S15–06/07

1609.1.1, 3108, Chapter 35

THIS PROPOSAL IS ON THE AGENDA OF THE IBC STRUCTURAL AND THE IBC GENERAL CODE DEVELOPMENT COMMITTEES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

Proponent: Thomas Hoenninger, Stainless LLC, representing the TIA Subcommittee TR14.7

PART I – IBC STRUCTURAL

1. Revise as follows:

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

Exceptions:

1. Subject to the limitations of Section 1609.1.1.1, the provisions of SBCCI SSTD 10 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 W WFCM.
3. Designs using NAAMM FP 1001.
4. Designs using ~~TIA/EIA-222~~ TIA-222 for antenna-supporting structures and antennas.

3108.4 Loads. Towers shall be designed to resist wind loads in accordance with ~~TIA/EIA-222~~ TIA-222. Consideration shall be given to conditions involving wind load on ice-covered sections in localities subject to sustained freezing temperatures.

2. Delete and substitute standard in Chapter 35 as follows:

~~TIA/EIA-222-F-96~~ ~~Structural Standard for Antenna Supporting Structures and Antennas~~

TIA-222-G-2005 Structural Standard for Antenna Supporting Structures and Antennas

PART II – IBC GENERAL

Delete and substitute as follows:

~~SECTION 3108~~
~~RADIO AND TELEVISION TOWERS~~

~~**3108.1 General.** Subject to the provisions of Chapter 16 and the requirements of Chapter 15 governing the fire-resistance ratings of buildings for the support of roof structures, radio and television towers shall be designed and constructed as herein provided.~~

~~**3108.2 Location and access.** Towers shall be located and equipped with step bolts and ladders so as to provide ready access for inspection purposes. Guy wires or other accessories shall not cross or encroach upon any street or other public space, or over above-ground electric utility lines, or encroach upon any privately owned property without written consent of the owner of the encroached-upon property, space or above-ground electric utility lines.~~

~~**3108.3 Construction.** Towers shall be constructed of approved corrosion-resistant noncombustible material. The minimum type of construction of isolated radio towers not more than 100 feet (30 480 mm) in height shall be Type IIB.~~

~~**3108.4 Loads.** Towers shall be designed to resist wind loads in accordance with TIA/EIA-222. Consideration shall be given to conditions involving wind load on ice-covered sections in localities subject to sustained freezing temperatures.~~

~~**3108.4.1 Dead load.** Towers shall be designed for the dead load plus the ice load in regions where ice formation occurs.~~

~~**3108.4.2 Wind load.** Adequate foundations and anchorage shall be provided to resist two times the calculated wind load.~~

~~**3108.5 Grounding.** Towers shall be permanently and effectively grounded.~~

SECTION 3108
TELECOMMUNICATION AND BROADCAST TOWERS

3108.1 General. Towers shall be designed and constructed in accordance with the provisions of TIA-222.

3108.2 Location and access. Towers shall be located such that guy wires and other accessories shall not cross or encroach upon any street or other public space, or over above-ground electric utility lines, or encroach upon any privately owned property without the written consent of the owner of the encroached-upon property, space or above-ground electric utility lines. Towers shall be equipped with climbing and working facilities in compliance with TIA-222. Access to the tower sites shall be limited as required by applicable OSHA, FCC and EPA regulations.

Reason: (Part I) Revise outdated material.

TIA-222-G was published in August 2005 and was made effective January 1, 2006. It replaces TIA/EIA-222-F, which is no longer maintained or supported by the Telecommunications Industry Association (TIA). TIA-222-G is an ANSI approved standard.

The major changes from 222-F that are incorporated in 222-G are:

222-G is based on the ASCE 7-05 three-second gust basic wind speed map. 222-F is based on the ASCE 7-93 fastest mile basic wind speed map and results in confusion when comparing to the ASCE 7-05 and IBC2006.

222-G includes reliability classes for telecommunication and broadcast structures that correspond to the building and structure categories of ASCE 7-05. 222-F does not include reliability classes.

222-G incorporates the same exposure categories and provisions for topographic features as ASCE 7-05. 222-F does not include multiple exposure categories and provisions for topographic features.

222-G incorporates appropriate provisions for the latest AISC and ACI standards that pertain to telecommunication and broadcast structures.

222-G incorporates the ASCE 7-05 ice maps. 222-F does not include ice map data.

222-G contains a section for proper earthquake analysis and design for telecommunication and broadcast structures. 222-F does not include earthquake provisions.

222-G contains updated, comprehensive provisions for climbing and working facilities.

(Part II) TIA-222-G is the current standard and was published in August 2005 and was made effective January 1, 2006. This is the structural standard for antenna supporting structures and antennas and is ANSI approved. IBC2006 references TIA/EIA-222-F, which is an outdated TIA standard.

The title “Telecommunication and Broadcast Towers” was substituted for “Radio and Television towers” because TIA-222 applies to more than just radio and television towers.

Section 3108.1 was substituted because it is clearer and more concise language. Deleted the reference to Chapter 15 because it does not apply.

Section 3108.2 was substituted because it is clearer and more concise language.

Sections 3108.3, 3108.4 and 3108.5 were deleted because the language in the sections either does not apply or it is covered in TIA-222.

Cost Impact: In general, the code change proposal will not increase the cost of construction. However, some specific tower projects may experience an increase in construction cost.

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

PART I – IBC STRUCTURAL

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

PART I – IBC GENERAL

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

S16–06/07
1609.1.1

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

1. Revise as follows:

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

Exceptions:

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

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

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

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

2. Add standard to Chapter 35 as follows:

ASCE xx-yy Wind Tunnel Testing

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

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

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

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

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

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

The purpose of this change is 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.

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

Bibliography:

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

National Institute of Standards and Technology. 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: Results of review of the proposed standard(s) will be posted on the ICC website by August 20, 2006.

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

S17–06/07
1609.1.1, 1609.1.1.2 through 1609.1.1.2.2 (New)

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

1. Revise as follows:

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

Exceptions:

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

2. Add new text as follows:

<u>**1609.1.1.2 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.</u>

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

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

Reason: This code change brings forward recommendations currently in the ASCE 7-05 commentary and gives the limitations the force of code provisions. Recent comparisons between wind tunnel studies for the same building have demonstrated a difference of up to 40% in results between laboratories. These provisions will provide a limit on reductions that will provide a baseline threshold value. This is being proposed in the IBC at this time because it is our understanding that ASCE 7 will not be revised again until 2010.

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

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

S18–06/07
1609.1.2

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

Revise as follows:

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

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

Exceptions:

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

Reason: Substitute revised material for current provision of the Code.

ASCE 7-98 and ASCE 7-02 require that "Glazing in the lower 60 ft. of Category II, III, or IV buildings sited in wind borne debris regions be impact resistant glazing or protected with an impact resistant covering. Alternatively, if these criteria are not met, the glazed opening must be considered to be "open" (not having any covering) if it receives positive external pressure, thus potentially changing the design of the building from an "enclosed building" to one that is "open" or "partially enclosed", depending on the size and number of openings. Generally this would mean that the interior walls would be designed for nearly the same wind pressures as the external walls. More importantly, even though the building can be designed to sustain the higher wind pressures, the interior of the building and its contents are subject to major damage from wind and wind-driven rain should the glazing be broken.

In the 2002 edition of ASCE 7, the language was changed to recognize the higher importance of certain structures. In all Category IV structures, and in Category II or III buildings used for health care, jail and detention facilities, power generating and other public utility facilities, glazing in the lower 60 ft. of the structure sited in wind borne debris regions was required to have either impact resistant glazing or be protected with an impact resistant covering, meeting the test requirements of ASTM E 1996. For glazed openings less than 30 feet above the ground, the Large Missile Test requirements apply. For Category II or III buildings with uses other than those enumerated above, openings in the lower 60 feet of the building could be left unprotected, provided that an unprotected opening that received positive external pressure was considered an opening for purposes of determining the building's enclosure classification.

ASCE 7-05 has been further changed to require glazing in all Category II, III or IV buildings to be impact-resistant glazing or protected with an impact-resistant covering if it is located as follows: in the lower 60 feet of the building, and equal to or less than 30 feet above an aggregate surfaced roof within 1500 feet of the building. The provision of ASCE 7-02 that permitted the glazed opening to be considered an opening for purposes of determining the enclosure classification of the building has been removed.

During the development of the IBC 2000 when the provisions of ASCE 7-98 were being considered, the home building industry successfully lobbied for an exception that allowed any one- or two-story building, regardless of Occupancy Category, to be constructed with neither non-impact resistant glazing nor a non-impact resistant covering provided the non-impact resistant glazing is covered with 7/16" thick wood structural panels. These panels are not required to meet either the Large or Small Missile test requirements of ASTM E 1996. The attachment of the panels are required only to meet the component and cladding wind load provisions of ASCE 7, but there is no such requirement for the panels themselves. In addition, the panels are allowed to span as far as 8 ft. without any stiffeners if the panel itself cannot meet the component and cladding wind pressure provisions of ASCE 7 for the design wind speed. The only additional requirement for the panels is that they are fastened at the edges.

These wood structural panels do not afford the same level of protection as impact resistant coverings (i.e., hurricane shutters), which have met the Large Missile Impact requirements of ASTM E 1996. Further, there is no recognition of the higher importance of health care facilities, jails, public utility facilities, etc. in the IBC requirements. While the use of wood structural panels (e.g., plywood and OSB) may be adequate for the protection of openings in one- and two-family dwellings; the use of these panels, without more stringent requirements for their attachment and intervals of support, is not adequate for health care facilities, facilities where the occupants have limited mobility, and other facilities where the panels may not be installed prior to arrival of the hurricane.

For these reasons, the proposed change limits the use of the wood structural panels to Group R-3 and R-4 buildings, so that the intent of ASCE 7 to provide a higher level of protection for all other building occupancy groups is maintained.

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

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

S19–06/07

1609.1.2, Table 1602.1.2; IRC R301.2.1.2, Table R301.2.1.2

THIS PROPOSAL IS ON THE AGENDA OF THE IBC STRUCTURAL AND THE IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

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

PART I – IBC

Revise as follows:

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

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

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

   Exceptions:

   1. Wood structural panels with a minimum thickness of 7/16 inch (11 mm) and a maximum span of 8 feet (2438 mm) shall be permitted for opening protection in one- and-two-story buildings. Panels shall be pre-cut so that they shall be attached to the framing surrounding the opening containing the product with the glazed opening. Panels shall be predrilled as required for the anchorage method and shall be secured with the attachment hard ware provided. Attachments shall be designed to resist the component and cladding loads determined in accordance with ASCE 7, with permanent corrosion resistant attachment hardware provided and anchors permanently installed on the building. Attachment in accordance with Table 1609.1.2 with permanent corrosion resistant attachment hardware provided and anchors permanently installed on the building is permitted for buildings with a mean roof height of 33 feet (10 058 mm) or less where wind speeds do not exceed 130 miles per hour (58 m/s).
   2. Glazing in Occupancy Category I buildings as defined in Section 1604.5, including greenhouses that are occupied for growing plants on a production or research basis, without public access shall be permitted to be unprotected.
   3. Glazing in Occupancy Category II, III or IV buildings located over 60 feet (18 288 mm) above the ground and over 30 feet (9144 mm) above aggregate surface roofs located within 1,500 feet (458 m) of the building shall be permitted to be unprotected

TABLE 1609.1.2
WIND-BORNE DEBRIS PROTECTION FASTENING SCHEDULE
FOR WOOD STRUCTURAL PANELS[a,b,c,d]

FASTENER TYPE	FASTENER SPACING (in.)		
	Panel span ≤ 4 foot	4 feet < panel span ≤ 6 feet	6 feet < panel span ≤ 8 feet
~~No. 6 Screws~~ No. 8 Wood Screw based anchor with 2-inch embedment length	16	~~12~~ 10	~~9~~ 8
~~No. 8 Screws~~ No. 10 Wood Srew based anchor with 2-inch embedment length	16	~~16~~ 12	~~12~~ 9
¼ Lag screw based anchor with 2-inch embedment length	16	16	16

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound = 4.448N,
1 mile per hour = 0.447 m/s.

a. This table is based on 130 mph wind speeds and a 33-foot mean roof height.
b. Fasteners shall be installed at opposing ends of the wood structural panel. Fasteners shall be located a minimum of 1 inch from the edge of the panel.
c. Anchors shall penetrate through the exterior wall covering with an embedment length of 2 inches minimum into the building frame. ~~Fasteners shall be long enough to penetrate through the exterior wall covering and a minimum of 1 ¼ inches~~ into wood wall framing and ~~a minimum of1 ¼ inches into~~ concrete block or concrete. ~~, and into steel framing a minimum of 3 exposed threads.~~ Fasteners shall be located a minimum of 2 ½ inches from the edge of concrete block or concrete.
d. Where panels ~~screws~~ are attached to masonry or masonry/stucco, they shall be attached using vibration-resistant anchors having a minimum ultimate withdrawal capacity of 1500 ~~490~~ pounds.

PART II – IRC

Revise as follows:

R301.2.1.2 Protection of openings. Windows in buildings located in windborne debris regions shall have glazed openings protected from windborne debris. Glazed opening protection for windborne debris shall meet the requirements of the Large Missile Test of an approved impact resisting standard or ASTM E 1996 and ASTM E 1886 referenced therein.

Exception: Wood structural panels with a minimum thickness of 7/16 inch (11 mm) and a maximum span of 8 feet (2438 mm) shall be permitted for opening protection in one- and-two-story buildings. Panels shall be pre-cut so that they shall be attached to the framing surrounding the opening containing the product with the glazed opening. Panels shall be predrilled as required for the anchorage method and shall be secured with the attachment hard ware provided. Attachments shall be designed to resist the component and cladding loads determined in accordance with either Table R301.2(2) or Section 1609.6.5 of the *International Building Code*, with permanent corrosion resistant attachment hardware provided and anchors permanently installed on the building. Attachment in accordance with Table R301.2.1.2 is permitted for buildings with a mean roof height of 33 feet (10 058 mm) or less where wind speeds do not exceed 130 miles per hour (58 m/s).

TABLE R301.2.1.2
WIND-BORNE DEBRIS PROTECTION FASTENING SCHEDULE
FOR WOOD STRUCTURAL PANELS[a,b,c,d]

FASTENER TYPE	FASTENER SPACING (in.)[1,2]		
	Panel span ≤ 4 foot	4 feet < panel span ≤ 6 feet	6 feet < panel span ≤ 8 feet
~~No. 6 Screws~~ No. 8 Wood Screw based anchor with 2-inch embedment length	16	~~12~~ 10	~~9~~ 8
~~No. 8 Screws~~ No. 10 Wood Srew based anchor with 2-inch embedment length	16	~~16~~ 12	~~12~~ 9
¼ Lag screw based anchor with 2-inch embedment length	16	16	16

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound = 4.448N,
1 mile per hour = 0.447 m/s.

a. This table is based on 130 mph wind speeds and a 33-foot mean roof height.
b. Fasteners shall be installed at opposing ends of the wood structural panel. Fasteners shall be located a minimum of 1 inch from the edge of the panel.
c. Anchors shall penetrate through the exterior wall covering with an embedment length of 2 inches minimum into the building frame. ~~Fasteners shall be long enough to penetrate through the exterior wall covering and a minimum of 1 ¼ inches~~ into wood wall framing and ~~a minimum of1 ¼ inches into~~ concrete block or concrete. ~~, and into steel framing a minimum of 3 exposed threads.~~ Fasteners shall be located a minimum of 2 ½ inches from the edge of concrete block or concrete.
d. Where panels ~~screws~~ are attached to masonry or masonry/stucco, they shall be attached using vibration-resistant anchors having a minimum ultimate withdrawal capacity of 1500 ~~490~~ pounds.

Reason: The purpose of this code change is primarily to require permanently mounted hardware when using wood structural panel shutters for window protection for new construction. It is our belief that using wood structural panels as window protection in the manner currently prescribed by the code, is basically an emergency option for protection of existing buildings where the homeowner does not have some permanent shutter system in place.

While the code requires the panels to be precut and the attachment hardware provided, there are potentially many logistical problems with homeowners actually installing the panels as required by the code. It's not clear that the homeowners will be sufficiently instructed on (or remember at a later date) how to attach the panels, in particular using the prescribed minimum spacing. Additionally, it can be extremely cumbersome to attempt to nail a sheet of plywood over a window, particularly on the second story of a building. Additionally, we are concerned about the capacity of nailed connections where the nails are installed in the same hole repeatedly.

This proposed change also increases the minimum required capacity of masonry anchors from 490 lbs to 1500 lbs. Evaluation reports (ICC, NES, and SBCCI) for masonry anchors require a Factor of Safety (FS) of 4.0 if a special inspection is performed on the anchor installation. Without a special inspection, the reports require a FS of 8.0. Based on the load conditions specified, the 490 lb required capacity implies a FS of 2.5. We do not believe that special inspections are or will be performed on these anchors. Therefore, raising the required capacity of the masonry anchors to 1500 lbs provides a FS more in line with the evaluation reports for masonry anchors.

At the time of preparation of this proposal, the Florida Building Commission Structural Technical Advisory Committee unanimously approved this code change for the 2006 glitch amendment cycle.

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

PART I – IBC

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

PART II – IRC

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

S20–06/07

1609.1.2.2, Chapter 35

Proponent: Joseph R. Hetzel, P.E., Door & Access Systems Manufacturers Association

1. Add new text as follows:

1609.1.2.2 Garage doors. Garage door glazed opening protection for wind-borne debris shall meet the requirements of an approved impact resisting standard or ANSI/DASMA 115.

2. Add standard to Chapter 35 as follows:

DASMA
ANSI/DASMA 115-03, Standard Method for Testing Garage Doors: Determination of Structural Performance Under Missile Impact and Cyclic Wind Pressure