IBC – STRUCTURAL

Code Change No: S1-06/07

Original Proposal

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~~ <u>containing</u> 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~~ <u>containing adult education facilities, such as</u> colleges ~~or adult education facilities~~ <u>and universities, with an occupant load greater than 500.</u> • ~~Health care facilities~~ <u>Group I-2 occupancies</u> with an occupant load of 50 or more resident patients, but not having surgery or emergency treatment facilities. • ~~Jails and detention facilities~~ <u>Group I-3 occupancies.</u> • 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~~ <u>Group I-2 occupancies</u> 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<u>s</u>, and operation<u>s</u> 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 Results

Committee Action: **Approved as Submitted**

Committee Reason: Agreement with the proponent's reason which indicates that the proposal provides editorial cleanup as well as important coordination with the occupancy classifications in Chapter 3 of the IBC.

Assembly Action: **None**

Final Hearing Results

S1-06/07 **AS**

Code Change No: S3-06/07

Original Proposal

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~~ <u>Buildings and other</u> 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 <u>facilities</u> 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<u>, ambulance</u> 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~~ <u>storage</u> facilities <u>and pump structures</u> 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 Results

Errata: Under Occupancy Category III, change the seventh bulleted item to read as follows:

Power-generating stations, water treatment facilities for potable water, waste water treatment facilities and other public utility facilities not included in Occupancy Category IV.

Committee Action: **Approved as Submitted**

Committee Reason: The proposal makes a minor change in the wording of the Occupancy Category table to provide agreement with the ASCE 7 load standard.

Assembly Action: **None**

Final Hearing Results

S3-06/07 **AS**

Code Change No: S4-06/07

Original Proposal

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<u>[a]</u>

(Portions of table not shown do not change)

<u>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.</u>

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 Results

Committee Action: **Approved as Modified**

Modify proposal 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 <u>public assembly</u> ~~Group A1, A2, A3, or A4~~ with an occupant load greater than 300 • Buildings and other structures <u>with</u> ~~containing Group E occupancies~~ <u>elementary school, secondary school or day care facilities</u> 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 <u>for colleges or adult education</u> • ~~Buildings and other structures containing Group I-2~~ Healthcare facilities ~~which provide care on a 24 hour basis for more than~~ <u>with an occupant load of</u> 50 or more <u>resident</u> patients but ~~which do~~ not <u>having</u> ~~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.~~ <u>Any other occupancy</u> 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)

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.

Committee Reason: The proposal clarifies the calculation of occupant load for the purpose of Occupancy Category III determination where an occupant load exceeds 5000. The modification retains the current code text in favor of the other proposed clarifications to Occupancy Category III thresholds.

Assembly Action: **None**

Final Hearing Results

S4-06/07 **AM**

Code Change No: S7-06/07

Original Proposal

Section: 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~~ increases<u>s</u> 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 Results

Committee Action: **Approved as Submitted**

Committee Reason: This change clarifies the application of wood stress adjustments that are permitted when using the basic allowable stress load combinations.

Assembly Action: **None**

Final Hearing Results

S7-06/07 **AS**

Code Change No: S8-06/07

Original Proposal

Sections: 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.~~

$\cancel{1.2D + f_1L + E_m}$ ~~**(Equation 16-22)**~~
$\cancel{0.9D + E_m}$ ~~**(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 Results

Committee Action: **Approved as Modified**

Modify proposal 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~~ , 1605.3.1 or 1605.3.2 and Chapters 18 through 23, and the ~~special seismic~~ overstrength factor load combinations of Section 12.4.3.2 of ASCE 7 where required by Section 12.3.3.3 or 12.10.2.1 of ASCE 7. With the simplified procedure of ASCE 7 Section 12.14, the overstrength factor load combinations of Section 12.14.3.2 of ASCE 7 shall be used. Applicable loads shall be considered, including both earthquake and wind, in accordance with the specified load combinations. Each load combination shall also be investigated with one or more of the variable loads set to zero.

Committee Reason: This proposal clarifies application of the special seismic load combinations when using allowable stress design by referring to ASCE 7. The modification substitutes the ASCE 7 term "overstrength factor load combinations" for consistency with that document.

Assembly Action: **None**

Public Comments

Individual Consideration Agenda

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

Public Comment 2:

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

Modify proposal as follows:

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

1. The load combinations specified in Section 1605.2, 1605.3.1 or 1605.3.2, ~~and~~

2. The load combinations specified in Chapters 18 through 23, and
3. The ~~overstrength factor~~ load combinations ~~of~~ with overstrength factor specified in Section 12.4.3.2 of ASCE 7 where required by Section 12.3.3.3 or 12.10.2.1 of ASCE 7.

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

The load combinations with overstrength factor in Section 12.4.3.2 of ASCE 7 shall be used in lieu of the following:

1. The load combinations for strength design in lieu of Equations 16-5 and 16-7 in Section 1605.2.1.
2. The load combinations for allowable stress design in lieu of Equations 16-12, 16-13 and 16-15 in Section 1605.3.1.
3. The load combinations for allowable stress design in lieu of Equations 16-20 and 16-21 in Section 1605.3.2.

Commenter's Reason: The purpose for this public comment is the same as for Public Comment #1 except it takes into account the apparent intent of the proponent that the load combinations for allowable stress design with overstrength factor in Section 12.4.3.2 of ASCE 7 shall substitute for the seismic load combinations in IBC Section 1605.3.2 (Alternative ASD) as well as IBC Section 1605.3.1 (Basic ASD).

Public Comment 3:

W. Lee Shoemaker, P.E., Thomas Associates, Inc, representing Metal Building Manufacturers Association Inc, requests Approval as Modified by this public comment.

Modify proposal 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, 1605.3.1, or 1605.3.2 and Chapters 18 through 23, and the overstrength factor load combinations of Section 12.4.3.2 of ASCE 7 where required by Section 12.2.5.2, 12.3.3.3 or 12.10.2.1 of ASCE 7. With the simplified procedure of ASCE 7 Section 12.14, the overstrength factor load combinations of Section 12.14.3.2 of ASCE 7 shall be used. Applicable loads shall be considered, including both earthquake and wind, in accordance with the specified load combinations. Each load combination shall also be investigated with one or more of the variable loads set to zero.

Commenter's Reason: After the Public Hearings in Florida, it was realized that there was one additional Section in ASCE 7 where the special load combinations are referenced. For consistency and completeness, Section 12.2.5.2 should also be added as noted above.

Final Hearing Results

S8-06/07 **AMPC 2, 3**

Code Change No: S9-06/07

Original Proposal

Sections: 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)	60	____
Follow spot, projections, and control rooms	50	
Lobbies	100	
Movable seats	100	
Stages and platforms	125	
Other assembly areas	100	
5. Balconies (exterior) and decks[h]	~~100~~	____
~~On one- and two-family residences only, and not exceeding 100 sq ft~~	~~60~~	
	Same as occupancy served	
~~9. Decks~~	~~Same as occupancy served[h]~~	____
28. Residential		
One- and two-family dwellings		
Uninhabitable attics without storage[i]	10	
Uninhabitable attics with storage[i, j, k]	20	
Habitable attics and sleeping areas	30	____
All other areas ~~except balconies and decks~~	40	
Hotels and multifamily dwellings		
Private rooms and corridors serving them	40	
Public rooms and corridors serving them	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.

Public Hearing Results

PART I – IBC STRUCTURAL
Committee Action: Disapproved

Committee Reason: This code change was disapproved because the revision made by S10-06/07 was preferred.

Assembly Action: None

PART II - IRC
Committee Action: Approved as Submitted

Committee Reason: This change serves to eliminate the differences between balcony and deck live loads and adds needed clarity to the code language.

Assembly Action: None

Public Comments

Individual Consideration Agenda

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

Public Comment:

Gary Ehrlich, P.E., National Association of Home Builders, requests Approval as Submitted for Part I.

Commenter's Reason: As discussed at the hearings in Orlando, there has not been substantial evidence of deck or balcony failures due to overloading of the floor joists, girders, or piers. In fact, the evidence presented by NCSEA suggests that decks and balconies only see a live load of 20 psf even in a heavily-loaded state. The documented failures have been at the deck ledger connection to the floor framing and would likely have happened regardless of the design live load for decks and balconies. This proposal would allow decks in R-3 and R-4 occupancies and IRC dwellings forced to use the IBC provisions to be designed for a 40psf live load just as they would under the IRC. This would allow an engineer to make use of design aids developed for 40psf deck live loads such as the residential ledger table being implemented in the IRC. NAHB asks for your support in approving this proposal as submitted and reversing the committee’s action.

Final Hearing Results

S9-06/07, Part I	**AS**
S9-06/07, Part II	**AS**

Code Change No: S14-06/07

Original Proposal

Section: 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 Results

Committee Action: **Approved as Submitted**

Committee Reason: This proposal deleted the permitted stress increases for guards and handrails because they are no longer needed with the unified ASD/LRFD formats provided in the material standards.

Assembly Action: **None**

Final Hearing Results

S14-06/07 **AS**

Code Change No: S15-06/07

Original Proposal

Sections: 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.

Public Hearing Results

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

Analysis: Review of proposed standard indicated that, in the opinion of ICC staff, the standard did comply with ICC criteria for referenced standards.

PART I — IBC STRUCTURAL
Committee Action: **Approved as Submitted**

Committee Reason: This code change makes an appropriate update to the latest edition of TIA-222 for antenna-supporting structures.

Assembly Action: **None**

PART II — IBC GENERAL
Committee Action: **Approved as Submitted**

Committee Reason: The revision of Section 3108 as proposed correlates with what is actually being applied in the field. In addition the proposal removes non building code related issues and leaves such issues to the standard itself.

Assembly Action: **None**

Final Hearing Results

S15-06/07, Part I	**AS**
S15-06/07, Part II	**AS**

Code Change No: S17-06/07

Original Proposal

Sections: 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</u>

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.

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 Results

Committee Action: **Approved as Modified**

Modify proposal 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 R2 and R-3 buildings.
2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of the AF&PA WFCM.
3. Designs using NAAMM FP 1001.
4. Designs using TIA/EIA-222 for antenna-supporting structures and antennas.
5. Wind Tunnel tests in accordance with Section 6.6 of ASCE 7, subject to the limitations in Section 1609.1.1.2.

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.

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

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

Committee Reason: This proposal implements in the code the limitations on wind load testing that are currently noted in the commentary to ASCE 7. The modification changes wind pressure to overturning moment to address that specific requirement.

Assembly Action: **None**

Final Hearing Results

S17-06/07 **AM**

Code Change No: S18-06/07

Original Proposal

Section: 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 <u>classified as Group R-3 or R-4 occupancy</u>. 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 Results

Committee Action: **Approved as Submitted**

Committee Reason: This code change places an appropriate limit on the prescriptive opening protection option utilizing wood structural panels by limiting their use to Groups R-3 and R-4.

Assembly Action: **None**

Final Hearing Results

S18-06/07 **AS**

Code Change No: S19-06/07

Original Proposal

Sections: 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.

Public Hearing Results

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

Modify proposal 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~~ 45 feet (~~10 058~~ mm) or less where wind speeds do not exceed ~~130~~ 140 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. 8 Wood Screw based anchor with 2-inch embedment length	16	10	8
No. 10 Wood Screw based anchor with 2-inch embedment length	16	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~~ 140 mph wind speeds and a ~~33~~ 45-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. ~~into wood wall framing and concrete block or concrete~~. Fasteners shall be located a minimum of 2 ½ inches from the edge of concrete block or concrete.
d. Where panels are attached to masonry or masonry/stucco, they shall be attached using vibration-resistant anchors having a minimum ultimate withdrawal capacity of 1500 pounds.

Committee Reason: The proposal makes clarifications to the prescriptive option for protection of glazed openings and specifically requires permanent anchorage to be provided. The modification extends the wind speed and roof height limits to be consistent with the revised fastener spacing. The word permanent immediately preceeding “corrosion resistant” was also deleted to avoid confusion.

Assembly Action: None

PART II – IRC
Committee Action: Disapproved

Committee Reason: There was insufficient technical data to support this change. A safety factor of 8 would be excessive. If this proposal were passed it would no longer allow the use of masonry screws. In addition, the increase in cost predicted to be from 33 to 53 percent was not justified.

Assembly Action: None

Final Hearing Results

S19-06/07, Part I AM
S19-06/07, Part II AS

Code Change No: S20-06/07

Original Proposal

Section: 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

Reason: The purpose of this proposed code change is to reference an ANSI standard published specifically for the windborne debris resistance testing of garage doors. ANSI/DASMA 115 should be the primary standard referenced for this purpose. Other standards exist that could be deemed "approved impact resisting standards", including ASTM E 1886 / ASTM E 1996 and TAS 201 / TAS 203. It should be noted that ASTM E 1886 and ASTM E 1996 require interpretation regarding their use with garage doors. ASTM E6.51.17 (impact resistance task group) has not developed specific references to garage doors in those standards because of their awareness of the existence of, and industry use of, ANSI/DASMA 115.

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 Results

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

Analysis: Review of proposed new standard indicated that, in the opinion of ICC staff, the standard did comply with ICC criteria for referenced standards.

Committee Action: **Approved as Submitted**

Committee Reason: This code change adds a needed reference standard for protection of garage door openings.

Assembly Action: **None**

Final Hearing Results

S20-06/07 **AS**

Code Change No: S22-06/07

Original Proposal

Sections: 1612.3.1 (New), 106.2.1 (New)

Proponent: Rebecca C. Quinn, RCQuinn Consulting, Inc., representing US Dept. of Homeland Security, Federal Emergency Management Agency

Revise as follows:

1612.3 Establishment of flood hazard areas. To establish flood hazard areas, the governing body shall adopt a flood hazard map and supporting data. The flood hazard map shall include, at a minimum, areas of special flood hazard as identified by the Federal Emergency Management Agency in an engineering report entitled "The Flood Insurance Study for (insert Name of Jurisdiction)," dated (insert date of issuance), as amended or revised with the accompanying Flood Insurance Rate Map (FIRM) and Flood Boundary and Floodway Map (FBFM) and related supporting data along with any revisions thereto. The adopted flood hazard map and supporting data are hereby adopted by reference and declared to be part of this Section.

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

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

106.2 Site plan. The construction documents submitted with the application for permit shall be accompanied by a site plan showing to scale the size and location of new construction and existing structures on the site, distances from lot lines, the established street grades and the proposed finished grades and, as applicable, flood hazard areas, floodways, and design flood elevations; and it shall be drawn in accordance with an accurate boundary line survey. In the case of demolition, the site plan shall show construction to be demolished and the location and size of existing structures and construction that are to remain on the site or plot. The building official is authorized to waive or modify the requirement for a site plan when the application for permit is for alteration or repair or when otherwise warranted.

106.2.1 Design flood elevations. Where design flood elevations are not specified, they shall be established in accordance with Section 1612.3.1.

Reason: The purpose of this code change proposal is to clarify how design flood elevations are to be obtained or determined for those flood hazard areas shown on community flood hazard maps that do not have such flood elevation already specified.

This proposed code change to Section 1612.3 clarifies the authority of the code official to require use of other data which may be obtained from other sources, or to require the applicant to develop flood hazard data. Section 106.2 requires that the construction documents submitted are to be accompanied by a site plan. The site plan is to show, as applicable "flood hazard areas, floodways, and design flood elevations." As written, there appears to be an assumption in Sec. 106.2 and in Section 1612 that flood elevations and floodway designations are available on all flood hazard maps. A large percentage of areas that are mapped as special flood hazard area by the National Flood Insurance Program do not have flood elevations and/or do not have floodway designations (floodways are areas along riverine bodies of water that convey the bulk of floodwaters). The language in this code change proposal parallels language in the IRC at R106.1.3 and R324.1.3.1 (Determination of design flood elevations).

The technical information used to substantiate this proposal is the NFIP regulation §60.3(b)(4).

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: Where design flood elevations are not established, this proposal gives the needed guidance for making that determination and grants the building official the necessary authority to require that determination to be made.

Assembly Action: **None**

Final Hearing Results

S22-06/07 **AS**

Code Change No: S23-06/07

Original Proposal

Section: 1613.1

Proponent: Phillip A. Brown, American Fire Sprinkler Association

Revise as follows:

1613.1 Scope. Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7, excluding Chapter 14 and Appendix 11A. The seismic design category for a structure is permitted to be determined in accordance with Section 1613 or ASCE 7.

Exceptions:

1. Detached one- and two-family dwellings, assigned to Seismic Design Category A, B or C, or located where the mapped short-period spectral response acceleration, S_S, is less than 0.4 g.
2. The seismic-force-resisting system of wood-frame buildings that conform to the provisions of Section 2308 are not required to be analyzed as specified in this section.
3. Agricultural storage structures intended only for incidental human occupancy.
4. Structures that require special consideration of their response characteristics and environment that are not addressed by this code or ASCE 7 and for which other regulations provide seismic criteria, such as vehicular bridges, electrical transmission towers, hydraulic structures, buried utility lines and their appurtenances and nuclear reactors.
5. <u>Automatic fire sprinkler systems, designed and constructed in accordance with NFPA 13 shall be deemed to meet the requirements of ASCE 7.</u>

Reason: The purpose of the code change proposal is to add a new exception to the code. The proposed exception will bring the IBC to the forefront of the latest developments in earthquake protection for automatic fire sprinkler systems. With the help of professionals involved with the NEHRP and ASCE 7, NFPA 13 has undergone extensive enhancements to its earthquake protection criteria, as is evident in the tentative interim amendment to the 2002 edition and the forthcoming 2006 edition criteria. The new criteria meet or exceed ASCE 7 requirements with regard to force and displacement. As such, ASCE 7, in its interim update process, will defer to NFPA 13 without caveat for earthquake protection of fire sprinkler systems.

The NFPA Technical Committee on Hanging and Bracing of Water-Based Fire Protection Systems made several modifications to Section 9.3.5 to ensure that the seismic brace criteria within NFPA 13 would properly align with the requirements and permitted limits of ASCE 7. These changes, in combination with the initial TIA, and the Report on Proposals will ensure that NFPA 13 is applicable for all seismic applications, In addition, the changes provide a simplified method to meet the requirements of ASCE 7 without having to develop a complete engineering analysis. These requirements do not prohibit an engineer from doing a complete design and analysis in compliance with ASCE 7 requirements, but have been developed to address the requirements of ASCE 7 and present the material in a way that allows for the requirements to remain in NFPA 13 for seismic design of sprinkler systems. Additionally, these requirements have been developed to provide as much material as possible in NFPA 13 while limiting the amount of required information needed from outside sources.

Bibliography: 13-259a Log #CC101 AUT-HBS NFPA 13 Report on Comments A2006 – Copyright NFPA

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

Public Hearing Results

Committee Action: **Approved as Modified**

Replace proposal with the following:

<u>**1613.6.3 Automatic fire sprinkler systems.** Automatic fire sprinkler systems designed and installed in accordance with NFPA 13 shall be deemed to meet the requirements of Section 13.6.8 of ASCE 7.</u>

Chapter 35 NFPA
13-~~02~~ <u>07</u> Installation of Sprinkler Systems

Committee Reason: The proposal recognizes automatic sprinkler systems installed under the 2007 edition of NFPA 13 as complying with the ASCE 7 seismic load provisions. The modification places this provision in a more appropriate code section.

Assembly Action: **None**

Final Hearing Results

S23-06/07 **AM**

Code Change No: S24-06/07

Original Proposal

Sections: 1613.6, 2101.2.2

Proponent: Ronald E. Barnett, AERCON Florida, LLC, representing Autoclaved Aerated Concrete Products Association

1. Add new text as follows:

1613.6.3 Autoclaved aerated concrete (AAC) masonry shear wall design coefficients and system limitations. Add the following text at the end of Section 12.2.1 of ASCE 7:

For ordinary reinforced AAC masonry shear walls used in the seismic force-resisting system of structures, the response modification factor, R, shall be permitted to be taken as 3, the deflection application factor, C_d, shall be permitted to be taken as 3, and the system overstrength factor, Ω_o, shall be permitted to be taken as 2½. The maximum height for ordinary reinforced AAC masonry shear walls shall not be limited for buildings assigned to Seismic Design Category B , shall be limited to 160 feet (48768 mm) for buildings assigned to Seismic Design Category C, shall be limited to 65 feet (19812 mm) for buildings assigned to Seismic Design Category D, and is not permitted for buildings assigned to Seismic Design Categories E and F.

For ordinary plain (unreinforced) AAC masonry shear walls used in the seismic force-resisting system of structures, the response modification factor, R, shall be permitted to be taken as 1½, the deflection application factor, C_d, shall be permitted to be taken as 1½, and the system overstrength factor, Ω_o, shall be permitted to be taken as 2½. The maximum height for ordinary plain (unreinforced) AAC masonry shear walls shall not be limited for buildings assigned to Seismic Design Category B and is not permitted for buildings assigned to Seismic Design Category B and is not permitted for buildings assigned to Seismic Design Categories C, D, E and F..

2. Revise as follows:

2101.2.2 Strength design. Masonry designed by the strength design method shall comply with the provisions of Sections 2106 and 2108, except that AAC masonry shall comply with the provisions of Section 2106, Section 1613.6.3, and Chapter 1 and Appendix A of ACI 530/ASCE 5/TMS 402. ~~AAC Masonry shall not be used in the seismic force-resisting system of structures classified as Seismic Design Category B, C, D, E or F.~~

Reason: The sentence "AAC Masonry shall not be used in the seismic force resisting system of structures classified as Seismic Design Category B, C, D, E or F." is not in the 2005 MSJC *Code and Specification*. It was added during the 2005 ICC Structural Hearings in Cincinnati through a floor amendment proposed by the Code Resource Support Committee (CRSC) of the Building Seismic Safety Council (BSSC). This amendment was presented due to CRSC's concerns that the seismic provisions had not been reviewed by BSSC, as well as a specific concern over the lack of height limitations in the 2005 MSJC provisions. We bring this modified proposal to you at these hearings after amicable and productive dialog with many key members of BSSC, CRSC, and ASCE 7. Based on those discussions, this modified proposal includes prescriptive height limitations; it limits ductile reinforced AAC masonry shear-wall systems to SDC A through D; it limits unreinforced AAC masonry shear-wall systems to SDC A and B; and it prescribes seismic design factors for each system. The adoption of this proposal will permit this well-established and well-documented structural system to be used in the U.S.A. as it is in Europe, Japan and elsewhere. Since the ASCE 7 cycle is not complete, we are proposing to add information on the seismic factors to Section 1613.6 Alternatives to ASCE 7 as a temporary measure until they can be incorporated into ASCE 7. This is not a unique situation; the proposed exception would join two other exceptions to ASCE 7 in that section. Through ASCE 7 itself, and through the BSSC PUC working group charged with coordination with ASCE 7, we intend to work for direct inclusion of these seismic design factors into ASCE 7 as soon as the ASCE 7 cycle permits**.**

The proposed modification is based on the results of the ANSI-consensus provisions of the 2005 MSJC *Code and Specification*, including its provisions for AAC masonry and considerable review and discussion with other committees charged with protection of the seismic safety of the public. This is a far better solution than the current prohibition outside of SDC A, which was made on procedural rather than technical grounds. Also, the proposed values of R and CD, which were determined in accordance with AC215 and already approved by ICC-ES, are consistent with the stated intentions of the Code Resource Support Committee (CRSC) of the Building Seismic Safety Council (BSSC) for development of acceptance criteria for new materials.

The first basis for our proposed modification is technical. No technical data was advanced in support of the current exclusion -- only that the intervening group had not had time to study it. They have now undertaken that study. Ample technical justification confirming the seismic reliability of AAC masonry, and supporting the proposed seismic design factors, has been published in refereed conference proceedings since June 2003. The 2005 MSJC provisions for AAC masonry are supported by three refereed journal papers, many refereed conference proceedings, and a coherent and rigorously applied body of uncontested technical information. The refereed journal papers are listed below:

Tanner, J.E., Varela, J.L., Klingner, R.E., "Design and Seismic Testing of a Two-story Full-scale Autoclaved Aerated Concrete (AAC) Assemblage Specimen," *Structures Journal*, American Concrete Institute, Farmington Hills, Michigan, vol. 102, no. 1, January - February 2005, pp. 114-119.

Tanner, J.E., Varela, J.L., Klingner, R.E., Brightman M. J. and Cancino, U., "Seismic Testing of Autoclaved Aerated Concrete (AAC) Shear Walls: A Comprehensive Review," *Structures Journal*, American Concrete Institute, Farmington Hills, Michigan, vol. 102, no. 3, May - June 2005, pp. 374-382.

Varela, J. L., Tanner, J. E. and Klingner, R. E., "Development of Seismic Force-Reduction and Displacement Amplification Factors for AAC Structures," *EERI Spectra* (accepted for publication, May 2005).

Prior to our work in developing the procedure for AAC as described in the ICC-ES AC215 document entitled “ACCEPTANCE CRITERIA FOR SEISMIC DESIGN FACTORS AND COEFFICIENTS FOR SEISMIC-FORCE-RESISTING SYSTEMS OF AUTOCLAVED AERATED CONCRETE (AAC)”, approved in October 2003, there had not been any procedure developed for the establishment of R and CD values for new materials. The values proposed in this modification for AAC were developed in accordance with the acceptance criteria approved in AC215 and are consistent with the stated intentions of the Code Resource Support Committee (CRSC) of the Building Seismic Safety Council (BSSC) for development of acceptance criteria for new materials.

As can be seen in that research, for ductile reinforced AAC masonry shear walls the requested values of 3 for the response modification factor, R and the deflection application factor, CD, are conservative, which provides an additional factor of safety beyond that which would normally be expected for any building material or system. In addition, for unreinforced AAC masonry shear walls, the requested values of 1.5 for the response modification factor, R and the deflection application factor, CD, are consistent with the values currently prescribed for unreinforced masonry.

The second basis for our proposed modification is the life safety of the public that the ICC is charged with protecting. There is no justification for denying to that public a building system that was introduced in 1929; that is demonstrably safe, environmentally friendly, energy-efficient, and comfortable to live in; and that is recognized throughout the world, including many areas of high seismic risk and strict design and construction standards.

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

Public Hearing Results

Errata: Revise Item 1 of proposal to read as follows:

1613.6.3 Autoclaved aerated concrete (AAC) masonry shear walldesign coefficients and system limitations. Add the following text at the end of Section 12.2.1 of ASCE 7:

For ordinary reinforced AAC masonry shear walls used in the seismic force-resisting system of structures, the response modification factor, R, shall be permitted to be taken as 3, the deflection amplification factor, C_d, shall be permitted to be taken as 3, and the system overstrength factor, $_o$, shall be permitted to be taken as 2½. The maximum height for ordinary reinforced AAC masonry shear walls shall not be limited for buildings assigned to Seismic Design Category B, shall be limited to 160 feet (48768 mm) for buildings assigned to Seismic Design Category C, shall be limited to 65 feet (19812 mm) for buildings assigned to Seismic Design Category D, and is not permitted for buildings assigned to Seismic Design Categories E and F.

For ordinary plain (unreinforced) AAC masonry shear walls used in the seismic force-resisting system of structures, the response modification factor, R, shall be permitted to be taken as 1½, the deflection amplification factor, C_d, shall be permitted to be taken as 1½, and the system overstrength factor, $_o$, shall be permitted to be taken as 2½. The maximum height for ordinary plain (unreinforced) AAC masonry shear walls shall not be limited for buildings assigned to Seismic Design Category B and is not permitted for buildings assigned to Seismic Design Categories C, D, E and F.

Committee Action: Approved as Modified

Modify proposal as follows:

1613.6.3 Autoclaved aerated concrete (AAC) masonry shear walldesign coefficients and system limitations. Add the following text at the end of Section 12.2.1 of ASCE 7:

For ordinary reinforced AAC masonry shear walls used in the seismic force-resisting system of structures, the response modification factor, R, shall be permitted to be taken as ~~3~~ 2, the deflection amplification factor, C_d, shall be permitted to be taken as ~~3~~ 2, and the system overstrength factor, $_o$, shall be permitted to be taken as 2½. ~~The maximum height for o~~ Ordinary reinforced AAC masonry shear walls shall not be limited in height for buildings assigned to Seismic Design Category B, shall be limited in height to ~~160~~ 35 feet (48768 mm) for buildings assigned to Seismic Design Category C, ~~shall be limited to 65 feet (19812 mm) for buildings assigned to Seismic Design Category D,~~ and ~~is~~ are not permitted for buildings assigned to Seismic Design Categories D, E and F.

For ordinary plain (unreinforced) AAC masonry shear walls used in the seismic force-resisting system of structures, the response modification factor, R, shall be permitted to be taken as 1½, the deflection amplification factor, C_d, shall be permitted to be taken as 1½, and the system overstrength factor, $_o$, shall be permitted to be taken as 2½. ~~The maximum height for o~~ Ordinary plain (unreinforced) AAC masonry shear walls shall not be limited in height for buildings assigned to Seismic Design Category B and ~~is~~ are not permitted for buildings assigned to Seismic Design Categories C, D, E and F.

2101.2.2 Strength design. Masonry designed by the strength design method shall comply with the provisions of Sections 2106 and 2108, except that AAC masonry shall comply with the provisions of Section 2106, Section 1613.6.3, and Chapter 1 and Appendix A of ACI 530/ASCE 5/TMS 402.

Committee Reason: This code change adds seismic design coefficients and limitations for autoclaved aerated masonry shear wall systems, thus extending the use of these systems in seismic applications. The modification provides that these systems will not be used in Seismic Design Category D, E or F buildings.

Assembly Action: None

Final Hearing Results

S24-06/07 AM

Code Change No: S26-06/07

Original Proposal

Section: 1613.7 (New)

Proponent: Edward A. Donoghue, Edward A. Donoghue Associates Inc., representing National Elevator Industry, Inc.

Add new text as follows:

1613.7 Modifications to ASCE 7.

1613.7.1 ASCE 7, Section 13.6.10.3 Seismic Controls. Modify ASCE 7, Section 13.6.10.3 to read as follows:

13.6.10.3 Seismic Controls. Seismic switches shall be in accordance with Section 8.4.10 of ASME A17.1.

Reason: Delete duplicate requirements.

Seismic switches and elevator operation, after switch activation, are already defined in A17.1-2004, Section 8.4.10, Emergency Operation and Signaling Devices. A17.1 and ASCE 7 seismic switch and mounting requirements are different and inconsistent. A17.1 requires mounting the switch, when exclusively to control elevators, in the machine room and, where possible, adjacent to a vertical load bearing building structural member with its axis of sensitivity in the vertical direction and set to trigger at 0.15g. Placing the switch near a vertical structural support member will prevent significant amplification of vertical motions between the foundation and seismic trigger. At small and moderate ground motions, buildings often exhibit horizontal amplifications at the top of 3 or more. Thus, with horizontal ground motions of 0.1 g, a value commonly experienced in California, the ASCE 7 seismic switch shall trigger elevator shutdowns. This requires the elevator be inspected by an elevator mechanic, adding unnecessary disruption and cost. The seismic switch used by A17.1 is designed to trigger on the P wave so that, in many cases, it will provide adequate time to stop the elevator and allow passengers to exit the elevator before severe shaking in the building starts. This earlier trigger is also more likely to allow passengers to exit the elevator prior to loss of building power, a common occurrence in moderate earthquakes. This early exit would avoid having passengers trapped in an elevator during an earthquake and the need for first responders to rescue them from the elevator.

Movement of the elevator after the seismic switch has triggered, even in "life-safety" facilities, is a significant elevator operation. It is felt that ASCE 7 does not go into enough detail of procedures for this operation. A17.1-2004, 8.4.10, has a prescribed elevator operation in the event a seismic switch or counterweight (CWT) derailment switch is activated. An activated CWT derailment switch indicates that the CWT is out of its normal running position and may be totally out of its guide rails. Upon activation of the CWT derailment switch, A17.1 does not allow movement of the car towards the CWT, in order to prevent the possible collision of the elevator car and CWT. There could be great risk of injury and death to elevator passengers and damage to building structures by having an elevator running while its CWT derailment switch is activated. A17.1 Earthquake Safety Committee is currently working on a post-earthquake procedure/operation guideline to expedite returning elevator service to key buildings.

Reference ASME A17.1 – 2004, 8.4.10.

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

Public Hearing Results

Committee Action: **Approved as Modified**

Replace proposal with the following:

1613.6.3 Seismic controls for elevators. Seismic switches in accordance with Section 8.4.10 of ASME A17.1 shall be deemed to comply with Section 13.6.10.3 of ASCE 7.

Committee Reason: This proposal clarifies the application of multiple provisions in referenced standards for seismic controls on elevators. The modification adds an IBC provision, recognizing seismic switches in accordance with ASME A17.1 as deemed to comply with the ASCE 7 requirements for seismic controls.

Assembly Action: **None**

Final Hearing Results

S26-06/07 **AM**

Code Change No: S28-06/07

Original Proposal

Section: 1702.1

Proponent: Sam Francis, American Forest & Paper Association

Revise as follows:

SECTION 1702
DEFINITIONS

FABRICATED ITEM. Structural, load-bearing or lateral load-resisting assemblies consisting of materials assembled prior to installation in a building or structure, or subjected to operations such as heat treatment, thermal cutting, cold working or reforming after manufacture and prior to installation in a building or structure. Materials produced in accordance with ~~standard specifications referenced by this code, such as rolled structural steel shapes, steel-reinforcing bars, masonry units, and wood structural panels~~ a standard, listed in Chapter 35, that requires quality control to be provided under the supervision of a third party quality control agency shall not be considered "fabricated items."

Reason: This proposal is intended to clarify the code requirements for special inspections. Many common items are fabricated under standards cited in the IBC. Many of those items are fabricated with strict quality assurance done under third party supervision. In addition, the proposal also eliminates laundry lists from the code text. Such lists make interpretation and maintenance of the code awkward at best but potentially very, very difficult.

The reason for this change is to eliminate what amounts to a duplicate requirement in the code. Special Inspections are exactly analogous to the QC program required by some standards listed in the code. This change makes it clear that the code is not intended to require a redundant set of inspection requirements for those items. They have been produced satisfactorily for many years under the code mandated third party QC and inspection.

Cost Impact: The code change proposal will not increase the cost of construction. Elimination of unnecessary and redundant requirements lower costs of various amounts.

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

FABRICATED ITEM. Structural, load-bearing or lateral load-resisting assemblies consisting of materials assembled prior to installation in a building or structure, or subjected to operations such as heat treatment, thermal cutting, cold working or reforming after manufacture and prior to installation in a building or structure. Materials produced in accordance with standard specifications referenced by this code, such as rolled structural steel shapes, steel-reinforcing bars, masonry units, and wood structural panels or in accordance with a standard, listed in Chapter 35, ~~that requires~~ which provides requirements for quality control ~~to be provided~~ done under the supervision of a third party quality control agency shall not be considered "fabricated items."

Committee Reason: The proposal clarifies the definition of fabricated item so that the special inspection requirements can be applied more uniformly. The modification retains the current list of specific items that are excluded from the definition of fabricated items.

Assembly Action: **None**

Final Hearing Results

S28-06/07 **AM**

Code Change No: S29-06/07

Original Proposal

Sections: 1702, 202

Proponent: William W. Stewart, FAIA, Chesterfield, MO, representing himself

1. Add new definitions as follows:

SECTION 202
DEFINITIONS

LABEL. ~~See Section 1702.1.~~ An identification applied on a product by the manufacturer that contains the name of the manufacturer, the function and performance characteristics of the product or material, and the name and identification of an approved agency and that indicates that the representative sample of the product or material has been tested and evaluated by an approved agency (see Section 1703.5 and “Inspection certificate,” “Manufacturer’s designation” and “Mark”).

MANUFACTURER’S DESIGNATION. ~~See Section 1702.1.~~ An identification applied on a product by the manufacturer indicating that a product or material complies with a specified standard or set of rules (see also “Inspection certificate,” “Label” and “Mark”).

MARK. ~~See Section 1702.1.~~ An identification applied on a product by the manufacturer indicating the name of the manufacturer and the function of a product or material (see also “Inspection certificate,” “Label” and “Manufacturer’s designation”).

2. Delete definitions without substitution:

SECTION 1702
DEFINITIONS

~~**LABEL.** An identification applied on a product by the manufacturer that contains the name of the manufacturer, the function and performance characteristics of the product or material, and the name and identification of an approved agency and that indicates that the representative sample of the product or material has been tested and evaluated by an approved agency (see Section 1703.5 and “Inspection certificate,” “Manufacturer’s designation” and “Mark”).~~

~~**MANUFACTURER’S DESIGNATION.** An identification applied on a product by the manufacturer indicating that a product or material complies with a specified standard or set of rules (see also “Inspection certificate,” “Label” and “Mark”).~~

~~**MARK.** An identification applied on a product by the manufacturer indicating the name of the manufacturer and the function of a product or material (see also “Inspection certificate,” “Label” and “Manufacturer’s designation”).~~

Reason: Since label is a term used in many sections (715.3.5, 1404.10, 2303.21 and 2405.5 for example) the definition should be in Chapter 2 not Chapter 17. I have not changed any text, just moved it to 202. The same is true for Manufacturer’s Designation and Mark. They are rarely used in the code but they appear more often in other chapters than in Chapter 17. If moved then they should then be deleted from Chapter 17.

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This code change clarifies applicability for the definitions of “mark” and “label” by more appropriately locating them in Chapter 2.

Assembly Action: **None**

Final Hearing Results

S29-06/07 **AS**

Code Change No: S30-06/07

Original Proposal

Sections: 1703.6, 2403.1.1 (New) [IEBC 302.1.1 (New)]

Proponent: William W. Stewart, FAIA, Chesterfield, MO, representing himself

1. Delete without substitution:

~~**1703.6 Heretofore approved materials.** The use of any material already fabricated or of any construction already erected, which conformed to requirements or approvals heretofore in effect, shall be permitted to continue, if not detrimental to life, health or safety to the public.~~

(Renumber subsequent sections)

2. Add new text as follows:

<u>**3403.1.1 (IEBC 302.1.1) Heretofore approved materials.** The use of any material already fabricated or of any construction already erected, which conformed to requirements or approvals heretofore in effect, shall be permitted to continue, if not detrimental to life, health or safety to the public.</u>

Reason: This section covers all existing materials and belongs in Chapter 34 Existing Structures. Section 1703.6 has been moved, with no changes to Chapter 34.

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

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: Rather than relocate to Chapter 34, it is felt that the provision for "heretofore approved materials" is appropriate in its current location, since it would apply to work under construction.

Assembly Action: **None**

Public Comments

Individual Consideration Agenda

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

Public Comment:

William W. Stewart, FAIA, Chesterfield, MO, representing himself, requests Approval as Submitted.

Commenter=s Reason: The committee disapproved this change because they felt it would affect materials in buildings under construction. The commentary makes it clear that this section refers to code complying materials in buildings that have been completed. The commentary says, "If a material or system had been approved before the code took effect, it can continue to be used as long as it can be shown that the material or system is not detrimental to the health or safety of the building occupants or the public. In other words the code is not retroactive."

Therefore this section is more appropriately located in Chapter 34 since Chapter 34 addresses existing buildings. Additionally this stipulation is lost in a section entitled Structural Design since it covers all the materials in a building, not just structural materials.

Final Hearing Results

S30-06/07 **AS**

Code Change No: S31-06/07

Original Proposal

Section: 1704.1

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

Revise as follows:

1704.1 General. Where application is made for construction as described in this section, the owner or the registered design professional in responsible charge acting as the owner's agent shall employ one or more special inspectors to provide inspections during construction on the types of work listed under Section 1704. The special inspector shall be a qualified person who shall demonstrate competence, to the satisfaction of the building official, for inspection of the particular type of construction or operation requiring special inspection. These inspections are in addition to the inspections specified in Section 109.

Exceptions:

1. Special inspections are not required for work of a minor nature or as warranted by conditions in the jurisdiction as approved by the building official.
2. Special inspections are not required for building components unless the design involves the practice of professional engineering or architecture as defined by applicable state statutes and regulations governing the professional registration and certification of engineers or architects.
3. Unless otherwise required by the building official, special inspections are not required for occupancies in ~~Group R-3 as applicable in Section 101.2 and occupancies in~~ Group U that are accessory to a residential occupancy including, but not limited to, those listed in Section 312.1.

Reason: In the 2003 IBC, there were approximately 45 provisions applicable to "Group R-3 as applicable in Section 101.2." Section 101.2 requires application of the provisions of the IBC to every building or structure or any appurtenances connected to or attached to such buildings or structures. Detached one- and two-family dwellings and multiple single-family dwellings not more than three stories above grade with a separate means of egress and their accessory structures, however, are required to comply with the IRC. Existing buildings undergoing repair, alteration or additions and changes of occupancy are permitted to comply with the IEBC.

In the 2006 IBC, the phrase, "as applicable in Section 101.2," has been deleted in virtually all cases except for Exception 3 to Section 1704.1. Currently, Exception 3 exempts Group R-3 occupancies complying with the IRC from the requirements for special inspection in the IBC. If deletion of the phrase, "as applicable in Section 101.2," in Section 1704.1 were to occur, Group R-3 occupancies complying with the IBC would be exempt from the requirements for special inspection in the IBC.

The proposal deletes the exemption for Group R-3 occupancies. The structural systems of Group R-3 buildings can just as complex and challenging as those of commercial structures. The use of high-strength concrete, structural steel, high-strength bolting, complete-penetration grove welds, engineered masonry, pile foundations and other materials, components and systems that typically receive special inspection in commercial structures are often seen in Group R-3 buildings. In Seismic Design Categories C, D, E and F, engineered seismic-force-resisting systems are also common.

The requirement for special inspection of Group R-3 occupancies in the IBC is warranted and should be retained. Exception 1 to Section 1704.1 will continue to provide the building official with the discretion to exempt work of a minor nature or as warranted by conditions in the jurisdiction from the requirement for special inspection.

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: Removing the special inspection exemption for Group R-3 is an improvement that is also consistent with action taken in previous code development cycle.

Assembly Action: **None**

Final Hearing Results

S31-06/07 **AS**

Code Change No: S32-06/07

Original Proposal

Section: 1704.1

Proponent: Brian Scot Tollisen, P.E., New York Department of State Division of Code Enforcement and Administration

Revise as follows:

1704.1 General. Where application is made for construction as described in this section, the owner or the registered design professional in responsible charge acting as the owner's agent shall employ one or more special inspectors to provide inspections during construction on the types of work listed under Section 1704. ~~The special inspector shall be a qualified person who shall demonstrate competence, to the satisfaction of the building official, for the inspection of the particular type of construction or operation requiring special inspection.~~ These inspections are in addition to the inspections identified in Section 109.

The special inspector shall be a qualified person who shall demonstrate competence, to the satisfaction of the building official, for the inspection of the particular type of construction or operation requiring special inspection. The special inspector shall provide written documentation to the building official demonstrating their competence and relevant experience. Experience shall be considered relevant when the documented experience is related in complexity to the same type of special inspection activities for projects of similar complexity and material qualities. These qualifications are in addition to qualifications specified in other sections of this code.

Exceptions:

1. Special inspections are not required for work of a minor nature or as warranted by conditions in the jurisdiction as approved by the building official.
2. Special inspections are not required for building components unless the design involves the practice of professional engineering or architecture as defined by applicable state statutes and regulations governing the professional registration and certification of engineers or architects.
3. Unless otherwise required by the building official, special inspections are not required for occupancies in Group R-3 as applicable in Section 101.2 and occupancies in Group U that are accessory to a residential occupancy including, but not limited to, those listed in Section 312.1.

Reason: The purpose of this proposal is to provide an adequate pool of qualified and knowledgeable special inspectors. The additions to IBC Section 1704.1 are proposed with the intention to standardize special inspection qualifications within the model code.

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

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

1704.1 General. Where application is made for construction as described in this section, the owner or the registered design professional in responsible charge acting as the owner's agent shall employ one or more special inspectors to provide inspections during construction on the types of work listed under Section 1704. These inspections are in addition to the inspections identified in Section 109. The special inspector shall be a qualified person who shall demonstrate competence, to the satisfaction of the building official, for the inspection of the particular type of construction or operation requiring special inspection. The special inspector shall provide written documentation to the building official demonstrating their competence and relevant experience or training. Experience or training shall be considered relevant when the documented experience or training is related in complexity to the same type of special inspection activities for projects of similar complexity and material qualities. These qualifications are in addition to qualifications specified in other sections of this code.

Exceptions:

1. Special inspections are not required for work of a minor nature or as warranted by conditions in the jurisdiction as approved by the building official.

2. Special inspections are not required for building components unless the design involves the practice of professional engineering or architecture as defined by applicable state statutes and regulations governing the professional registration and certification of engineers or architects.
3. Unless otherwise required by the building official, special inspections are not required for occupancies in Group R-3 as applicable in Section 101.2 and occupancies in Group U that are accessory to a residential occupancy including, but not limited to, those listed in Section 312.1.

Committee Reason: This proposal will promote standardized special inspector qualifications. It is felt that poorly trained special inspectors have impacted construction in some areas. The modification adds training as a qualification that would be equivalent to relevant experience.

Assembly Action: **None**

Final Hearing Results

S32-06/07 **AM**

Code Change No: S35-06/07

Original Proposal

Sections: 1704.4, 1705.3, 1709.2, 1805.5.7, 1805.9, 1808.2.5, 1808.2.23.2, 1904.2.2, 1909.4, 1915.5, 2308.3.3, 2308.11.1, 2308.12.1

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

Revise as follows:

1704.4 Concrete construction. The special inspections and verifications for concrete construction shall be as required by this section and Table 1704.4.

Exception: Special inspections shall not be required for:

1. Isolated spread concrete footings of buildings three stories or less ~~in height~~ <u>above grade plane</u> that are fully supported on earth or rock.
2. Continuous concrete footings supporting walls of buildings three stories or less ~~in height~~ <u>above grade plane</u> that are fully supported on earth or rock where:
   2.1. The footings support walls of light frame construction;
   2.2. The footings are designed in accordance with Table 1805.4.2; or
   2.3. The structural design of the footing is based on a specified compressive strength, f'_c, no greater than 2,500 pounds per square inch (psi) (17.2 Mpa), regardless of the compressive strength specified in the construction documents or used in the footing construction.
3. Nonstructural concrete slabs supported directly on the ground, including prestressed slabs on grade, where the effective prestress in the concrete is less than 150 psi (1.03 Mpa).
4. Concrete foundation walls constructed in accordance with Table 1805.5(5).
5. Concrete patios, driveways and sidewalks, on grade.

1705.3 Seismic resistance. The statement of special inspections shall include seismic requirements for the following cases:

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C, D, E or F, in accordance with Section 1613.
2. Designated seismic systems in structures assigned to Seismic Design Category D, E or F.
3. The following additional systems and components in structures assigned to Seismic Design Category C:
   3.1. Heating, ventilating and air-conditioning (HVAC) ductwork containing hazardous materials and anchorage of such ductwork.
   3.2. Piping systems and mechanical units containing flammable, combustible or highly toxic materials.

3.3. Anchorage of electrical equipment used for emergency or standby power systems.
4. The following additional systems and components in structures assigned to Seismic Design Category D:
   4.1. Systems required for Seismic Design Category C.
   4.2. Exterior wall panels and their anchorage.
   4.3. Suspended ceiling systems and their anchorage.
   4.4. Access floors and their anchorage.
   4.5. Steel storage racks and their anchorage, where the importance factor is equal to 1.5 in accordance with Section 15.5.3 of ASCE 7.
5. The following additional systems and components in structures assigned to Seismic Design Category E or F:
   5.1. Systems required for Seismic Design Categories C and D.
   5.2. Electrical equipment.

Exception: Seismic requirements are permitted to be excluded from the statement of special inspections for structures designed and constructed in accordance with the following:

1. The structure consists of light-frame construction; the design spectral response acceleration at short periods, S_{DS}, as determined in Section 1613.5.4, does not exceed 0.5g; and the height of the structure does not exceed 35 feet (10 668 mm) above grade plane; or
2. The structure is constructed using a reinforced masonry structural system or reinforced concrete structural system; the design spectral response acceleration at short periods, S_{DS}, as determined in Section 1613.5.4, does not exceed 0.5g, and the height of the structure does not exceed 25 feet (7620 mm) above grade plane; or
3. Detached one- or two-family dwellings not exceeding two stories ~~in height~~ above grade plane, provided the structure does not have any of the following plan or vertical irregularities in accordance with Section 12.3.2 of ASCE 7:
   3.1. Torsional irregularity.
   3.2. Nonparallel systems.
   3.3. Stiffness irregularity–extreme soft story and soft story.
   3.4. Discontinuity in capacity–weak story.

1709.2 Structural observations for seismic resistance. Structural observations shall be provided for those structures included in Seismic Design Category D, E or F, as determined in Section 1613.5.6, where one or more of the following conditions exist:

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

1805.5.7 Pier and curtain wall foundations. Except in Seismic Design Categories D, E and F, pier and curtain wall foundations are permitted to be used to support light-frame construction not more than two stories ~~in height~~ above grade plane, provided the following requirements are met:

1. All load-bearing walls shall be placed on continuous concrete footings bonded integrally with the exterior wall footings.
2. The minimum actual thickness of a load-bearing masonry wall shall not be less than 4 inches (102 mm) nominal or 3-5/8 inches (92 mm) actual thickness, and shall be bonded integrally with piers spaced 6 feet (1829 mm) on center (o.c.).
3. Piers shall be constructed in accordance with Chapter 21 and the following:
   3.1. The unsupported height of the masonry piers shall not exceed 10 times their least dimension.
   3.2. Where structural clay tile or hollow concrete masonry units are used for piers supporting beams and girders, the cellular spaces shall be filled solidly with concrete or Type M or S mortar.

   Exception: Unfilled hollow piers are permitted where the unsupported height of the pier is not more than four times its least dimension.

   3.3. Hollow piers shall be capped with 4 inches (102 mm) of solid masonry or concrete or the cavities of the top course shall be filled with concrete or grout.

4. The maximum height of a 4-inch (102mm)load-bearing masonry foundation wall supporting wood frame walls and floors shall not be more than 4 feet (1219 mm) in height.
5. The unbalanced fill for 4-inch (102 mm) foundation\ walls shall not exceed 24 inches (610 mm) for solid masonry, nor 12 inches (305mm)for hollow masonry.

1805.9 Seismic requirements. See Section 1908 for additional requirements for footings and foundations of structures assigned to Seismic Design Category C, D, E or F.

For structures assigned to Seismic Design Category D, E or F, provisions of ACI 318, Sections 21.10.1 to 21.10.3, shall apply when not in conflict with the provisions of Section 1805. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.

Exceptions:

1. Group R or U occupancies of light-framed construction and two stories or less ~~in height~~ above grade plane are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2 MPa) at 28 days.
2. Detached one- and two-family dwellings of light-frame construction and two stories or less ~~in height~~ above grade plane are not required to comply with the provisions of ACI 318, Sections 21.10.1 to 21.10.3.

1808.2.5 Stability. Piers or piles shall be braced to provide lateral stability in all directions. Three or more piles connected by a rigid cap shall be considered braced, provided that the piles are located in radial directions from the centroid of the group not less than 60 degrees (1 rad) apart. A two-pile group in a rigid cap shall be considered to be braced along the axis connecting the two piles. Methods used to brace piers or piles shall be subject to the approval of the building official.

Piles supporting walls shall be driven alternately in lines spaced at least 1 foot (305 mm) apart and located symmetrically under the center of gravity of the wall load carried, unless effective measures are taken to provide for eccentricity and lateral forces, or the wall piles are adequately braced to provide for lateral stability. A single row of piles without lateral bracing is permitted for one- and two-family dwellings and lightweight construction not exceeding two stories above grade plane or 35 feet (10 668 mm) in height, provided the centers of the piles are located within the width of the foundation wall.

1808.2.23.2 Seismic Design Category D, E or F. Where a structure is assigned to Seismic Design Category D, E or F in accordance with Section 1613, the requirements for Seismic Design Category C given in Section 1808.2.23.1 shall be met, in addition to the following. Provisions of ACI 318, Section 21.10.4, shall apply when not in conflict with the provisions of Sections 1808 through 1812. Concrete shall have a specified compressive strength of not less than 3,000 psi (20.68 MPa) at 28 days.

Exceptions:

1. Group R or U occupancies of light-framed construction and two stories or less ~~in height~~ above grade plane are permitted to use concrete with a specified compressive strength of not less than 2,500 psi (17.2 MPa) at 28 days.
2. Detached one- and two-family dwellings of light-frame construction and two stories or less in height are not required to comply with the provisions of ACI 318, Section 21.10.4.
3. Section 21.10.4.4(a) of ACI 318 need not apply to concrete piles.

1904.2.2 Concrete properties. Concrete that will be subject to the following exposures shall conform to the corresponding maximum water-cementitious materials ratios and minimum specified concrete compressive strength requirements of ACI 318, Section 4.2.2.

1. Concrete intended to have low permeability where exposed to water;
2. Concrete exposed to freezing and thawing in a moist condition or deicer chemicals; or
3. Corrosion protection of reinforcement in concrete exposed to chlorides from deicing chemicals, salt, salt water, brackish water, seawater, or spray from these sources.

Exception: For occupancies and appurtenances thereto in Group R occupancies that are in buildings less than four stories ~~in height~~ above grade plane, normal-weight aggregate concrete shall comply with the requirements of Table 1904.2.2(2) based on the weathering classification (freezing and thawing) determined from Figure 1904.2.2.

In addition, concrete that will be exposed to deicing chemicals shall conform to the limitation of Section 1904.2.3.

1909.4 Design. Structural plain concrete walls, footings and pedestals shall be designed for adequate strength in accordance with ACI 318, Sections 22.4 through 22.8.

Exception: For Group R-3 occupancies and buildings of other occupancies less than two stories ~~in height~~ above grade plane of light-frame construction, the required edge thickness of ACI 318 is permitted to be reduced to 6 inches (152 mm), provided that the footing does not extend more than 4 inches (102 mm) on either side of the supported wall.

1915.5 Fire-resistance-rating protection. Pipe columns shall be of such size or so protected as to develop the required fire-resistance ratings specified in Table 601. Where an outer steel shell is used to enclose the fire protective covering, the shell shall not be included in the calculations for strength of the column section. The minimum diameter of pipe columns shall be 4 inches (102 mm) except that in structures of Type V construction not exceeding three stories above grade plane or 40 feet (12 192 mm) in height, pipe columns used in the basement and as secondary steel members shall have a minimum diameter of 3 inches (76 mm).

2308.3.3 Sill anchorage. Where foundations are required by Section 2308.3.4, braced wall line sills shall be anchored to concrete or masonry foundations. Such anchorage shall conform to the requirements of Section 2308.6 except that such anchors shall be spaced at not more than 4 feet (1219 mm) o.c. for structures over two stories ~~in height~~ above grade plane. The anchors shall be distributed along the length of the braced wall line. Other anchorage devices having equivalent capacity are permitted.

2308.11.1 Number of stories. Structures of conventional light-frame construction shall not exceed two stories ~~in height~~ above grade plane in Seismic Design Category C.

2308.12.1 Number of stories. Structures of conventional light-frame construction shall not exceed one story ~~in height~~ above grade plane in Seismic Design Category D or E.

Reason: All of the code sections in this proposal have one thing in common. They specify requirements for a building based on its number of stories. A story is defined in Sections 202 and 502.1 as "that portion of a building included between the upper surface of a floor and the upper surface of the floor or roof next above," which includes portions of a building below grade plane (i.e., basements). Consequently, the number of stories specified in of these code sections would be determined beginning at the bottommost level in the building, which could be several levels (stories) below grade plane. The proposal will establish that the determination begins at the first story above grade plane, which is the probable intent in each case.

This proposal does not include each code section in the IBC that specifies requirements for a building based on its number of stories. There are cases where the determination of the number of stories in a building beginning at the bottommost level is warranted. Please refer to Sections 406.1.1, 415.4, 415.5, 415.7.3.3, 903.2.8.1(1), 1015.2, 2305.1.5, 2305.2.5, 2603.5, 3310.1 and 3311.1.

There are also several code sections in the IBC that currently specify requirements for a building based on its number of stories above grade plane. Please refer to Sections 101.2 (Exception 1), 402.1, 402.7.3, 415.7.3.5, 415.9.2.3, 903.2.3(2), 903.2.6(2), 903.2.8(2), 1009.12, 1002.6 (Exception 1), 1018.2 (Item 1), 1025.1, 1407.11.1, 1407.11.2, 1705.1 (Exceptions 1 and 2), 1509.5, 1807.1.1, 2308.2(1), 2308.2.2 (Exception 2), 2308.11.2 (Exception 1), 2308.12.2 (Exception 1), 2607.3, 2608.2 (Item 1) and 3002.4. See code change proposal G44-04/05 (AM) for further information.

This proposal is partly a continuation of code change proposal G44-04/05 (AM), which successfully distinguished between requirements based on the height or number of stories of a building by measuring from grade plane versus the height of a component of a building by measuring from grade. There is also a similar proposal before the IBC General Committee.

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This code change will clarify the determination of a building's height in stories in the structural chapters by referring to grade plane.

Assembly Action: **None**

Final Hearing Results

S35-06/07 **AS**

Code Change No: S37-06/07

Original Proposal

Sections: 1704.7, 1704.8, 1704.9, 1708.4, 1708.5, 1709.2, 1709.3

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

Revise as follows:

1704.7 Soils. Special inspections for existing site soil conditions, fill placement and load-bearing requirements shall be as required by this section and Table 1704.7. The approved soils report, required by Section 1802.2, and the documents prepared by the registered design professional ~~in responsible charge~~ shall be used to determine compliance. During fill placement, the special inspector shall determine that proper materials and procedures are used in accordance with the provisions of the approved soils report, as specified in Section 1803.5.

Exception: Special inspection is not required during placement of controlled fill having a total depth of 12 inches (305 mm) or less.

1704.8 Pile foundations. Special inspections shall be performed during installation and testing of pile foundations as required by Table 1704.8. The approved soils report, required by Section 1802.2, and the documents prepared by the registered design professional ~~in responsible charge~~ shall be used to determine compliance.

1704.9 Pier foundations. Special inspections shall be performed during installation and testing of pier foundations as required by Table 1704.9. The approved soils report, required by Section 1802.2, and the documents prepared by the registered design professional ~~in responsible charge~~ shall be used to determine compliance.

1708.4 Structural steel. The testing contained in the quality assurance plan shall be as required by AISC 341 and the additional requirements herein. The acceptance criteria for nondestructive testing shall be as required in AWS D1.1 as specified by the registered design professional.

Base metal thicker than 1.5 inches (38 mm), where subject to through-thickness weld shrinkage strains, shall be ultrasonically tested for discontinuities behind and adjacent to such welds after joint completion. Any material discontinuities shall be accepted or rejected on the basis of ASTM A 435 or ASTM A 898 (Level 1 criteria) and criteria as established by the registered design professional~~(s)~~ in responsible charge and the construction documents.

1708.5 Seismic qualification of mechanical and electrical equipment. The registered design professional ~~in responsible charge~~ shall state the applicable seismic qualification requirements for designated seismic systems on the construction documents. Each manufacturer of designated seismic system components shall test or analyze the component and its mounting system or anchorage and submit a certificate of compliance for review and acceptance by the registered design professional ~~in responsible charge of~~ <u>for</u> the design of the designated seismic system and for approval by the building official. Qualification shall be by actual test on a shake table, by three-dimensional shock tests, by an analytical method using dynamic characteristics and forces, by the use of experience data (i.e., historical data demonstrating acceptable seismic performance) or by more rigorous analysis providing for equivalent safety.

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

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

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

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

Reason: This proposal is in response to Recommendation 28 of the "Final Report on the Collapse of the World Trade Center Towers" (NIST NCSTAR 1). The recommendation assumes that the IBC already defines "design professional in responsible charge," which is not the case. A related proposal before the General Code Committee will provide a definition in the IBC as well as the IEBC. Note that the term "registered design professional in responsible charge" is used throughout the IBC (i.e., Sections 106.3.4.1, 106.3.4.2, 1704.1, 1704.1.1, 1704.1.2, 1704.7, 1704.8, 1704.9, 1705.1, 1708.4, 1708.5, 1709.2 and 1709.3) as well as the IEBC. Note also that the term is typically not used in other codes published by the International Code Council. Specially, there are no instances of the term "registered design professional in responsible charge" in the 2003 IECC, IFC, IFGC, IMC, IPC or IRC.

The proposed definition in the related proposal before the General Code Committee is consistent with the role the IBC and IEBC currently specify for the registered design professional in responsible charge, which is typically the review and coordination of submittal documents prepared by others, deferred submittal documents and phased submittal documents for compatibility with the design of the building or structure. Refer to Section 106.3.4 in the IBC and IEBC for specific language.

This proposal revises references to "registered design professional in responsible charge" in the structural chapters of the IBC to eliminate conflicts with Section 106.3.4. Sections 1704.7, 1704.8 and 1704.9 use "registered design professional in responsible charge" when referring to the registered design professional responsible for the structural design of a building or structure. Section 1708.5 uses the same term when referring to the registered design professional responsible for the structural design of a designated seismic system in at a building or structure. Section 106.3.4.1, however, states that the registered design professional in responsible charge is "responsible for reviewing and coordinating submittal documents prepared by others, including phased and deferred submittal items, for compatibility with the design of the building." Sections 1704.7, 1704.8, 1704.9 and 1708.5 are in conflict with Section 106.3.4.1 by using the same term for registered design professionals with responsibilities other than as specified in Section 106.3.4.1.

The IBC currently uses the term "registered design professional" in approximately 50 code sections, in addition to the code sections using the term "registered design professional in responsible charge." "Registered design professional" refers to an individual responsible for an aspect of the design of a building or structure. Typical examples are the architect, structural engineer, mechanical engineer, electrical engineer, civil engineer, fire protection engineer and others. Referring, instead, to such an individual as a registered design professional in responsible charge is an exercise in stating the obvious. Such individuals are in responsible charge of their individual responsibilities in the design of the building or structure. The term "in responsible change" is implicit in the term "registered design professional."

The proposal will eliminate the conflict between Section 106.3.4, which assigns responsibilities to a registered design professional acting as the registered design professional in responsible charge of a project, and other sections of the IBC, which use the same term but do not intend that the responsibilities in Section 106.3.4 also apply.

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

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

1704.7 Soils. Special inspections for existing site soil conditions, fill placement and load-bearing requirements shall be as required by this section and Table 1704.7. The approved soils report, required by Section 1802.2, and the documents prepared by the registered design professional shall be used to determine compliance. During fill placement, the special inspector shall determine that proper materials and procedures are used in accordance with the provisions of the approved soils report, as specified in Section 1803.5.

Exception: Special inspection is not required during placement of controlled fill having a total depth of 12 inches (305 mm) or less.

1704.8 Pile foundations. Special inspections shall be performed during installation and testing of pile foundations as required by Table 1704.8. The approved soils report, required by Section 1802.2, and the documents prepared by the registered design professional shall be used to determine compliance.

1704.9 Pier foundations. Special inspections shall be performed during installation and testing of pier foundations as required by Table 1704.9. The approved soils report, required by Section 1802.2, and the documents prepared by the registered design professional shall be used to determine compliance.

1708.4 Structural steel. The testing contained in the quality assurance plan shall be as required by AISC 341 and the additional requirements herein. The acceptance criteria for nondestructive testing shall be as required in AWS D1.1 as specified by the registered design professional. Base metal thicker than 1.5 inches (38 mm), where subject to through-thickness weld shrinkage strains, shall be ultrasonically tested for discontinuities behind and adjacent to such welds after joint completion. Any material discontinuities shall be accepted or rejected on the basis of ASTM A 435 or ASTM A 898 (Level 1 criteria) and criteria as established by the registered design professional in responsible charge and the construction documents.

1708.5 Seismic qualification of mechanical and electrical equipment. The registered design professional shall state the applicable seismic qualification requirements for designated seismic systems on the construction documents. Each manufacturer of designated seismic system components shall test or analyze the component and its mounting system or anchorage and submit a certificate of compliance for review and acceptance by the registered design professional for the design of the designated seismic system and for approval by the building official. Qualification shall be by actual test on a shake table, by three-dimensional shock tests, by an analytical method using dynamic characteristics and forces, by the use of experience data (i.e., historical data demonstrating acceptable seismic performance) or by more rigorous analysis providing for equivalent safety.

1709.2 Structural observations for seismic resistance. Structural observations shall be provided for those structures included in Seismic Design Category D, E or F, as determined in Section 1613, where one or more of the following conditions exist: The structure is classified as Occupancy Category III or IV in accordance with Section 1604.5.

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

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

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

Committee Reason: The proposal helps to distinguish between the various registered design professionals that are involved in a project by eliminating the phrase “in responsible charge” from Chapter 17. The modification clarifies that it is the registered design professional responsible for the structural design who may require structural observations in structures having higher wind or seismic risks.

Assembly Action: **None**

Final Hearing Results

S37-06/07 **AM**

Code Change No: S39-06/07

Original Proposal

Section: 1704.10

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

Revise as follows:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Bibliography:

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

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

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Assembly Action: None

Public Comments

Individual Consideration Agenda

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

Public Comment:

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

Modify proposal as follows:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

In Section 1704.10.4.3.3, "W-shape columns" are changed to "wide-flanged columns" for consistency with Section 714.8.3.2 in Proposal FS100-06/07. Note that W-shaped, M-shaped, S-shaped and HP-shaped" structural steel columns are manufactured. The current AISC Specification (AISC 360-05) typically refers to "I-shaped members" (i.e., Chapter F). The 2005 AISC Steel Construction Manual, however, typically refers to W-shapes, M-shapes, S-shapes and HP-shapes, which are described collectively as "H-shaped" and "I-shaped" (i.e., Scope).

Because of this level of detail, relying on "W-shape columns" in Section 1704.10.4.3.3 can lead to considerable confusion that can be avoided by use of the more generic "wide-flanged columns."

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

Final Hearing Results

S39-06/07 **AMPC1**

Code Change No: S42-06/07

Original Proposal

Section: 1707.7

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

Revise as follows:

1707.7 Architectural components. Periodic special inspection during the erection and fastening of exterior cladding, interior and exterior nonbearing walls and interior and exterior veneer in structures assigned to Seismic Design Category D, E or F.

Exceptions:

1. Special inspection is not required ~~for architectural components~~ in structures 30 feet (9144 mm) or less in height.
2. Special inspection is not required for cladding and veneer weighing 5 psf (24.5N/m^2) or less.
3. Special inspection is not required for ~~interior~~ nonbearing walls weighing 15 psf (73.5 N/m^2) or less.
4. <u>Special inspection is not required for exterior cladding and exterior veneer 30 feet (9144 mm) or less in height above grade.</u>

Reason: In Seismic Design Categories, D, E and F, Section 1707.7 specifies periodic special inspection during the erection and fastening of certain types of architectural components provided certain thresholds are reached. Currently, the charging statement specifies special inspection for exterior cladding, interior and exterior nonbearing walls and interior and exterior veneer. Exception 1, however, exempts architectural components, which are not specified in the charging statement. Presumably, referring to architectural components does not imply that Section 1707.7 applies to architectural components other than exterior cladding, interior and exterior nonbearing walls and interior and exterior veneer (i.e., interior cladding). Exception 2 exempts cladding and veneer weighing 5 psf or less from special inspection. Presumably, referring to cladding and veneer does not imply that Section 1707.7 applies to interior cladding. Exception 3 exempts interior nonbearing walls, but not exterior nonbearing walls, weighing 15 psf or less from special inspection. In summary, for all structures more than 30 feet in height, periodic special inspection is required for the erection and fastening of (1) all exterior nonbearing walls, (2) all exterior cladding and interior and exterior veneer weighing more than 5 psf, and (3) all interior nonbearing walls weighing more than 15 psf.

The current provisions create several unintended consequences. For example, at a structure more than 30 feet in height, special inspection is required, for example, at anchored brick masonry veneer supported by a concrete foundation and extending from finish grade to a few feet above grade (i.e., wainscot). For the same structure, special inspection is not required for any exterior cladding, interior veneer or exterior veneer weighing less than 5 psf, or for any interior nonbearing walls weighing less than 15 psf, but it s required for all of the exterior nonbearing walls. Special inspection is also required for all exterior cladding, interior veneer and exterior veneer weighing more than 5 psf, and for all interior nonbearing walls weighing more than 15 psf, no matter how close the component is to the ground surface (exterior cladding and veneer) or to the floor surface (interior veneer and nonbearing walls). The current requirements for periodic special inspection are summarized in the table below.

	Exterior Cladding	Nonbearing Walls		Veneer	
		Interior	Exterior	Interior	Exterior
Structure ≤ 30'0"	No	No	No	No	No
Structure > 30'0" and Exterior	Component	≤ 30'0":			
Component ≤ 5 psf	No	No	No	No	No
5 psf < Component ≤ 15 psf	Yes	No	Yes	Yes	Yes
Component > 15 psf	Yes	Yes	Yes	Yes	Yes
Structure > 30'0" and Exterior	Component	> 30'0":			
Component ≤ 5 psf	No	No	No	No	No
5 psf < Component ≤ 15 psf	Yes	No	Yes	Yes	Yes
Component > 15 psf	Yes	Yes	Yes	Yes	Yes

The proposed changes will establish thresholds for requiring special inspection that are more consistent with the relative risk posed by exterior cladding, interior and exterior nonbearing walls and interior and exterior veneer in Seismic Design Categories, D, E and F. As modified by the proposal, the requirements for periodic special inspection are summarized in the table below. Differences with the current requirements are highlighted in bold.

	Exterior Cladding	Nonbearing Walls		Veneer	
		Interior	Exterior	Interior	Exterior
Structure ≤ 30'0"	No	No	No	No	No
Structure > 30'0" and Exterior	Component	≤ 30'0":			
Component ≤ 5 psf	No	No	No	No	No
5 psf < Component ≤ 15 psf	**No**	No	**No**	Yes	**No**
Component > 15 psf	**No**	Yes	Yes	Yes	**No**
Structure > 30'0" and Exterior	Component	> 30'0":			
Component ≤ 5 psf	No	No	No	No	No
5 psf < Component ≤ 15 psf	Yes	No	**No**	Yes	Yes
Component > 15 psf	Yes	Yes	Yes	Yes	Yes

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

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: The proposed change was disapproved at the request of the proponent who intends to submit a modified proposal.

Assembly Action: **None**

Public Comments

Individual Consideration Agenda

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

Public Comment:

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

Replace proposal with the following:

1707.7 Architectural components. Periodic special inspection during the erection and fastening of exterior cladding, interior and exterior nonbearing walls and interior and exterior veneer in structures assigned to Seismic Design Category D, E or F.

Exceptions:

1. Special inspection is not required for ~~architectural components in structures~~ cladding, nonbearing walls and veneer 30 feet (9144 mm) or less in height above grade or walking surface.
2. Special inspection is not required for cladding and veneer weighing 5 psf (24.5N/m^2) or less.
3. Special inspection is not required for interior nonbearing walls weighing 15 psf (73.5 N/m^2) or less.

Commenter=s Reason: The public comment will align Section 1707.7 with Section 2.3.9 of the NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA 450) and achieve better consistency with Section 11A.1.3.9 of ASCE 7-05. As modified by this public comment, the requirements for periodic special inspection are summarized in the table below. Differences between the public comment and the current proposal are highlighted in **bold**.

	Exterior Cladding	Nonbearing Walls		Veneer	
		Interior	Exterior	Interior	Exterior
Structure ≤ 30’0”	No	No	No	No	No
Structure > 30’0” and Exterior	Component	≤ 30’0”:			
Component ≤ 5 psf	No	No	No	No	No
5 psf < Component ≤ 15 psf	No	No	No	**No**	No
Component > 15 psf	No	**No**	**No**	**No**	No
Structure > 30’0” and Exterior	Component	> 30’0”:			
Component ≤ 5 psf	No	No	**Yes**	No	No
5 psf < Component ≤ 15 psf	Yes	No	**Yes**	Yes	Yes
Component > 15 psf	Yes	Yes	Yes	Yes	Yes

Although the title of Section 1707.7 is “architectural components,” the charging language of Section 1707.7 does not specify architectural components. Rather, it specifies exterior cladding, interior and exterior nonbearing walls and interior and exterior veneer, which are examples of architectural components but are not the only types of architectural components. Architectural components other than exterior cladding, interior and exterior nonbearing walls and interior and exterior veneer are not subject to the requirements of Section 1707.7.

Final Hearing Results

S42-06/07 AMPC1

Code Change No: S44-06/07

Original Proposal

Section: 1708 (New)

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

Add new text as follows:

SECTION 1708
SPECIAL INSPECTIONS FOR WIND REQUIREMENTS

1708.1 Special inspections for wind requirements. Special inspections itemized in Sections 1708.2 and 1708.3, unless exempted by the exceptions to Section 1704.1, are required for buildings and structures constructed in the following areas:

1. In wind Exposure Category B, where the 3-second-gust basic wind speed is 120 miles per hour (52.8 m/sec) or greater.
2. In wind Exposure Categories C or D, where the 3-second-gust basic wind speed is 110 mph (49 m/sec) or greater.

1708.2 Structural wood. Continuous special inspection is required during field gluing operations of elements of the main wind-force-resisting system. Periodic special inspection is required for nailing, bolting, anchoring and other fastening of components within the main wind-force-resisting system, including wood shear walls, wood diaphragms, drag struts, braces and hold-downs.

Exception: Special inspection is not required for wood shear walls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other components of the main wind-force-resisting system, where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center.

1708.3 Cold-formed steel framing. Periodic special inspection is required during welding operations of elements of the main wind-force-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the main wind-force-resisting system, including struts, braces, and hold-downs.

(Renumber subsequent sections)

Reason: In areas of high seismic risk (i.e., Seismic Design Categories C, D, E and F), the IBC currently requires special inspection of seismic-force-resisting systems in buildings of light-frame construction (wood framing and cold-formed steel framing). The risk addressed by these requirements is equally present in areas of high wind forces and special inspection of main wind-force-resisting systems in buildings of light-frame construction is equally warranted. The purpose of this proposal is to establish these requirements in areas of high wind forces. This proposal is, in part, a response to comments made by the proponent during floor discussion of code change proposal S72-04/05 at the 2004/2005 code development hearings in Cincinnati.

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This code change is consistent with the current wind requirements in the statement of special inspection. It closes the loop and meets the intent of Section 1705.4.2.

Assembly Action: **None**

Final Hearing Results

S44-06/07 **AS**

Code Change No: S45-06/07

Original Proposal

Section: 1708 (New)

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

Add new text as follows:

SECTION 1708
SPECIAL INSPECTIONS FOR WIND REQUIREMENTS

1708.1 Special inspections for wind requirements. Special inspections itemized in Section 1708.2, unless exempted by the exceptions to Section 1704.1, are required for buildings and structures constructed in the following areas:

1. In wind Exposure Category B, where the 3-second-gust basic wind speed is 120 miles per hour (52.8 m/sec) or greater.
2. In wind Exposure Categories C or D, where the 3-second-gust basic wind speed is 110 mph (49 m/sec) or greater.

1708.2 Wind-resisting components. Periodic special inspection is required for the following systems and components:

1. Roof cladding.
2. Wall cladding.

Reason: In areas of high seismic risk (i.e., Seismic Design Categories C, D, E and F), the IBC currently requires special inspection of seismic-force-resisting systems in buildings of light-frame construction (wood framing and cold-formed steel framing). The risk addressed by these requirements is equally present in areas of high wind forces and special inspection of main wind-force-resisting systems in buildings of light-frame construction is equally warranted. A related proposal addresses main wind-force-resisting systems. This proposal addresses the cladding on buildings and structures in areas of high wind forces. Damage to buildings due to high wind forces often begins with failure of the cladding system, which often exposes the main wind-force-resisting system to damage from wind-driven rain and other forces that the wind-force-resisting system is typically not designed to withstand.

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This proposal completes the wind resistance special inspections and is consistent with the approval of S44-06/07.

Assembly Action: **None**

Final Hearing Results

S45-06/07	**AS**

Code Change No: S46-06/07

Original Proposal

Section: 1708.2

Proponent: William W. Stewart, FAIA, Chesterfield, MO, representing himself

Revise as follows:

1708.2 Testing for seismic resistance. The tests specified in sections 1708.3 through 1708.6 are required for the following:

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C, D, E or F, as determined in Section 1613.
2. ~~Designated seismic~~ Vibration isolated systems in structures assigned to Seismic Design Category C, D, E or F where the construction documents require a nominal clearance of 0.25 inches or less between the equipment support frame and restraint.
3. ~~Architectural, mechanical and electrical components in structures assigned to Seismic Design Category C, D, E or F that are required in Section 1708.5.~~

Reason: Based on the definition of designated seismic system in 1702, item 3 is redundant and can be deleted since it is a duplication of item 2.

There is a major fault with the pointer in the deleted item 3. Per item 3, 1708.5 is supposed to identify which designated seismic systems need Structural Testing. 1708.5 does not identify any components, therefore no designated seismic systems need Structural Testing. Based on Section 1708.5 I believe that the intent of item 3 was to cover mechanical equipment (mainly air handlers) that have spring mounted vibration isolators with snubbers. I have inserted text from item 5 of 1707.8 that describes those components.

Another way of looking at items 2 & 3; Item 2 currently says Structural Tests are necessary for all Designated Seismic Systems, which by definition are all architectural, mechanical and electrical components. Item 3, as explained above says no structural tests are necessary. Thus items 2 & 3 in conflict. If item 2 were correct, every partition, ceiling, light fixture, etc. that has an I_p greater than 1.0 would need Structural Testing. . This is obviously overkill. Item 3 is the intent of the code.

Seismic Design Category C was added to the retained exception since it was in deleted item 3.

Section 1708.1 and 1707.1 seem to have the same origin. You will see a similar change to 1707.1. The pointer in item 3 of 1707.1 did point to systems that needed special inspections. This change to Section 1708.1 will put the systems that need structural testing directly in 1708.1 and then refer to the qualification and testing as outlined in 1708.5.

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

Public Hearing Results

Committee Action: **Approved as Modified**

Modify the proposal as follows:

1708.2 Testing and qualification for seismic resistance. The ~~tests~~ testing and qualification specified in sections 1708.3 through 1708.6 are required ~~for the following~~ as follows:

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C, D, E or F, as determined in Section 1613 shall meet the requirements of Sections1708.3 and 1708.4, as applicable.

2. Designated seismic systems in structures assigned to Seismic Design Category D, E or F in Section 13.2.2 of ASCE 7 shall meet the requirements of Section 1708.5.
3. Architectural, mechanical and electrical components in structures assigned to Seismic Design Category C, D, E or F with an I_p = 1.0 shall be permitted to be seismically qualified by meeting the requirements of ~~that are required in~~ Section 1708.5.
4. The seismic isolation system in seismically isolated structures shall meet the testing requirements of Section 1708.6.

Committee Reason: This code change makes corrections to the organization of Section 1708.2 which lists items that require testing for seismic resistance. The modification provides consistency with the following sections that are referenced by Section 1708.2.

Assembly Action: **None**

Final Hearing Results

S46-06/07 **AM**

Code Change No: S47-06/07

Original Proposal

Sections: 1702.1, 1709.1

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

Revise as follows:

1702.1 General. The following words and terms shall, for the purposes of this chapter and as used elsewhere in this code, have the meanings shown herein.

STRUCTURAL OBSERVATION. The visual observation of the structural system by a registered design professional for general conformance to the approved construction documents ~~at significant construction stages and at completion of the structural system~~. Structural observation does not include or waive the responsibility for the inspection required by Section 109, 1704 or other sections of this code.

1709.1 General. Where required by the provisions of Section 1709.2 or 1709.3, the owner shall employ a registered design professional to perform structural observations as defined in Section 1702.

Prior to the commencement of observations, the structural observer shall submit to the building official a written statement identifying the frequency and extent of structural observations.

At the conclusion of the work included in the permit, the structural observer shall submit to the building official a written statement that the site visits have been made and identify any reported deficiencies which, to the best of the structural observer's knowledge, have not been resolved.

Reason: The purpose of this proposal is to delete an inappropriate phrase from the definition of structural observation. A definition should specify the meaning of the term being defined. If there are technical requirements associated with use of the term, they should be located elsewhere. The current definition of structural observation includes when visual observation by the structural observer shall occur: at significant construction stages and at completion of the structural system. These are technical requirements related to the frequency of observations, which should be placed elsewhere in the IBC. But if such requirements were located elsewhere, they should be substantially altered from their present form. A requirement for visual observation to occur at "significant construction stages" is vague and unenforceable, which is not appropriate for a regulatory document such as the IBC. The determination of the frequency of structural observations by the structural observer is best left to the structural observer and the owner in consultation with the local building official.

In place of the deleted language from the definition for structural observation, the proposal adds a requirement in Section 1709.1 for submittal of a written statement by the structural observer to the building official prior to the commencement of observations identifying the frequency and extent of structural observations. Requiring the submittal prior to commencement of construction authorized by the building permit is avoided because the structural observer will not necessarily know when construction begins and should not be expected to meet a deadline established by others beyond his or her control.

Note that the owner is required to employ a registered design professional to perform structural observation when required by Section 1709.2 or 1709.3. The structural observer, in turn, is required to submit a written statement to the building official at the conclusion of the work included in the permit (see Section 1709.1). Note also that one of the conditions for structural observation by a registered design professional is when so designated by the registered design professional in responsible charge of the design, which is typically the architect of record. See condition #4 in Sections 1709.2 and 1709.3.

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This code change will improve construction quality by requiring identification of the frequency and extent of structural observations.

Assembly Action: **None**

Final Hearing Results

S47-06/07 **AS**

Code Change No: S48-06/07

Original Proposal

Section: 1714.5.2, Chapter 35

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

1. Revise as follows:

1714.5.2 Exterior windows and door assemblies not provided for in Section 1714.5.1. Exterior window and door assemblies shall be tested in accordance with ASTM E 330. <u>Structural performance of garage doors shall be determined in accordance with either ASTM E 330 or ANSI/DASMA 108, and shall meet the acceptance criteria of ANSI/DASMA 108.</u> Exterior window and door assemblies containing glass shall comply with Section 2403. The design pressure for testing shall be calculated in accordance with Chapter 16. Each assembly shall be tested for 10 seconds at a load equal to 1.5 times the design pressure.

2. Add standard to Chapter 35 as follows:

DASMA

<u>ANSI/DASMA 108-02,</u> <u>Standard Method for Testing Sectional Garage Doors: Determination of Structural Performance Under Uniform Static Air Pressure Difference</u>

Reason: The purpose of this proposed code change is to reference an ANSI standard published specifically for the static air pressure testing of garage doors. ANSI/DASMA 108 includes garage door acceptance criteria, which is not contained within ASTM E 330. Similar language to what is being proposed is contained in 2006 IRC Section R613.5.

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 Results

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

Analysis: Review of proposed new standard indicated that, in the opinion of ICC staff, the standard did comply with ICC criteria for referenced standards.

Committee Action: **Approved as Submitted**

Committee Reason: The proposal appropriately adds an additional test standard that is specific to garage doors.

Assembly Action: **None**

Final Hearing Results

S48-06/07 **AS**

Code Change No: S51-06/07

Original Proposal

Section: 1808.2.23.1

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

Revise as follows:

1808.2.23.1 Seismic Design Category C. Where a structure is assigned to Seismic Design Category C in accordance with Section 1613, the following shall apply. Individual pile caps, piers or piles shall be interconnected by ties. Ties shall be capable of carrying, in tension and compression, a force equal to the product of the larger pile cap or column load times the seismic coefficient, S_{DS}, divided by 10 unless it can be demonstrated that equivalent restraint is provided by reinforced concrete beams within slabs on grade or reinforced concrete slabs on grade or confinement by competent rock, hard cohesive soils or very dense granular soils.

> **Exception:** In Group R-3 and U occupancies of light-frame construction, pier~~s~~ foundations supporting foundation walls, isolated interior posts detailed so the pier is not subject to lateral loads, or ~~lightly loaded~~ exterior decks and patios, ~~of Group R-3 and U occupancies not exceeding two stories of light-frame construction,~~ are not subject to interconnection if it can be shown the soils are of adequate stiffness, subject to the approval of the building official.

Reason: The current language was approved as submitted by code change proposal 1807.2.23.1 at the IBC First Draft Public Hearings in April, 1998. The staff analysis following the reason statement indicated that the “exception to Section 1807.2.23.1 is not understandable in its current format. Itemizing each individual condition is suggested.” I agree. The purpose of this proposal is to modify the exception to Section 1808.2.23.1 so that it is understandable to the average code user and to the proponent of this proposal. The change from “piers” to “pier foundations” is for consistency with the definition of “pier foundation” in Section 1808.1. The phrase “lightly loaded” is deleted because the language is vague and unenforceable.

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This code change makes the exception to interconnecting pier foundations in Group R-3 occupancies more understandable.

Assembly Action: **None**

Final Hearing Results

S51-06/07 **AS**

Code Change No: S52-06/07

Original Proposal

Section: 1808.2.23.2.1

Proponent: Michael Valley, Magnusson Klemencic Associates representing Structural Engineers Association of Washington Earthquake Engineering Committee

Revise as follows:

1808.2.23.2.1 Design details for piers, piles and grade beams. Piers or piles shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains modified for soil-pile-structure interaction coupled with pier or pile deformations induced by lateral pier or pile resistance to structure seismic forces. Concrete piers or piles on Site Class E or F sites, as determined in Section 1613.5.2, shall be designed and detailed in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within seven pile diameters of the pile cap and the interfaces of soft to medium stiff clay or liquefiable strata. For precast prestressed concrete piles, detailing provisions as given in Section 1809.2.3.2.1 and 1809.2.3.2.2 shall apply.

Grade beams shall ~~be designed as beams in accordance with~~ <u>comply with the provisions in Section 21.10.3 of</u> ACI 318 <u>for grade beams, except where they</u>~~, Chapter 21. When grade beams~~ have the capacity to resist the forces from the load combinations in Section 1605.4~~, they need not conform to ACI 318, Chapter 21~~.

Reason: Clarify the Code. This change clarifies the intent of the design requirement for grade beams in the last paragraph of the section. The provision currently requires grade beams to be designed as "beams" (not "grade beams") in accordance with Chapter 21 of ACI 318. Chapter 21 of ACI 318 has design provisions for grade beams (21.10.3), beams in special moment frames (21.3), coupling beams (21.7.7), and beams in intermediate moment frames (21.12.4). The revised text indicates which requirements apply.

This clarification makes the text similar to that in Section 14.2.7.2.2 of ASCE 7-05.

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: The proposal clarifies where grade beams need to comply with the referenced provision in ACI 318.

Assembly Action: **None**

Final Hearing Results

S52-06/07 **AS**

Code Change No: S53-06/07

Original Proposal

Section: 1808.2.23.2.1

Proponent: Michael Valley, Magnusson Klemencic Associates, representing Structural Engineers Association of Washington Earthquake Engineering Committee

Revise as follows:

1808.2.23.2.1 Design details for piers, piles and grade beams. Piers or piles <u>on Site Class E or F sites, as determined in Section 1613.5.2,</u> shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains modified for soil-pile-structure interaction coupled with pier or pile deformations induced by lateral pier or pile resistance to structure seismic forces. Where constructed of nonprestressed ~~C~~concrete, such piers or piles ~~on Site Class E or F sites, as determined in Section 1613.5.2,~~ shall be designed and detailed in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within seven pile diameters of the pile cap and within seven pile diameters of the interfaces of strata that are hard or stiff and ~~soft to medium stiff clay or~~strata that are liquefiable or are composed of soft to medium stiff clay~~strata~~.

Exception: Piers or piles that satisfy the following additional detailing requirements shall be deemed to comply with the curvature capacity requirements of this section.

1. ~~For~~ Precast prestressed concrete piles~~,~~ detailed ~~ing provisions as given~~ in accordance with Section ~~1809.2.3.2.1 and~~ 1809.2.3.2.2 ~~shall apply~~.
2. Cast-in-place concrete piles with a minimum longitudinal reinforcement ratio of 0.005 extending throughout the region detailed in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318, but not less than the length required in Section 1810.1.2.2.

Grade beams shall be designed as beams in accordance with ACI 318, Chapter 21. When grade beams have the capacity to resist the forces from the load combinations in Section 1605.4, they need not conform to ACI 318, Chapter 21.

Reason: Revise the scope of these additional pile analysis requirements. Clarify the portions of piles affected. Clarify the exception for precast prestressed piles. Add an exception for prescriptively detailed cast-in-place concrete piles.

Design for "pier or pile moments, shears and lateral deflections" is already required by Section 1808.2.23.1.2, and ductile detailing within three pile diameters of the pile cap is already required by Sections 1809.2.2.2.2 and 1810.1.2.2. The requirements of Section 1808.2.23.2.1, which add to those requirements, are taken from the NEHRP *Recommended Provisions* and are motivated by concern with pile response in soft or liquefiable soils (extended hinging region and kinematic interaction), as indicated in the NEHRP *Commentary* copied below. Such soils are assigned to Site Class E or F, as indicated in IBC Table 1615.1.1, so the corresponding additional requirements should be scoped accordingly.

At present this section applies to all buildings on piers or piles for all site classes, but that scope is inconsistent with both the rationale for the requirement and the state-of-the-practice. Requiring all geotechnical engineers to address the kinematic interaction issue for all projects will result in a large range of response ranging from nothing to potential recommendations for more expensive foundation types that don't significantly reduce societal risk.

As indicated in the NEHRP *Commentary* copied below, properly detailed piles provide the desired performance. For nonprestressed concrete piles, such proper detailing is defined in Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318. For precast prestressed piles, such proper detailing is defined in Section 1809.2.3.2.2 (which adds to the requirements in 1809.2.3.2.1). The section as modified maintains those detailing requirements and clarifies that this special detailing addresses the requirement related to "maximum imposed curvature."

The text defining the soil interfaces of concern is revised for clarity based on Section 14.2.7.2.1 of ASCE 7-05.

Commentary to the 2003 NEHRP *Recommended Provisions* Section 7.5.4 [**emphasis** added]:

> Special consideration is required in the design of concrete piles subject to significant bending during earthquake shaking. Bending can become crucial to pile design where portions of the foundation piles are supported in soils such as **loose granular materials and/or soft soils** that are susceptible to large deformations and/or strength degradation. Severe pile bending problems may result from various combinations of soil conditions during strong ground shaking, for example:
>
> 1. Soil settlement at the pile-cap interface either from consolidation of **soft soil** prior to the earthquake or from soil compaction during the earthquake can create a free-standing short column adjacent to the pile cap.
> 2. Large deformations and/or reduction in strength resulting from **liquefaction of loose granular materials** can cause bending and/or conditions of free-standing columns.
> 3. Large deformations in **soft soils** can cause varying degrees of pile bending. The degree of pile bending will depend upon thickness and strength of the soft soil layer(s) and/or the properties of the soft/stiff soil interface(s).

Such conditions can produce shears and/or curvatures in piles that may exceed the bending capacity of conventionally designed piles and result in severe damage. …

> The desired foundation performance can be accomplished by **proper selection and detailing** of the pile foundation system. Such design should accommodate bending from both reaction to the building's inertial loads and those induced by the motions of the soils themselves. Examples of designs of concrete piles include:
>
> 1. Use of a **heavy spiral reinforcement** and
> 2. Use of exterior steel liners to confine the concrete in the zones with large curvatures or shear stresses.

These provide proper confinement to ensure adequate ductility and maintenance of functionality of the confined core of the pile during and after the earthquake.

Precast prestressed concrete piles are exempted from the concrete special moment frame detailing requirements adapted for concrete piles since these provisions were never intended for slender precast prestressed concrete elements and will result in unbuildable piles. Piles with substantially less confinement reinforcement than required by ACI 318 equation 10-6 have been proven through cyclic testing to have adequate performance (Park and Hoat Joen, 1990).

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This proposal provides guidance to engineers on the additional pile analysis requirements for structures that are classified as Seismic Design Category D, E or F.

Assembly Action: **None**

Public Comments

Individual Consideration Agenda

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

Public Comment:

John Diebold, Structural Engineers Association of California, SEAOC Seismology Committee, requests Approval as Modified by this public comment.

Modify proposal as follows:

1808.2.23.2.1 Design details for piers, piles and grade beams. Piers or piles on Site Class E or F sites, as determined in Section 1613.5.2, shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains modified for soil-pile-structure interaction coupled with pier or pile deformations induced by lateral pier or pile resistance to structure seismic forces. ~~Where constructed of nonprestressed concrete such piers or piles shall be designed and detailed in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 within seven pile diameters of the pile cap and within seven pile diameters of the interfaces of strata that are hard or stiff and strata that are liquefiable or are composed of soft to medium stiff clay.~~

Exception: Piers or piles that satisfy the following additional detailing requirements shall be deemed to comply with the curvature capacity requirements of this section.

1. Precast prestressed concrete piles detailed in accordance with Section 1809.2.3.2.2.
2. Cast-in-place concrete piles with a minimum longitudinal reinforcement ratio of 0.005 extending the full length of the pile ~~extending throughout the region~~ and detailed in accordance with Sections 21.4.4.1, 21.4.4.2 and 21.4.4.3 of ACI 318 as required by this section.~~, but not less than the length required in Section 1810.1.2.2~~.

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

Commenter=s Reason: The Structural Engineers Association of California Seismology Committee agrees with the intent and spirit of the original proposal.

There are two points of concern that our Structural Engineers of California Seismology Committee believes are valid and need to be addressed as indicated in this modification. They are as follows:

1. The 0.005 longitudinal reinforcing ratio for the pile should be for the full length of the pile and not just for the "area detailed". The reason for this is that without the curvature analysis, the actual required flexural length of the pile has not been determined - so the point where this reinforcing could be reduced is unclear. Placing this reinforcement throughout the pile length compensates for what is unknown. The original proposal does not adequately address this. See Further Elaboration of Point #1 below for further discussion.
2. The closer tie spacing requirement for the 7 pile diameters above and below the interface and below the pile cap should be stated such that it clearly also applies to Exception 2. Otherwise, our concern is that the way the original S53 proposal reads could be misinterpreted to mean that the exception takes away the requirement for the stricter tie spacing within the 7 pile diameter distance of the interface and the bottom of the pile cap.

This can be clarified by moving the following phrase after the exception, or placing the exception prior to this phrase:

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

Further Elaboration of Point #1:
The exception is intended to provide equivalence to the curvature analysis and curvature capacity requirements. Therefore, the intent to require the .005 reinforcement the entire length of the pile is to address the imposed curvatures, which would have been calculated by the kinematic analysis but is being "waived" by this exception.

The original proposal points to extending the longitudinal reinforcing a length that "should be not less than the length required in Section 1810.1.2.2". The original proposal does not provide clarity regarding the length of the vertical reinforcing, nor does it provide any added detailing requirement to justify not performing the curvature analysis. 1810.1.2.2 requires that the "flexural length" be known - which could be determinable for Site Class D, but requires the more complicated kinematic analysis for multiple strata associated with Site Class E or F.

Because we are attempting to address a complex field condition where there are differing soil strata that will impose curvatures on the pile, it makes sense to provide this minimum amount (.005) of "flexural" longitudinal reinforcement to provide a level of structural integrity. It should run the full length of the pile, because the analysis to determine the extent of the curvature has not been done per the exception.

Final Hearing Results

S53-06/07 **AMPC1**

Code Change No: S54-06/07

Original Proposal

Section: 1810.8

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

Revise as follows:

1810.8 Micropiles. Micropiles shall ~~conform to~~ comply with the requirements of Sections 1810.8.1 through 1810.8.5.

1810.8.1 Construction. Micropiles shall consist of a grouted section reinforced with steel pipe or steel ~~reinforcing~~ reinforcement. Micropiles shall develop their load-carrying capacity through a bond zone in soil, bedrock or a combination of soil and bedrock. The ~~full length of the micropile shall contain either a~~ steel pipe or steel reinforcement shall extend the full length of the micropile.

1810.8.2 Materials. Grout shall have a ~~28-day~~ specified compressive strength (f'_c) of not less than 4,000 psi (27.58 Mpa). The grout mix shall be designed and proportioned so as to produce a pumpable mixture. Reinforcement ~~steel~~ shall ~~be~~ consist of deformed reinforcing bars in accordance with ASTM A 615 Grade 60 or Grade 75 or ASTM A 722 Grade 150.

~~Pipe/casing~~ The steel pipe shall have a minimum wall thickness of 3/16 inch (4.8 mm) ~~and as required to meet~~. Splices shall comply with Section 1808.2.7. ~~Pipe/casing~~ The steel pipe shall ~~meet the tensile requirements of~~ be in accordance with ASTM A 252 Grade 3, except the minimum yield strength shall be as used in the design submittal ~~[typically 50,000 psi to 80,000 psi (345 MPa to 552 MPa)]~~ and minimum elongation shall be 15 percent.

1810.8.3 Allowable stresses. The allowable ~~design~~ compressive stress ~~on~~ in the grout shall not exceed 0.33 f'_c. The allowable ~~design~~ compressive stress ~~on~~ in the steel pipe and steel reinforcement shall not exceed the lesser of 0.4 F_y, ~~or~~ and 32,000 psi (220 Mpa). The allowable ~~design~~ tensile stress ~~for~~ in the steel reinforcement shall not exceed 0.60 F_y. The allowable ~~design~~ tensile stress ~~for~~ in the cement grout shall be zero.

1810.8.4 Reinforcement. For piles or portions of piles grouted inside a temporary or permanent casing or inside a hole drilled into bedrock or a hole drilled with grout, the steel pipe or steel reinforcement shall be designed to carry at least 40 percent of the design compression load. Piles or portions of piles grouted in an open hole in soil without temporary or permanent casing and without suitable means of verifying the hole diameter during grouting shall be designed to carry the entire compression load in the reinforcing steel. Where a steel pipe is used for reinforcement, the portion of the ~~cement~~ grout enclosed within the pipe is permitted to be included ~~at~~ in the determination of the allowable stress ~~of~~ in the grout.

1810.8.4.1 Seismic reinforcement. Where a structure is assigned to Seismic Design Category C, a permanent steel casing shall be provided from the top of the pile down a minimum of 120 percent ~~times~~ of the flexural length. ~~The flexural length shall be determined in accordance with Section 1808.1.~~ Where a structure is assigned to Seismic Design D, E or F, the pile shall be considered as an alternative system~~. In~~ in accordance with Section 104.11~~, the~~ The alternative pile system design, supporting documentation and test data shall be submitted to the building official for review and approval.

1810.8.5 Installation. The pile shall be permitted to be formed in a hole advanced by rotary or percussive drilling methods, with or without casing. The pile shall be grouted with a fluid cement grout. The grout shall be pumped through a tremie pipe extending to the bottom of the pile until grout of suitable quality returns at the top of the pile. The following requirements apply to specific installation methods:

1. For piles grouted inside a temporary casing, the reinforcing ~~steel~~ bars shall be inserted prior to withdrawal of the casing. The casing shall be withdrawn in a controlled manner with the grout level maintained at the top of the pile to ensure that the grout completely fills the drill hole. During withdrawal of the casing, the grout level inside the casing shall be monitored to check that the flow of grout inside the casing is not obstructed.
2. For a pile or portion of a pile grouted in an open drill hole in soil without temporary casing, the minimum design diameter of the drill hole shall be verified by a suitable device during grouting.
3. For piles designed for end bearing, a suitable means shall be employed to verify that the bearing surface is properly cleaned prior to grouting.

4. Subsequent piles shall not be drilled near piles that have been grouted until the grout has had sufficient time to harden.
5. Piles shall be grouted as soon as possible after drilling is completed.
6. For piles designed with ~~casing~~ a full length casing, the casing ~~must~~ shall be pulled back to the top of the bond zone and reinserted or some by other suitable means employed to assure grout coverage outside the casing.

Reason: Substitute revised material for current provision of the Code.
The purpose of this proposal is to make editorial improvements to the language, which was approved by code change proposal S121-04/05(AM). In Section 1810.8.2, compliance with Section 1808.2.7 is specified for the splices of the steel pipe, which is the subject of Section 1808.2.7. The current language requires the steel pipe, not the splices of the steel pipe, to comply with Section 1808.2.7.

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

Public Hearing Results

Committee Action: **Approved as Modified**

Modify the proposal as follows:

1810.8 Micropiles. Micropiles shall comply with the requirements of Sections 1810.8.1 through 1810.8.5.

1810.8.1 Construction. Micropiles shall consist of a grouted section reinforced with steel pipe or steel reinforcement. Micropiles shall develop their load-carrying capacity through a bond zone in soil, bedrock or a combination of soil and bedrock. The steel pipe or steel reinforcement shall extend the full length of the micropile.

1810.8.2 Materials. Grout shall have a specified compressive strength (f'c) of not less than 4,000 psi (27.58 Mpa). The grout mix shall be designed and proportioned so as to produce a pumpable mixture. Reinforcement shall consist of deformed reinforcing bars in accordance with ASTM A 615 Grade 60 or Grade 75 or ASTM A 722 Grade 150.
The steel pipe shall have a minimum wall thickness of 3/16 inch (4.8 mm). Splices shall comply with Section 1808.2.7. The steel pipe shall ~~be in accordance with ASTM A 252 Grade 3, except the~~ have a minimum yield strength ~~shall be as used in the design submittal~~ exceeding 45,000 p.s.i. (310 MPa) and a minimum elongation ~~shall be~~ of 15 percent as shown by mill certifications or two coupon test samples per 40 000 pounds (kg) of pipe.

1810.8.3 Allowable stresses. The allowable compressive stress in the grout shall not exceed 0.33 f'c. The allowable compressive stress in the steel pipe and steel reinforcement shall not exceed the lesser of 0.4 Fy, and 32,000 psi (220 Mpa). The allowable tensile stress in the steel reinforcement shall not exceed 0.60 Fy. The allowable tensile stress in the cement grout shall be zero.

1810.8.4 Reinforcement. For piles or portions of piles grouted inside a temporary or permanent casing or inside a hole drilled into bedrock or a hole drilled with grout, the steel pipe or steel reinforcement shall be designed to carry at least 40 percent of the design compression load. Piles or portions of piles grouted in an open hole in soil without temporary or permanent casing and without suitable means of verifying the hole diameter during grouting shall be designed to carry the entire compression load in the reinforcing steel. Where a steel pipe is used for reinforcement, the portion of the grout enclosed within the pipe is permitted to be included in the determination of the allowable stress in the grout.

1810.8.4.1 Seismic reinforcement. Where a structure is assigned to Seismic Design Category C, a permanent steel casing shall be provided from the top of the pile down a minimum of 120 percent of the flexural length. Where a structure is assigned to Seismic Design D, E or F, the pile shall be considered as an alternative system in accordance with Section 104.11. The alternative pile system design, supporting documentation and test data shall be submitted to the building official for review and approval.

1810.8.5 Installation. The pile shall be permitted to be formed in a hole advanced by rotary or percussive drilling methods, with or without casing. The pile shall be grouted with a fluid cement grout. The grout shall be pumped through a tremie pipe extending to the bottom of the pile until grout of suitable quality returns at the top of the pile. The following requirements apply to specific installation methods:

1. For piles grouted inside a temporary casing, the reinforcing bars shall be inserted prior to withdrawal of the casing. The casing shall be withdrawn in a controlled manner with the grout level maintained at the top of the pile to ensure that the grout completely fills the drill hole. During withdrawal of the casing, the grout level inside the casing shall be monitored to check that theflow of grout inside the casing is not obstructed.
2. For a pile or portion of a pile grouted in an open drill hole in soil without temporary casing, the minimum design diameter of the drill hole shall be verified by a suitable device during grouting.
3. For piles designed for end bearing, a suitable means shall be employed to verify that the bearing surface is properly cleaned prior to grouting.
4. Subsequent piles shall not be drilled near piles that have been grouted until the grout has had sufficient time to harden.
5. Piles shall be grouted as soon as possible after drilling is completed.
6. For piles designed with a full length casing, the casing shall be pulled back to the top of the bond zone and reinserted or some by other suitable means employed to assure grout coverage outside the casing.

Committee Reason: The proposal makes editorial improvements so that the provisions for micropiles are clearer. The modification further clarifies the minimum material requirements for steel pipe.

Assembly Action: **None**

Final Hearing Results

S54-06/07 **AM**

Code Change No: S56-06/07

Original Proposal

Section: 1908.1.16

Proponent: John F. Silva, SE, Hilti, Inc.

Revise as follows:

1908.1.16 ACI 318, Section D.3.3. Modify ACI 318, Sections D.3.3.2 through D.3.3.5 to read as follows:

D.3.3.2 – *In structures assigned to Seismic Design Category C, D, E or F,* post-installed anchors for use under D.2.3 shall have passed the Simulated Seismic Tests of ACI 355.2.

D.3.3.3 – *In structures assigned to Seismic Design Category C, D, E or F,* the design strength of anchors shall be taken as 0.75 N_n and 0.75 V_n, where is given in D.4.4 or D.4.5, and N_n and V_n are determined in accordance with D.4.1.

D.3.3.4 – *In structures assigned to Seismic Design Category C, D, E or F,* anchors shall be designed to be governed by tensile or shear strength of a ductile steel element, unless D.3.3.5 is satisfied.

> ***Exception:** Anchors in concrete designed to support non-structural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.4*.

D.3.3.5 – Instead of D.3.3.4, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a load level corresponding to anchor forces not greater that the design strength of anchor specified in D.3.3.3, *or the minimum design strength of the anchor shall be at least 2.5 times the factored forces transmitted by the attachment*.

> ***Exception:** Anchors in concrete designed to support non-structural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.5.*

Reason: The purpose of the proposed code change is to correct an error that arises from the multiple provisions that address the design of non-ductile anchors.

This code change proposal corrects an inadvertent problem in coordination between the NEHRP Provisions and the ACI code in anchorage force requirements for the seismic design of nonstructural components. Currently, both ASCE 7 Section 13.4.2, which regulates the design of non-structural components for earthquake loading, and Section 1908.1.16, which addresses the design of anchors in concrete, impose additional load factors on anchors in SDC C and above. Increases for non-ductile anchorage forces are provided in ASCE 7-05 Section 13.4.2 and the changes to ACI 318-05 provided in IBC Section 1908.1.16 provide similar increases. It was never intended that non-ductile anchor force increase factors for nonstructural components be applied twice.

Cost Impact: This change is expected to reduce the cost of anchorage of nonstructural components attached to concrete.

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This code change adds exceptions to correct a duplication in the penalty on non-ductile anchors supporting non-structural components.

Assembly Action: **None**

Final Hearing Results

S56-06/07 **AS**

Code Change No: S57-06/07

Original Proposal

Sections: 2101.3.1, 2305.1.3

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

Revise as follows:

2101.3.1 Fireplace drawings. The construction documents shall describe in sufficient detail the location, size and construction of masonry fireplaces. The thickness and characteristics of materials and the clearances from walls, partitions and ceilings shall be ~~clearly~~ indicated.

2305.1.3 Openings in shear panels. Openings in shear panels that materially affect their strength shall be ~~fully~~ detailed on the plans, and shall have their edges adequately reinforced to transfer all shearing stresses.

Reason: Substitute revised material for current provision of the code. This proposal is a continuation of code change proposal S3-04/05 (AM), which proposed revisions similar to those in this proposal. The purpose of the proposal is to clarify the provisions of IBC Sections 2101.3.1 and 2305.1.3 for documentation on construction documents. The term "clearly" is deleted from Section 2101.3.1 because it is superfluous to require clearances to be clearly indicated on construction documents. Requiring clearances to be indicated on construction documents is sufficiently clear. The term "fully" is deleted from Section 2205.1.3 because it is not possible to fully detail openings on plans. What is needed is a sufficient number of details so that the building or structure can be constructed as intended by the design team, which is conveyed by requiring that the openings be detailed.

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This proposal makes editorial changes to provisions that require information to be included in construction documents. The revised wording is more concise.

Assembly Action: **None**

Final Hearing Results

S57-06/07 **AS**

Code Change No: S64-06/07

Original Proposal

Section: 2302.1

Proponent: Sam Francis, American Forest & Paper Association

Revise as follows:

SECTION 2302
DEFINITIONS

NATURALLY DURABLE WOOD. The heartwood of the following species with the exception that an occasional piece with corner sapwood is permitted if 90 percent or more of the width of each side on which it occurs is heartwood.

Decay resistant. Redwood, cedar, black locust and black walnut.

Termite resistant. Redwood, Alaska yellow cedar, Eastern red cedar and both heartwood and all sapwood of western red cedar.

Reason: This proposal is intended to clarify the code requirements for special inspections. Many common items are fabricated under standards cited in the IBC. Many of those items are fabricated with strict quality assurance done under third party supervision. In addition, the proposal also eliminates laundry lists from the code text. Such lists make interpretation and maintenance of the code awkward at best but potentially very, very difficult.

This change introduces species recently found to be termite resistant. Special emphasis of the study was Formosan termite resistance which is of great importance to gulf coast states trying to rebuild following recent hurricanes. These states are particularly susceptible to the Formosan termite.

Cost Impact: In areas suffering widespread damage, construction materials can become scarce and, thus, costly. More choices typically lead to less cost pressure.

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This code change identifies termite resistant properties in order to clarify the definition of naturally durable wood.

Assembly Action: **None**

Final Hearing Results

S64-06/07 **AS**

Code Change No: S65-06/07

Original Proposal

Section: 2302.1

Proponent: Joseph Holland, Hoover Treated Wood Products

Revise as follows:

SECTION 2302
DEFINITIONS

~~**PRESERVATIVE-TREATED WOOD**. Wood (including plywood) pressure-treated with preservatives in accordance with Section 2303.1.8.~~

TREATED WOOD. ~~Wood impregnated under pressure with compounds that reduce its susceptibility to flame spread or to deterioration caused by fungi, insects or marine borers.~~ Wood and wood based materials that use vacuum-pressure impregnation processes to enhance fire retardant or preservative properties.

Fire-retardant-treated wood. Pressure-treated lumber and plywood that exhibit reduced surface burning characteristics and resist (prevent) propagation of fire.

Preservative-treated wood. Pressure-treated wood products that exhibit reduced susceptibility to damage by fungi, termites, or marine borers.

Reason: Add additional required information for user in determining what treated wood is. Revise definition of preservative treated wood. Add definition for fire-retardant-treated wood. Make preservative and fire-retardant treated wood a subset of treated wood.

Currently there are two types of treated wood: fire-retardant-treated wood and preservative-treated wood. The current definition only speaks on one of the attributes for the fire-retardant-treated wood. The ability of the wood to extinguish itself once the source of ignition is consumed or removed is an important element of the material. The definition of preservative treated wood is not a definition; it's merely a reference to another section of the code. In addition, preservative treated wood will not reduce susceptibility to all insects, only those that actually eat the wood.

Section 2303.2 requires testing in accordance with ASTM E84. The section requires the test to be continued 20 minutes beyond the 10 minutes required to establish the flame spread. According to Section 2303.2, there can be no significant progressive combustion.

Cost Impact: The code change proposal will not increase the cost of construction. Material in marketplace already meets the requirements of Section 2303.2 IBC.

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

SECTION 2302
DEFINITIONS

TREATED WOOD. Wood and wood based materials that use vacuum-pressure impregnation processes to enhance fire retardant or preservative properties.

Fire-retardant-treated wood. Pressure-treated lumber and plywood that exhibit reduced surface burning characteristics and resist ~~(prevent)~~ propagation of fire.

Preservative-treated wood. Pressure-treated wood products that exhibit reduced susceptibility to damage by fungi, ~~termites~~ insects, or marine borers.

Committee Reason: The proposal provides appropriate editorial clarifications in the definitions relating to treated wood. The modification retains insects rather than referring exclusively to termites under preservative-treated wood.

Assembly Action: **None**

Final Hearing Results

S65-06/07 **AM**

Code Change No: S66-06/07

Original Proposal

Section: 2303.4

Proponent: Kirk Grundahl, P.E., Wood Truss Council of America representing the Structural Building Components Industry

Revise as follows:

2303.4 Trusses.

2303.4.1 Design. Wood trusses shall be designed in accordance with the provisions of this code and accepted engineering practice. Members are permitted to be joined by nails, glue, bolts, timber connectors, metal connector plates or other approved framing devices.

~~**2303.4.1.1 Truss designer.** The individual or organization responsible for the design of trusses.~~

~~**2303.4.1.2**~~ **2303.4.1.1Truss design drawings.** The written, graphic and pictorial depiction of each individual truss shall be provided to the building official ~~and~~ for approval ~~approved~~ prior to installation. Truss design drawings shall also be provided with the shipment of trusses delivered to the job site. Truss design drawings shall include, at a minimum, the information specified below:

1. Slope or depth, span and spacing;
2. Location of all joints;
3. Required bearing widths;
4. Design loads as applicable;
   ~~5~~4.1. Top chord live load (including snow loads);
   ~~6~~4.2. Top chord dead load;
   ~~7~~4.3. Bottom chord live load;
   ~~8~~4.4. Bottom chord dead load;
   ~~9~~4.5. Concentrated loads and their points of application as applicable;
   ~~10~~4.6. Controlling wind and earthquake loads as applicable;

~~11~~5. Adjustments to wood member ~~lumber~~ and metal connector plate design value for conditions of use;
~~12~~6. Each reaction force and direction;
~~13~~7. Metal connector plate type, size, and thickness or gage, and the dimensioned location of each metal connector plate except where symmetrically located relative to the joint interface;
~~14~~8. ~~Lumber s~~ Size, species and grade for each wood member;
~~15~~9. Connection capacities for:
   ~~15.1~~9.1. Truss to truss;
   ~~15.2~~9.2. Truss ply to ply; and
   ~~15.3~~9.3. Field splices.

~~16~~10. Calculated deflection ratio and maximum vertical and horizontal deflection for live and total load as applicable;
~~17~~11. Maximum axial ~~tensile~~ tension and compression forces in the truss members; and
~~18~~12. Required permanent individual truss member bracing and method per Section 2303.4.1.5, unless a specific truss member permanent bracing plan for the roof or floor structural system is provided by a registered design professional.

~~Where required by one of the following, each individual truss design drawing shall bear the seal and signature of the truss designer:~~

~~1. Registered design professional; or~~
~~2. Building official; or~~
~~3. Statutes of the jurisdiction in which the project is to be constructed.~~

~~**Exceptions:**~~

1. ~~When a cover sheet/truss index sheet combined into a single cover sheet is attached to the set of truss design drawings for the project, the single sheet/truss index sheet is the only document that needs to be signed and sealed within the truss submittal package.~~
2. ~~When a cover sheet and a truss index sheet are separately provided and attached to the set of truss design drawings for the project, both the cover sheet and the truss index sheet are the only documents that need to be signed and sealed within the truss submittal package.~~

~~**2303.4.1.3 Truss placement diagram.** The truss manufacturer shall provide a truss placement diagram that identifies the proposed location for each individually designated truss and references the corresponding truss design drawing. The truss placement diagram shall be provided as part of the truss submittal package, and with the shipment of trusses delivered to the job site. Truss placement diagrams shall not be required to bear the seal or signature of the truss designer.~~

~~**Exception:** When the truss placement diagram is prepared under the direct supervision of a registered design professional, it is required to be signed and sealed.~~

~~**2303.4.1.4 Truss submittal package.** The truss submittal package shall consist of each individual truss design drawing, the truss placement diagram for the project, the truss member permanent bracing specification and, as applicable, the cover sheet/truss index sheet.~~

~~2303.4.1.5~~ 2303.4.1.2 Truss member permanent bracing. Where permanent bracing of truss members is required on the truss design drawings, it shall be accomplished by one of the following methods:

1. The trusses shall be designed so that the buckling of any individual truss member ~~can be~~ is resisted internally by the ~~structure (e.g. buckling member T-bracing, L-bracing, etc.) of the~~ individual truss through suitable means (i.e., buckling reinforcement by T-bracing or L-bracing). The ~~truss individual member~~ buckling reinforcement of individual members of the trusses shall be installed as shown on the truss design drawing or on supplemental truss member buckling reinforcement ~~diagrams~~ details provided by the truss designer.

2. Permanent bracing shall be installed using standard industry lateral bracing details ~~that conform~~ in accordance with generally accepted engineering practice. ~~Individual truss member continuous~~ Locations for lateral bracing ~~location(s)~~ shall be ~~shown~~ identified on the truss design drawing.

2303.4.1.3 Truss designer. The individual or organization responsible for the design of trusses.

2303.4.1.3.1 Truss design drawings. Where required by the registered design professional, the building official, or the statutes of the jurisdiction in which the project is to be constructed, each individual truss design drawing shall bear the seal and signature of the truss designer:

Exceptions:

1. Where a cover sheet and truss index sheet are combined into a single sheet and attached to the set of truss design drawings the single cover/truss index sheet is the only document required to be signed and sealed by the truss designer.
2. When a cover sheet and a truss index sheet are separately provided and attached to the set of truss design drawings the cover sheet and the truss index sheet are the only documents required to be signed and sealed by the truss designer.

2303.4.2 Truss placement diagram. The truss manufacturer shall provide a truss placement diagram that identifies the proposed location for each individually designated truss and references the corresponding truss design drawing. The truss placement diagram shall be provided as part of the truss submittal package, and with the shipment of trusses delivered to the job site. Truss placement diagrams shall not be required to bear the seal or signature of the truss designer.

Exception: When the truss placement diagram is prepared under the direct supervision of a registered design professional, it is required to be signed and sealed.

2303.4.3 Truss submittal package. The truss submittal package shall consist of each individual truss design drawing, the truss placement diagram, the truss member permanent bracing details and, as applicable, the cover/truss index sheet.

~~2303.4.1.6~~ 2303.4.4 Anchorage. ~~All transfer~~ Transfer of loads and anchorage of each truss to the supporting structure is the responsibility of the registered design professional.

~~2303.4.1.7~~ 2303.4.5 Alterations to trusses. Truss members and components shall not be cut, notched, drilled, spliced or otherwise altered in any way without written concurrence and approval of a registered design professional. Alterations resulting in the addition of loads to any member (e.g., HVAC equipment, water heater) shall not be permitted without verification that the truss is capable of supporting such additional loading.

~~2303.4.2~~ 2303.4.6 Metal-plate-connected trusses. In addition to Sections 2303.4.1 through ~~2303.4.1.7~~ 2303.4.5, the design, manufacture and quality assurance of metal-plate-connected wood trusses shall be in accordance with TPI 1. Manufactured trusses shall comply with Section 1704.6 as applicable.

Reason: To make editorial improvements to the language and arrangement approved by code change S165-04/05. The language improvements are to provide more precision in the code. The restructuring of the section provides clearer presentation of the concepts.

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

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

2303.4 Trusses.

2303.4.1 Design. Wood trusses shall be designed in accordance with the provisions of this code and accepted engineering practice. Members are permitted to be joined by nails, glue, bolts, timber connectors, metal connector plates or other approved framing devices.

2303.4.1.1 Truss design drawings. The written, graphic and pictorial depiction of each individual truss shall be provided to the building official for approval prior to installation. Truss design drawings shall also be provided with the shipment of trusses delivered to the job site. Truss design drawings shall include, at a minimum, the information specified below:

1. Slope or depth, span and spacing;
2. Location of all joints;
3. Required bearing widths;
4. Design loads as applicable;
   4.1. Top chord live load (including snow loads);
   4.2. Top chord dead load;
   4.3. Bottom chord live load; 4.4. Bottom chord dead load;
   4.4. Concentrated loads and their points of application as applicable;
   4.6. Controlling wind and earthquake loads as applicable;
5. Adjustments to wood member and metal connector plate design value for conditions of use;
6. Each reaction force and direction;
7. Metal connector plate type, size, and thickness or gage, and the dimensioned location of each metal connector plate except where symmetrically located relative to the joint interface;
8. Size, species and grade for each wood member;
9. Specific connection capacities or ~~C~~ connection capacities required for:
   9.1. Truss to truss girder;
   9.2. Truss ply to ply; and
   9.3. Field ~~splices~~ assembly of a truss when the truss shown on the individual Truss Design Drawing is supplied in separate pieces that will be field connected.
10. Calculated deflection ratio and maximum vertical and horizontal deflection for live and total load as applicable;
11. Maximum axial tension and compression forces in the truss members; and
12. Required permanent individual truss member ~~bracing~~ restraint and method per Section 2303.4.1.2, unless a specific truss member permanent bracing plan for the roof or floor structural system is provided by a registered design professional.

2303.4.1.2 Permanent individual ~~T~~ truss member restraint ~~permanent bracing~~. Where permanent ~~bracing~~ restraint of truss members is required on the truss design drawings, it shall be accomplished by one of the following methods:

1. The trusses shall be designed so that the buckling of any individual truss member is resisted internally by the individual truss through suitable means (i.e., buckling reinforcement by T-reinforcement ~~bracing~~ or L-reinforcement ~~bracing~~). The buckling reinforcement of individual members of the trusses shall be installed as shown on the truss design drawing or on supplemental truss member buckling reinforcement details provided by the truss designer.
2. Permanent individual truss member restraint and diagonal bracing shall be installed using standard industry lateral restraint and diagonal bracing details in accordance with generally accepted engineering practice. Locations for lateral ~~bracing~~ restraint shall be identified on the truss design drawing.

2303.4.1.3 Truss designer. The individual or organization responsible for the design of trusses.

2303.4.1.3.1 Truss design drawings. Where required by the registered design professional, the building official, or the statutes of the jurisdiction in which the project is to be constructed, each individual truss design drawing shall bear the seal and signature of the truss designer:

Exceptions:

1. Where a cover sheet and truss index sheet are combined into a single sheet and attached to the set of truss design drawings the single cover/truss index sheet is the only document required to be signed and sealed by the truss designer.
2. When a cover sheet and a truss index sheet are separately provided and attached to the set of truss design drawings the cover sheet and the truss index sheet are the only documents required to be signed and sealed by the truss designer.

2303.4.2 Truss placement diagram. The truss manufacturer shall provide a truss placement diagram that identifies the proposed location for each individually designated truss and references the corresponding truss design drawing. The truss placement diagram shall be provided as part of the truss submittal package, and with the shipment of trusses delivered to the job site. Truss placement diagrams shall not be required to bear the seal or signature of the truss designer.

Exception: When the truss placement diagram is prepared under the direct supervision of a registered design professional, it is required to be signed and sealed.

2303.4.3 Truss submittal package. The truss submittal package shall consist of each individual truss design drawing, the truss placement diagram, the truss member permanent bracing details and, as applicable, the cover/truss index sheet.

2303.4.4 Anchorage. Transfer of loads and anchorage of each truss to the supporting structure is the responsibility of the registered design professional.

2303.4.5 Alterations to trusses. Truss members and components shall not be cut, notched, drilled, spliced or otherwise altered in any way without written concurrence and approval of a registered design professional. Alterations resulting in the addition of loads to any member (e.g., HVAC equipment, water heater) shall not be permitted without verification that the truss is capable of supporting such additional loading.

2303.4.6 Metal-plate-connected trusses. In addition to Sections 2303.4.1 through 2303.4.5, the design, manufacture and quality assurance of metal-plate-connected wood trusses shall be in accordance with TPI 1. Manufactured trusses shall comply with Section 1704.6 as applicable.

Committee Reason: The proposal provides better organization for wood truss design requirements. The truss placement plan should be part of the design drawings and should be reviewed by the engineer of record. The modification to Section 2304.1.1 clarifies the requirements applicable to truss connections. The modification to Section 2303.4.1.2 coordinates the code text with that of the ANSI/TPI 1 standard.

Assembly Action: **None**

Final Hearing Results

S66-06/07 **AM**

Code Change No: S68-06/07

Original Proposal

Section: 2303.4.2

Proponent: Kirk Grundahl, P.E., Wood Truss Council of America, representing the Structural Building Components Industry

Revise as follows:

2303.4.2 Metal-plate-connected trusses. In addition to Sections 2303.4.1 through 2303.4.1.7, the design, manufacture and quality assurance of metal-plate-connected wood trusses shall be in accordance with TPI 1. Manufactured trusses shall comply with Sections 106 and 109 ~~1704.6~~ as applicable.

Reason: The purpose of the proposed revision is to more clearly indicate:

1. That it is acceptable for a building code official to accept the requirements of TPI 1, as they pertain to 3rd party inspection and in-plant quality control, as meeting the requirements for approved inspections.
2. That the requirement for special inspections is job specific and is only imposed if special inspections are required per the construction documents as part of the submittal process to gain a permit.
   The provisions in Sections 104.4 Inspections, 106 Construction Documents and 109 Inspections lay out how the construction project process functions in terms of permits, construction documents, and general inspections and is the implementing language for when Chapter 17 applies.
3. Per 109.3, the building official is responsible for building construction inspections.
4. Per 109.3.4, the building official is responsible for providing the frame inspection.
5. Per 104.4 & 109.4, the building official is authorized to accept reports of an approved inspection agency.
6. As part of this frame inspection in 109.3.4, products that are manufactured in accordance with a referenced standard, under a quality assurance program audited by an approved inspection agency in accordance with 109.4 have typically been allowed for use in the construction process.
7. The inspection of truss manufacturing operations is required by ANSI/TPI 1-2002, the standard referenced for trusses, in Section 2303.4.2, ANSI/TPI 1includes the following requirement:
   3.1.3 Truss Manufacturers and inspection agencies shall establish methods that document the application of these quality assurance procedures throughout the manufacturing process. The Truss Manufacturers' methods shall be subject to periodic audit for compliance with the requirements of this standard by an approved inspection agency, where required by local authorities having jurisdiction, or other means.

The use of an approved third party inspection agency for the inspection of truss manufacturing operations meets the general inspection requirements of Section 109.4.

Special Inspections are required only if specifically required for a project as part of the submittal process to gain a permit for the building to be constructed (Sections 106.1 and 1704.1.1). A Special Inspection may be called for by one of three parties: the owner, the registered design professional, or the building official that reviews the application for a permit.

Section 1704.6 specifies that Special Inspections of the fabrication process of prefabricated wood structural elements and assemblies shall be in accordance with 1704.2.

Metal plate-connected wood truss manufacturers meet the requirements of Section 1704.2.2, Fabricator Approval, as referenced in the Exception in Section 1704.2.1:

> **Exception:** Special inspections as required by Section 1704.2 shall not be required where the fabricator is approved in accordance with Section 1704.2.2.

A metal plate-connected wood truss manufacturer may be approved to perform fabrication per the requirements of Section 1704.2.2 as follows:

1. An approved fabricator is one that has written procedural and quality control manuals and periodic auditing of fabrication practices by an approved special inspection agency, both of which are required by ANSI/TPI 1-2002:
   3.1.3 Truss Manufacturers and inspection agencies shall establish methods that document the application of these quality assurance procedures throughout the manufacturing process. The Truss Manufacturers' methods shall be subject to periodic audit for compliance with the requirements of this standard by an approved inspection agency, where required by local authorities having jurisdiction, or other means.
   3.2.1 An in-plant quality control manual shall be maintained for each truss manufacturing facility, which will include the requirements for daily quality control and any audits that will be performed."
2. An approved special inspection agency is certified by the International Accreditation Service (IAS) under the Accreditation Criteria for Inspection Agencies (AC98) or other approvals as accepted by the building official overseeing code compliance. It is our understanding that all the third party inspection agencies performing inspections in our industry are accredited by IAS.
3. The structural building components that are fabricated for the specific project are demonstrated to be compliant with the construction documents by the information provided on the Truss Design Drawings as required in Section 2303.4.1.2. It is still the responsibility of the building designer to review the Truss Design Drawings for compatibility with the design of the building per Section 106.3.4.

In addition to the reasoning provided above, we do not believe that metal plate connected wood trusses require special inspection. The definition for Special Inspection as provided in 1702.1 indicates that special inspections are required when "special expertise is needed to ensure compliance with approved construction documents and referenced standards." Inspection of the materials used in the manufacturing of metal plate connected wood trusses does require any special qualifications. The truss design drawing as defined in Section 2303.4.1.2 provides all the information that the building inspector or his or her designee needs to inspect the trusses at the job site as part of the framing inspection.

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

Public Hearing Results

Committee Action: **Approved as Modified**

Modify the proposal as follows:

2303.4.2 Metal-plate-connected trusses. In addition to Sections 2303.4.1 through 2303.4.1.7, the design, manufacture and quality assurance of metal-plate-connected wood trusses shall be in accordance with TPI 1. ~~Manufactured trusses shall comply~~ Jobsite inspections shall be in compliance with Section~~s 106 and~~ 109.4 as applicable.

Committee Reason: This code change clarifies the inspections of metal-plate-connected trusses. The modification provides a specific reference to the job site inspections that are typically required for these trusses.

Assembly Action: **None**

Final Hearing Results

S68-06/07 **AM**

Code Change No: S69-06/07

Original Proposal

Section: 2304.6.1

Proponent: Zeno Martin, P.E., APA-The Engineered Wood Association

Revise as follows:

2304.6.1 Wood structural panel sheathing. Where wood structural panel sheathing is used as the exposed finish on the exterior of outside walls, it shall have an exterior exposure durability classification. Where wood structural panel sheathing is used elsewhere, ~~on the exterior of outside walls~~ but not as the exposed finish, it shall be of a type manufactured with exterior glue (Exposure 1 or Exterior). ~~Where wood structural panel sheathing is used elsewhere, it shall be of a type manufactured with intermediate or exterior glue.~~

Reason: Delete reference to obsolete adhesive. Intermediate glue is no longer used in the manufacturing of structural panel sheathings trademarked to PS 1 or PS 2. This change improves the code by simplifying the provisions to reflect the product availability in the marketplace.

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This proposal appropriately deletes reference to an obsolete material.

Assembly Action: **None**

Final Hearing Results

S69-06/07 **AS**

Code Change No: S70-06/07

Original Proposal

Sections: 2304.6.1, Table 2304.6.1 (New); IRC R602.3, Table R602.3(3), R602.10.3

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: Zeno Martin, P.E., APA-The Engineered Wood Association

PART I – IBC

1. Revise as follows:

2304.6.1 Wood structural panel sheathing. Where wood structural panel sheathing is used as the exposed finish on the exterior of outside walls, it shall have an exterior exposure durability classification. Where wood structural panel sheathing is used on the exterior of outside walls but not as the exposed finish, it shall be of a type manufactured with exterior glue (Exposure 1 or Exterior). Where wood structural panel sheathing is used elsewhere, it shall be of a type manufactured with intermediate or exterior glue. Wood structural panel wall sheathing or siding used as structural sheathing shall be capable of resisting wind pressures in accordance with Section 1609. Maximum wind speeds for wood structural panel sheathing used to resist wind pressures shall be in accordance with Table 2304.6.1.

2. Add new table as follows:

TABLE 2304.6.1
MAXIMUM BASIC WIND SPEED (mph – 3 SECOND GUST) PERMITTED FOR WOOD STRUCTURAL PANEL WALL SHEATHING USED TO RESIST WIND PRESSURES[a,b,c]

MINIMUM NAIL		MINIMUM WOOD STRUCTURAL PANEL SPAN RATING	MINIMUM NOMINAL PANEL THICKNESS (INCHES)	MAXIMUM WALL STUD SPACING (INCHES)	PANEL NAIL SPACING		MAXIMUM WIND SPEED (MPH)		
							WIND EXPOSURE CATEGORY		
SIZE	PENETRATION (inches)				EDGES (INCHES O.C.)	FIELD (INCHES O.C.)	B	C	D
6d Common (0.113" x 2.0")	1.5	24/0	3/8	16	6	12	110	90	85
		24/16	7/16	16	6	12	110	100	90
						6	150	125	110
8d Common (0.131" x 2.5")	1.75	24/16	7/16	16	6	12	130	110	105
						6	150	125	110
				24	6	12	110	90	85
						6	110	90	85

a. Panel strength axis parallel or perpendicular to supports. Three-ply plywood sheathing with studs spaced more than 16 inches on center shall be applied with panel strength axis perpendicular to supports.
b. Table is based on wind pressures acting toward and away from building surfaces in accordance with Section 6.4.2.2 of ASCE 7. Lateral requirements shall be in accordance with Section 2305 or Section 2308.
c. Wood Structural Panels with span ratings of Wall-16 or Wall-24 shall be permitted as an alternate to panels with a 24/0 span rating. Plywood Siding rated 16 oc or 24 oc shall be permitted as an alternate to panels with a 24/16 span rating. Wall-16 and Plywood Siding 16 oc shall be used with studs spaced a maximum of 16 inches on center.

PART II – IRC

1. Revise as follows:

R602.3 Design and Construction. Exterior walls of wood-frame construction shall be designed and constructed in accordance with the provisions of this chapter and Figures R602.3(1) and R602.3.(2) or in accordance with AF&PA's NDS. Components of exterior walls shall be fastened in accordance with Table R602.3(1) through R602.3(4).

Exterior walls covered with foam plastic sheathing shall be braced in accordance with Section R602.10. Structural sheathing shall be fastened directly to structural framing members. Wall sheathing or siding shall be capable of resisting wind pressures listed in Table R301.2(2). Maximum wind speeds permitted for exterior walls covered with wood structural panel sheathing are listed in Table R602.3(3).

2. Delete Table R602.3(3) and substitute as follows:

TABLE R602.3(3)
MAXIMUM WIND SPEED (mph – 3 SECOND GUST) PERMITTED FOR
WOOD STRUCTURAL PANEL WALL SHEATHING USED TO RESIST WIND PRESSURES[a, b, c]

Minimum Nail		Minimum Wood Structural Panel Span Rating	Minimum Nominal Panel Thickness (inches)	Maximum Wall Stud Spacing (inches)	Panel Nail Spacing		Maximum Wind Speed (mph)		
Size	Penetration (inches)				Edges (inches o.c.)	Field (inches o.c.)	Wind Exposure Category		
							B	C	D
6d Common (0.113" x 2.0")	1.5	24/0	3/8	16	6	12	110	90	85
8d Common (0.131" x 2.5")	1.75	24/16	7/16	16	6	12	130	110	105
				24	6	12	110	90	85

a. Panel strength axis parallel or perpendicular to supports. Three-ply plywood sheathing with studs spaced more than 16 inches on center shall be applied with panel strength axis perpendicular to supports.
b. Table is based on wind pressures acting toward and away from building surfaces in accordance with Section R301.2. Lateral bracing requirements shall be in accordance with R602.10.
c. Wood Structural Panels with span ratings of Wall-16 or Wall-24 shall be permitted as an alternate to panels with a 24/0 span rating. Plywood Siding rated 16 oc or 24 oc shall be permitted as an alternate to panels with a 24/16 span rating. Wall-16 and Plywood Siding 16 oc shall be used with studs spaced a maximum of 16 inches on center.

R602.10.3 Braced wall panel construction methods. The construction of braced wall panels shall be in accordance with one of the following methods:

1. Nominal 1-inch-by-4-inch (25mmby 102 mm) continuous diagonal braces let in to the top and bottom plates and the intervening studs or approved metal strap devices installed in accordance with the manufacturer's specifications. The let-in bracing shall be placed at an angle not more than 60 degrees (1.06 rad) or less than 45 degrees (0.79 rad) from the horizontal.
2. Wood boards of 5/8 inch (16 mm) net minimum thickness applied diagonally on studs spaced a maximum of 24 inches (610 mm). Diagonal boards shall be attached to studs in accordance with Table R602.3(1).
3. Wood structural panel sheathing with a thickness not less than 5/16 inch (8 mm) for 16-inch (406 mm) stud spacing and not less than 3/8 inch (9 mm) for 24-inch (610 mm) stud spacing. Wood structural panels shall be installed in accordance with Table R602.3(~~3~~ 1).
4. One-half-inch (13 mm) or 25/32-inch (20 mm) thick structural fiberboard sheathing applied vertically or horizontally on studs spaced a maximum of 16 inches (406 mm) on center. Structural fiberboard sheathing shall be installed in accordance with Table R602.3(1).
5. Gypsum board with minimum 1/2-inch (13 mm) thickness placed on studs spaced a maximum of 24 inches (610 mm)on center and fastened at 7 inches (178 mm) on center with the size nails specified in Table R602.3(1) for sheathing and Table R702.3.5 for interior gypsum board.
6. Particleboard wall sheathing panels installed in accordance with Table R602.3(4).
7. Portland cement plaster on studs spaced a maximum of 16 inches (406 mm) on center and installed in accordance with Section R703.6.
8. Hardboard panel siding when installed in accordance with Table R703.4.

Exception: Alternate braced wall panels constructed in accordance with Section R602.10.6.1 or R602.10.6.2 shall be permitted to replace any of the above methods of braced wall panels.

Reason: The code change provides guidelines for using wood structural panel wall sheathing to resist wind loads.

Recent high wind events including Hurricane Katrina and several tornado storms have shown that failure of wall sheathing, in winds as low as 60 mph, has caused significant damage due to breaching of the wall envelope. This code change proposal provides wall sheathing solutions using wood structural panels to resist wind pressures. The code change proposal provides a new table 2304.6.1 which clearly shows the capabilities of wood structural panel cladding at varying wind speeds and exposures.

The proposed IBC Table 2304.6.1 was developed by comparing the wind pressures (wind speed) required by Section 1609.3 given in ASCE 7-05 Figure 6-3 (formerly 2003 IBC Table 1609.6.2.1(2)) with the wood structural panel capacities based on US DOC PS 2 standard, engineering calculations, and the Panel Design Specification referenced in 2006 IBC Section 2306.1. The proposed IRC Table R602.3(3) was developed by comparing the wind pressures (wind speed) given in Table R301.2(2) with the wood structural panel capacities based on US DOC PS 2 standard, engineering calculations, and the Panel Design Specification referenced in 2006 IBC Section 2306.1. Nail head pull through and withdrawal was also considered in addition to the panel stiffness and bending strength. The panel-fastener capacity was based on tributary to a single critical nail.

This code change proposal improves the code because it clearly defines the maximum wind speed permitted for wood structural panel wall sheathing.

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

Public Hearing Results

PART I — IBC
Committee Action: Approved as Modified

Modify proposal as follows:

2304.6.1 Wood structural panel sheathing. Where wood structural panel sheathing is used as the exposed finish on the exterior of outside walls, it shall have an exterior exposure durability classification. Where wood structural panel sheathing is used on the exterior of outside walls but not as the exposed finish, it shall be of a type manufactured with exterior glue (Exposure 1 or Exterior). Where wood structural panel sheathing is used elsewhere, it shall be of a type manufactured with intermediate or exterior glue. Wood structural panel wall sheathing or siding used as structural sheathing shall be capable of resisting wind pressures in accordance with Section 1609. Maximum wind speeds for wood structural panel sheathing used to resist wind pressures shall be in accordance with Table 2304.6.1 <u>for enclosed buildings with a mean roof height not greater than 30 feet (9144 mm) importance factor (I) of 1.0 and topographic factor (Kzt) of 1.0</u>.

TABLE 2304.6.1
MAXIMUM BASIC WIND SPEED (mph – 3 SECOND GUST) PERMITTED FOR WOOD STRUCTURAL PANEL WALL SHEATHING USED TO RESIST WIND PRESSURES[a,b,c]

MINIMUM NAIL		MINIMUM WOOD STRUCTURAL PANEL SPAN RATING	MINIMUM NOMINAL PANEL THICKNESS (INCHES)	MAXIMUM WALL STUD SPACING (INCHES)	PANEL NAIL SPACING		MAXIMUM WIND SPEED (MPH) WIND EXPOSURE CATEGORY		
SIZE	PENETRATION (INCHES)				EDGES (INCHES O.C.)	FIELD (INCHES O.C.)	B	C	D
6d Common (0.113" x 2.0")	1.5	24/0	3/8	16	6	12	110	90	85
		24/16	7/16	16	6	12	110	100	90
						6	150	125	110
8d Common (0.131" x 2.5")	1.75	24/16	7/16	16	6	12	130	110	105
						6	150	125	110
				24	6	12	110	90	85
						6	110	90	85

a. Panel strength axis parallel or perpendicular to supports. Three-ply plywood sheathing with studs spaced more than 16 inches on center shall be applied with panel strength axis perpendicular to supports.
b. Table is based on wind pressures acting toward and away from building surfaces in accordance with Section 6.4.2.2 of ASCE 7. Lateral requirements shall be in accordance with Section 2305 or Section 2308.
c. Wood Structural Panels with span ratings of Wall-16 or Wall-24 shall be permitted as an alternate to panels with a 24/0 span rating. Plywood Siding rated 16 oc or 24 oc shall be permitted as an alternate to panels with a 24/16 span rating. Wall-16 and Plywood Siding 16 oc shall be used with studs spaced a maximum of 16 inches on center.

Committee Reason: This code change adds requirements for wood structural panel wall sheathing that addresses concerns associated with high wind speeds. The modification places limitations on the tabulated values that are consistent with the assumptions used to calculate them.

Assembly Action: None

PART II – IRC
Committee Action: Disapproved

Committee Reason: This code change is confusing as to when it is referencing sheathing or siding. Siding is wall covering and is not appropriate to be referenced in this section. This has the appearance of proprietary.

Assembly Action: None

Final Hearing Results

S70-06/07, Part I AM
S70-06/07, Part II D

Code Change No: S72-06/07

Original Proposal

Table 2304.7(3), Table 2306.3.1, Table 2306.4.1, 2308.9.3, Table 2308.9.3(3); IRC Tables R503.2.1.1(1) - R602.3(1)- R602.3(3), R602.10.3

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.

PART I – IBC

Proponent: Edward L. Keith, P.E., APA-The Engineered Wood Association

Revise tables as follows:

TABLE 2304.7(3)
ALLOWABLE SPANS AND LOADS FOR WOOD STRUCTURAL PANEL SHEATHIHNG AND SINGLE-FLOOR GRADES CONTINUOUS OVER TWO OR MORE SPANS WITH STRENGTH AXIS PERPENDICULAR TO SUPPORTS [a,b]

SHEATHING GRADES		ROOF[c]				FLOOR[d]
Panel span rating roof/floor span	Panel thickness (inches)	Maximum span (inches)		Load[e] (psf)		Maximum span (inches)
		With edge support[f]	Without edge support	Total load	Live load	
~~12/0~~	~~5/16~~	~~12~~	~~12~~	~~40~~	~~30~~	~~0~~
16/0	~~5/16~~ 3/8	16	16	40	30	0
20/0	~~5/16~~ 3/8	20	20	40	30	0

(Portions of table not shown do not change)

TABLE 2306.3.1
ALLOWABLE SHEAR (POUNDS PER FOOT) FOR WOOD STRUCTURAL PANEL DIAPHRAGMS WITH FRAMING OF DOUGLAS FIR-LARCH, OR SOUTHERN PINE[a] FOR WIND OR SEISMIC LOADING [h]

PANEL GRADE	COMMON NAIL SIZE[f] OR STAPLE LENGTH AND GAGE	MINIMUM FASTENER PENETRATION IN FRAMING (inches)	MINIMUM NOMINAL PANEL THICKNESS (inch)	MINIMUM NOMINAL WIDTH OF FRAMING MEMBERS AT ADJOINING PANEL EDGES AND BOUNDARIES[g] (inches)	BLOCKED DIAPHRAGMS				UNBLOCKED DIAPHRAGM	
					Fastener spacing (inches) at diaphragm boundaries (all cases) at continuous panel edges parallel to load (Cases 3, 4), and at all panel edge (Cases 5, 6)[b]				Fasteners spaced 6" max. at supported edges[b]	
					6	4	2 1/2[c]	2[c]	Case 1 (No unblocked edges or continuous panel joints parallel to load)	All other configurations (Cases 2, 3, 4, 5 and 6)
					Fastener spacing (inches) at other panel edges (Cases 1, 2, 3 and 4)[b]					
					6	6	4	3		
Structural I grades	~~6d[e] (2" x 0.113")~~	~~1 — 1/4~~	~~5/16~~	~~2~~	~~185~~	~~250~~	~~375~~	~~420~~	~~165~~	~~125~~
				~~3~~	~~210~~	~~280~~	~~420~~	~~475~~	~~185~~	~~140~~
	~~1 — 1/2 16 Gage~~	~~1~~		~~2~~	~~155~~	~~205~~	~~310~~	~~350~~	~~135~~	~~105~~
				~~3~~	~~175~~	~~230~~	~~345~~	~~390~~	~~155~~	~~115~~
Sheathing, single floor and other grades covered in DOC PS1 and PS2	~~6d[e] (2" x 0.113")~~	~~1 — 1/4~~	~~5/16~~	~~2~~	~~170~~	~~225~~	~~335~~	~~380~~	~~150~~	~~110~~
				~~3~~	~~190~~	~~250~~	~~280~~	~~430~~	~~170~~	~~125~~
	~~1 – 1/2 16 Gage~~	~~1~~		~~2~~	~~140~~	~~185~~	~~275~~	~~315~~	~~125~~	~~90~~
				~~3~~	~~155~~	~~205~~	~~310~~	~~350~~	~~140~~	~~105~~

(Portions of table not shown do not change)

TABLE 2306.4.1
ALLOWABLE SHEAR (POUNDS PER FOOT) FOOR WOOD STRUCTURAL PANEL SHEAR WALLS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE [a] FOR WIND OR SEISMIC LOADING [b, h, i, j, l]

PANEL GRADE	MINIMUM NOMINAL PANEL THICKNESS (inch)	MINIMUM FASTENER PENETRATION IN FRAMING (inches)	PANELS APPLIED DIRECT TO FRAMING					PANELS APPLIED OVER ½" OR 5/8" GYPSUM SHEATHING				
			NAIL (common or galvanized box) or staple size[k]	Fastener spacing at panel edges (inches) 6	4	3	2[e]	NAIL (common or galvanized box) or staple size[k]	Fastener spacing at panel edges (inches) 6	4	3	2[e]
Structural I Sheathing	~~5/16~~	~~1 1/4~~	~~6d (2" x 0.113" common, 2" x 0.099" galvanized box)~~	~~200~~	~~300~~	~~390~~	~~510~~	~~8d (2 ½" x 0.131" common., 2 ½" x 0.113" galvanized box)~~	~~200~~	~~300~~	~~390~~	~~510~~
		~~1~~	~~1 ½ 16 gage~~	~~165~~	~~245~~	~~325~~	~~415~~	~~2 16 gage~~	~~125~~	~~185~~	~~245~~	~~315~~
Sheathing, plywood siding,[g] except Group 5 Species	5/16[c] or 1/4[c]	1 1/4	6d (2" x 0.113" common, 2" x 0.099" galvanized box)	180	270	350	450	8d (2 ½" x 0.131" common., 2 ½" x 0.113" galvanized box)	181	270	350	450
		1	1 ½ 16 gage	145	220	295	375	2 16 gage	110	165	220	285
			Nail size (galvanized casing)					Nail size (galvanized casing)				
	5/16[c]	1 1/4	6d (2" x 0.99")	140	210	275	360	8d (2 ½" x 0.113")	140	210	275	360
	3/8[c]	1 3/8	8d (2 ½" x 0.113")	160	240	310	410	10d (3" x 0.128")	160	240	310[f]	410[f]

(Portions of table not shown do not change)

a. and b. (No change to current text)

c. 3/8-inch panel thickness or siding with a span rating of 16 inches on center is the minimum recommended where applied direct to framing as exterior siding. For grooved panel siding, the nominal panel thickness is the thickness of the panel measured at the point of nailing.

d. through i. (No change to current text)

2308.9.3 Bracing. Braced wall lines shall consist of braced wall panels that meet the requirements for location, type and amount of bracing as shown in Figure 2308.9.3, specified in Table 2308.9.3(1) and are in line or offset from each other by not more than 4 feet (1219 mm). Braced wall panels shall start not more than 12 1/2-feet (3810 mm) from each end of a braced wall line. Braced wall panels shall be clearly indicated on the plans. Construction of braced wall panels shall be by one of the following methods:

1. Nominal 1-inch by 4-inch (25 mm by 102 mm) continuous diagonal braces let into top and bottom plates and intervening studs, placed at an angle not more than 60 degrees (1.0 rad) or less than 45 degrees (0.79 rad) from the horizontal and attached to the framing in conformance with Table 2304.9.1.
2. Wood boards of 5/8 inch (15.9 mm) net minimum thickness applied diagonally on studs spaced not over 24 inches (610 mm) o.c.
3. Wood structural panel sheathing with a thickness not less than ~~5/16~~3/8 inch (~~7.9 mm~~ 9.5 mm) for 16-inch (406 mm) ~~stud spacing and not less than 3/8 inch (9.5 mm) for~~ or 24-inch (610 mm) stud spacing in accordance with Tables 2308.9.3(2) and 2308.9.3(3).

(Remainder of section unchanged)

TABLE 2308.9.3(3)
WOOD STRUCTURAL PANEL WALL SHEATHING[b]
(Not Exposed to the Weather, strength axis Parallel or Perpendicular to Studs Except as Indicated Below)

MINIMUM THICKNESS (inch)	PANEL SPAN RATING	STUD SPACING (inches)		
		Siding nailed to studs	Nailable sheathing	
			Sheathing parallel to studs	Sheathing perpendicular to studs
~~5/16~~	~~12/0, 16/0, 20/0 Wall 16" o.c.~~	~~16~~	-	~~16~~
3/8, 15/32, 1/2	16/0, 20/0, 24/0, 32/16, Wall – 24" o.c.	24	16	24
7/16, 15/32, 1/2	24/0, 24/16, 32/16, Wall – 24" o.c.	24	24[a]	24

(No change to footnotes)

PART II – IRC

Revise tables as follows:

TABLE R503.2.1.1(1)
ALLOWABLE SPANS AND LOADS FOR WOOD STRUCTURAL PANELS
FOR ROOF AND SUBFLOOR SHEATHING AND COMBINATION SUBFLOOR AND UNDERLAYMENT [a,b,c]

SPAN RATING	MINIMUM NOMINAL PANEL THICKNESS (inch)	ALLOWABLE LIVE LOAD (psf) [h,l]		MAXIMUM SPAN (inches)		LOAD (pounds per square foot, at maximum span)		MAXIMUM SPAN (inches)
		SPAN @ 16" o.c.	SPAN @ 24" o.c.	With edge support [d]	Without edge support	Total load	Live load	
Sheathing[e]				Roof[f]				Subfloor[j]
~~12/0~~	~~5/16~~	--	--	~~12~~	~~12~~	~~40~~	~~30~~	~~0~~
16/0	~~5/16~~ 3/8	30	--	16	16	40	30	0
20/0	~~5/16~~ 3/8	50	--	20	20	40	30	0

(Remainder of table unchanged)

TABLE R602.3(1)
FASTENER SCHEDULE FOR STRUCTURAL MAMBERS

DESCRIPTION OF BUILDING MATERIALS	DESCRIPTION OF FASTENER [b,c,e]	SPACING OF FASTENERS	
		Edges (inches)[i]	Intermediate Supports [c,e] (inches)
Wood structural panels, subfloor, roof and wall sheathing to framing, and particleboard wall sheathing to framing			
~~5/16"~~ 3/8" – ½"	6d common (2" x 0.113") nail (subfloor, wall) 8d common (2½" x 0.131") nail (roof)[f]	6	12[g]

(Remainder of table unchanged)

TABLE R602.3(3)
WOOD STRUCTURAL PANEL WALL SHEATHING

PANEL SPAN RATING	PANEL NOMINAL THICKNESS (inch)	MAXIMUM STUD SPACING (inches)	
		Siding nailed to:[a]	
		Stud	Sheathing
~~12/0,~~ 16/0, 20/0, or wall – 16 o.c.	~~5/16~~ 3/8	16	16[b]
24/0, 24/16, 32/16 or wall - 24 o.c.	3/8, 7/16, 15/32, 1/2	24	24[c]

(Notes unchanged)

R602.10.3 Braced wall panel construction methods. The construction of braced wall panels shall be in accordance with one of the following methods:

1. Nominal 1-inch-by-4-inch (25mmby 102 mm) continuous diagonal braces let in to the top and bottom plates and the intervening studs or approved metal strap devices installed in accordance with the manufacturer's specifications. The let-in bracing shall be placed at an angle not more than 60 degrees (1.06 rad) or less than 45 degrees (0.79 rad) from the horizontal.

2. Wood boards of 5/8 inch (16 mm) net minimum thickness applied diagonally on studs spaced a maximumof 24 inches (610 mm). Diagonal boards shall be attached to studs in accordance with Table R602.3(1).
3. Wood structural panel sheathing with a thickness not less than ~~5/16~~ 3/8 inch (~~8~~ 9.5 mm) for 16-inch (406 mm) ~~stud spacing and not less than 3/8 inch (9 mm) for~~ or 24-inch (610 mm) stud spacing. Wood structural panels shall be installed in accordance with Table R602.3(3).
4. One-half-inch (13 mm) or 25/32-inch (20 mm) thick structural fiberboard sheathing applied vertically or horizontally on studs spaced a maximum of 16 inches (406 mm) on center. Structural fiberboard sheathing shall be installed in accordance with Table R602.3(1).
5. Gypsum board with minimum 1/2-inch (13 mm) thickness placed on studs spaced a maximum of 24 inches (610 mm)on center and fastened at 7 inches (178 mm) on center with the size nails specified in Table R602.3(1) for sheathing and Table R702.3.5 for interior gypsum board.
6. Particleboard wall sheathing panels installed in accordance with Table R602.3(4).
7. Portland cement plaster on studs spaced a maximum of 16 inches (406 mm) on center and installed in accordance with Section R703.6.
8. Hardboard panel siding when installed in accordance with Table R703.4.

Exception: Alternate braced wall panels constructed in accordance with Section R602.10.6.1 or R602.10.6.2 shall be permitted to replace any of the above methods of braced wall panels.

Reason: Delete from code products no longer produced.

The 5/16" wood structural panels are currently a very small fraction of the panels produced today. While they have been the minimum panel thickness specified for many applications over the years, the building industry has shifted away from them due to manufacturing efficiencies and marketplace demand. The de facto minimum has become 3/8".

Note that the thickness of the panel at the point of nailing is what determines its shear capacity. A statement reflecting this was added to Footnote c in Table 2306.4.1. As such, panels as thick as 19/32" can have 3/8" to 5/16" remaining at the base of a groove. This is why the 5/16" minimum nominal was maintained for panels attached with siding nails. The annotation for Footnote c and f were added to Table 2306.4.1 as an editorial change.

Cost Impact: The code change proposal will not increase the cost of construction, as the current minimums are effectively no longer produced for structural purposes.

Public Hearing Results

PART I — IBC
Committee Action: **Approved as Submitted**

Committee Reason: This code change appropriately removes obsolete wood structural panel sizes from Chapter 23.

Assembly Action: **None**

PART II — IRC
Committee Action: **Approved as Submitted**

Committee Reason: This change deletes the 5/16" thick wood structural panel that is no longer widely available. The 3/8" thick wood structural panel is the proper thickness to be used for the bracing requirements.

Assembly Action: **None**

Final Hearing Results

S72-06/07, Part I	**AS**
S72-06/07, Part II	**AS**

Code Change No: S73-06/07

Original Proposal

Section: 2304.8

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

Revise as follows:

2304.8 Lumber decking.

2304.8.1 General. Lumber decking shall be designed and installed in accordance with the general provisions of this code and ~~the provisions of this~~ Section 2304.8. Each piece shall be square end-trimmed. When random lengths are furnished, each piece shall be square end trimmed across the face so that at least 90 percent of the pieces ~~will be~~ are within 0.5 degrees (0.00873 rad) of square. The ends of the pieces shall be permitted to be beveled up to 2 degrees (0.0349 rad) from the vertical with the exposed face of the piece slightly longer than the ~~back~~ opposite face of the piece. Tongue-and-groove decking shall be installed with the tongues up on sloped or pitched roofs with pattern faces down.

2304.8.2 Layup patterns. Lumber decking is permitted to be laid up following one of five standard patterns as defined in Sections 2304.8.2.1 through 2304.8.2.5. Other patterns are permitted to be used ~~if justified by~~ provided they are substantiated through engineering analysis.

2304.8.2.1 Simple span pattern. All pieces shall be supported on their ends (i.e., by two supports).

2304.8.2.2 Two-span continuous pattern. All pieces shall be supported by three supports, and all end joints shall occur in line on ~~every other~~ alternating supports. Supporting members shall be designed to accommodate the load redistribution caused by this pattern.

2304.8.2.3 Combination simple and two-span continuous pattern. Courses in end spans shall be alternating simple span pattern and two-span continuous pattern. End joints ~~are~~ shall be staggered in adjacent courses and ~~occur only over~~ shall bear on supports.

2304.8.2.4 Cantilevered pieces intermixed pattern. The decking shall ~~cover~~ extend across a minimum of three spans. Pieces in ~~the~~ each starter course and every third course shall be simple span pattern. Pieces in other courses shall be cantilevered over the supports with end joints at ~~alternate~~ alternating quarter or third points of the spans. ~~and~~ Each piece shall bear on at least one support.

2304.8.2.5 Controlled random pattern. The decking shall ~~cover~~ extend across a minimum of three spans. End joints of pieces within six inches (152 mm) of ~~being in line~~ the end joints of the adjacent pieces in either direction shall be separated by at least two intervening courses. In the end bays, each piece shall bear on at least one support. Where an end joint occurs in an end bay, the next piece in the same course shall continue over the first inner support for at least 24 inches (610 mm). The details of the controlled random pattern shall be as ~~described~~ specified for each decking material in Section~~s~~ 2304.8.3.3, 2308.4.3 or 2304.8.5.3.

~~For cantilevered spans with the controlled random pattern, special considerations shall be made when the overhang exceeds~~ Decking that cantilevers beyond a support for a horizontal distance greater than 18 inches (457 mm), 24 inches (610 mm) or 36 inches (914 mm) for two-inch (51 mm), three-inch (76 mm), and four-inch (102 mm) nominal thickness decking, respectively, shall comply with the following:

1. The maximum cantilevered length ~~for the controlled random pattern~~ shall be 30 percent of the length of the first adjacent interior span.
2. For cantilever overhangs within these limits, A structural fascia shall be fastened to each decking piece to maintain a continuous, straight ~~roof~~ line.
3. There shall be no end joints in the ~~cantilevered portion or within one-half~~ decking between the cantilevered end of the decking and the centerline of the first adjacent interior span.

2304.8.3 Mechanically laminated decking.

2304.8.3.1 General. Mechanically laminated decking consists of square edged dimension lumber laminations set on edge and nailed to ~~the~~ adjacent pieces and to the supports.

2304.8.3.2 Nailing. The length of nails connecting laminations shall not be less than two and one-half times the net thickness of each lamination. Where decking supports are 48 inches (1219 mm) on center (o.c.) or less, side nails shall be ~~spaced~~ installed not more than 30 inches (762 mm) o.c. ~~alternately near~~ alternating between top and bottom edges, and staggered one third of the spacing in adjacent laminations. Where supports are spaced more than 48 inches (1219 mm) o.c., side nails shall be ~~spaced~~ installed not more than 18 inches (457 mm) o.c. ~~alternately near~~ alternating between top and bottom edges and staggered one-third of the spacing in adjacent laminations. Two side nails shall be ~~used~~ installed at each end of butt-jointed pieces.

Laminations shall be toenailed to supports with 20d or larger common nails. Where the supports are 48 inches (1219 mm) o.c. or less, alternate laminations shall be toenailed to alternate supports; where supports are spaced more than 48 inches (1219 mm) o.c., alternate laminations shall be toenailed to every support.

2304.8.3.3 Controlled random pattern. There shall be a minimum distance of 24 inches (610 mm) between end joints in adjacent courses. The pieces in the first and second courses shall bear on at least two supports with end joints in these two courses occurring on alternate supports. A maximum of seven intervening courses shall be permitted before this pattern is repeated.

2304.8.4 Two-inch sawn tongue-and-groove decking.

2304.8.4.1 General. Two-inch (51 mm) decking shall have a maximum moisture content of 15 percent. Decking shall be machined with a single tongue-and-groove pattern. Each decking piece shall be nailed to each support ~~as required~~.

2304.8.4.2 Nailing. Each piece of decking shall be toenailed at each support with one 16d common nail through the tongue and face-nailed with one 16d common nail.

2304.8.4.3 Controlled random pattern. There shall be a minimum distance of 24 inches (610 mm) between end joints in adjacent courses. The pieces in the first and second courses shall bear on at least two supports with end joints in these two courses occurring on alternate supports. A maximum of seven intervening courses shall be permitted before this pattern is repeated.

2304.8.5 Three- and four-inch sawn tongue-and-groove decking.

2304.8.5.1 General. Three-inch (76 mm) and four-inch (102 mm) decking shall have a maximum moisture content of 19 percent. Decking shall be machined with a double tongue-and-groove pattern. Decking pieces shall be interconnected and ~~fastened~~ nailed to the supports ~~as required~~.

2304.8.5.2 Nailing. Each piece shall be toenailed at each support with one 40d common nail and face-nailed with one 60d common nail. Courses shall be spiked to each other with 8 inch (203 mm) spikes at maximum intervals ~~not to exceed~~ of 30 inches (762 mm) through predrilled edge holes penetrating to a depth of approximately 4 inches (102 mm). ~~and with~~ One spike shall be installed at a distance not exceeding 10 inches (254 mm) from the end of each piece.

2304.8.5.3 Controlled random pattern. There shall be a minimum distance of 48 inches (1219 mm) between end joints in adjacent courses. Pieces not bearing ~~over~~ on a support are permitted to ~~occur~~ be located in interior bays provided the adjacent pieces in the same course continue over the support for at least 24 inches (610 mm). This condition shall not occur more than once in every six courses in each interior bay.

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

The purpose of this proposal is to make editorial improvements to the language, which was approved by code change proposal S170-04/05(AMPC1).

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

Public Hearing Results

Committee Action: **Approved as Modified**

Modify the proposal as follows:

2304.8 Lumber decking.

2304.8.1 General. Lumber decking shall be designed and installed in accordance with the general provisions of this code and Section 2304.8. Each piece shall be square end-trimmed. When random lengths are furnished, each piece shall be square end trimmed across the face so that at least 90 percent of the pieces are within 0.5 degrees (0.00873 rad) of square. The ends of the pieces shall be permitted to be beveled up to 2 degrees (0.0349 rad) from the vertical with the exposed face of the piece slightly longer than the opposite face of the piece. Tongue-and-groove decking shall be installed with the tongues up on sloped or pitched roofs with pattern faces down.

2304.8.2 Layup patterns. Lumber decking is permitted to be laid up following one of five standard patterns as defined in Sections 2304.8.2.1 through 2304.8.2.5. Other patterns are permitted to be used provided they are substantiated through engineering analysis.

2304.8.2.1 Simple span pattern. All pieces shall be supported on their ends (i.e., by two supports).

2304.8.2.2 Two-span continuous pattern. All pieces shall be supported by three supports, and all end joints shall occur in line on alternating supports. Supporting members shall be designed to accommodate the load redistribution caused by this pattern.

2304.8.2.3 Combination simple and two-span continuous pattern. Courses in end spans shall be alternating simple span pattern and two-span continuous pattern. End joints shall be staggered in adjacent courses and shall bear on supports.

2304.8.2.4 Cantilevered pieces intermixed pattern. The decking shall extend across a minimum of three spans. Pieces in each starter course and every third course shall be simple span pattern. Pieces in other courses shall be cantilevered over the supports with end joints at alternating quarter or third points of the spans. Each piece shall bear on at least one support.

2304.8.2.5 Controlled random pattern. The decking shall extend across a minimum of three spans. End joints of pieces within six inches (152 mm) of the end joints of the adjacent pieces in either direction shall be separated by at least two intervening courses. In the end bays, each piece shall bear on at least one support. Where an end joint occurs in an end bay, the next piece in the same course shall continue over the first inner support for at least 24 inches (610 mm). The details of the controlled random pattern shall be as specified for each decking material in Section 2304.8.3.3, 2308.4.3 or 2304.8.5.3.

Decking that cantilevers beyond a support for a horizontal distance greater than 18 inches (457 mm), 24 inches (610 mm) or 36 inches (914 mm) for two-inch (51 mm), three-inch (76 mm), and four-inch (102 mm) nominal thickness decking, respectively, shall comply with the following:

1. The maximum cantilevered length shall be 30 percent of the length of the first adjacent interior span.
2. ~~For cantilever overhangs within these limits,~~ A structural fascia shall be fastened to each decking piece to maintain a continuous, straight line.
3. There shall be no end joints in the decking between the cantilevered end of the decking and the centerline of the first adjacent interior span.

2304.8.3 Mechanically laminated decking.

2304.8.3.1 General. Mechanically laminated decking consists of square edged dimension lumber laminations set on edge and nailed to adjacent pieces and to the supports.

2304.8.3.2 Nailing. The length of nails connecting laminations shall not be less than two and one-half times the net thickness of each lamination. Where decking supports are 48 inches (1219 mm) on center (o.c.) or less, side nails shall be installed not more than 30 inches (762 mm) o.c. alternating between top and bottom edges, and staggered one third of the spacing in adjacent laminations. Where supports are spaced more than 48 inches (1219 mm) o.c., side nails shall be installed not more than 18 inches (457 mm) o.c. alternating between top and bottom edges and staggered one-third of the spacing in adjacent laminations. Two side nails shall be installed at each end of butt-jointed pieces.

Laminations shall be toenailed to supports with 20d or larger common nails. Where the supports are 48 inches (1219 mm) o.c. or less, alternate laminations shall be toenailed to alternate supports; where supports are spaced more than 48 inches (1219 mm) o.c., alternate laminations shall be toenailed to every support.

2304.8.3.3 Controlled random pattern. There shall be a minimum distance of 24 inches (610 mm) between end joints in adjacent courses. The pieces in the first and second courses shall bear on at least two supports with end joints in these two courses occurring on alternate supports. A maximum of seven intervening courses shall be permitted before this pattern is repeated.

2304.8.4 Two-inch sawn tongue-and-groove decking.

2304.8.4.1 General. Two-inch (51 mm) decking shall have a maximum moisture content of 15 percent. Decking shall be machined with a single tongue-and-groove pattern. Each decking piece shall be nailed to each support.

2304.8.4.2 Nailing. Each piece of decking shall be toenailed at each support with one 16d common nail through the tongue and face-nailed with one 16d common nail.

2304.8.4.3 Controlled random pattern. There shall be a minimum distance of 24 inches (610 mm) between end joints in adjacent courses. The pieces in the first and second courses shall bear on at least two supports with end joints in these two courses occurring on alternate supports. A maximum of seven intervening courses shall be permitted before this pattern is repeated.

2304.8.5 Three- and four-inch sawn tongue-and-groove decking.

2304.8.5.1 General. Three-inch (76 mm) and four-inch (102 mm) decking shall have a maximum moisture content of 19 percent. Decking shall be machined with a double tongue-and-groove pattern. Decking pieces shall be interconnected and nailed to the supports.

2304.8.5.2 Nailing. Each piece shall be toenailed at each support with one 40d common nail and face-nailed with one 60d common nail. Courses shall be spiked to each other with 8 inch (203 mm) spikes at maximum intervals of 30 inches (762 mm) through predrilled edge holes penetrating to a depth of approximately 4 inches (102 mm). One spike shall be installed at a distance not exceeding 10 inches (254 mm) from the end of each piece.

2304.8.5.3 Controlled random pattern. There shall be a minimum distance of 48 inches (1219 mm) between end joints in adjacent courses. Pieces not bearing on a support are permitted to be located in interior bays provided the adjacent pieces in the same course continue over the support for at least 24 inches (610 mm). This condition shall not occur more than once in every six courses in each interior bay.

Committee Reason: This code change fixes the language of the lumber decking provisions that were incorporated into the IBC during the previous code development cycle. The modification makes an additional editorial correction.

Assembly Action: **None**

Final Hearing Results

S73-06/07 **AM**

Code Change No: S74-06/07

Original Proposal

Table 2304.9.1

Proponent: Randall Shackelford, Simpson Strong-Tie Co.

Revise table as follows:

TABLE 2304.9.1
FASTENING SCHEDULE

CONNECTION	FASTENING [a,m]	LOCATION
30. Ledger strip	3 - 16d common (3½"× 0.162″) 4 - 3" x 0.131″ nails 4 – 3" 14 gage staples	face nail <u>at each joist</u>

Reason: Clarify the code to show where nails must be applied. Fastening was added in the last code change cycle, but location of the fasteners was not added. Fasteners must be located under or near each joist. Legacy codes were silent on this, except for the Standard Building Code, which required "3 at each joist".

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This proposal provides a useful clarification on the location of nails to be provided for ledger strips.

Assembly Action: **None**

Final Hearing Results

S74-06/07 **AS**

Code Change No: S75-06/07

Original Proposal

Table 2304.9.1; IRC Table R602.3(1)

Proponent: Louis Wagner, American Fiberboard Association

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.

PART I – IBC

Revise as follows:

TABLE 2304.9.1
FASTENING SCHEDULE

i. Corrosion resistant staples with nominal 7/16-inch crown or 1-inch crown and ~~1 ⅛~~ 1¼-inch length for ½-inch sheathing and 1 ½-inch length for 25/32-inch sheathing. Panel supports at 16 inches (20 inches if strength axis in the long direction of the panel, unless otherwise marked).

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

PART II – IRC

Revise as follows:

TABLE R602.3(1)
FASTENER SCHEDULE FOR STRUCTURAL MEMBERS

DESCRIPTION OF BUILDING MATERIALS	DESCRIPTION OF FASTENER
½" structural cellulosic fiberboard sheathing	1 ½" galvanized roofing nail, ~~8d common (2 ½" X 0.131") nail;~~ 7/16" crown or 1" crown staple 16ga., ~~1 ½~~ 1 ¼" long
25/32" structural cellulosic fiberboard sheathing	1 3/4" galvanized roofing nail, ~~8d common (2 ½" X 0.131") nail;~~ 7/16" crown or 1" crown staple 16ga., ~~1 ¾~~ 1 ½" long

(Portions of table not shown do not change)

Reason: (IBC) This change introduces new information on the use of staples with fiberboard structural sheathing and along with changes to Table 2304.4.4 and IRC Table R602.3(1) will make reference to use of staples with fiberboard consistent within the two codes.

Crown size makes a difference in shear capacity of fiberboard sheathing. See Report number PFS 96-60 available at www.fiberboard.org and being used to revise Table 2304.4.4. The 1 1/8-inch staple is no longer readily available at job sites and should be replaced 1 1/4"-inch staples.

(IRC) This change introduces new information on the use of staples with fiberboard structural sheathing and along with changes to IBC Tables 2304.1 and 2304.4.4 will make reference to fastening of fiberboard consistent within the two codes.

Crown size makes a difference in shear capacity of fiberboard sheathing. See Report number PFS 96-60 available at www.fiberboard.org and being used to revise Table 2304.4.4. 8p common nails are no longer recommended for use with structural sheathing. Staple leg lengths can be shortened based on available proprietary ASTM E72 tests and fastener withdrawal calculations.

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

Public Hearing Results

PART I — IBC
Committee Action: **Approved as Submitted**

Committee Reason: This code change revises requirements for the attachment of fiberboard sheathing using staples that makes the code more consistent with current industry practice.

Assembly Action: **None**

PART II — IRC
Committee Action: **Approved as Submitted**

Committee Reason: This change updates the code to the current best practice for fastening structural fiberboard sheathing.

Assembly Action: **None**

Final Hearing Results

S75-06/07, Part I	**AS**
S75-06/07, Part II	**AS**

Code Change No: S76-06/07

Original Proposal

Sections: 2304.9.5; IRC R319.3

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.

PART I – IBC

Proponent: Joseph Holland, Hoover Treated Wood Products, Inc.

Delete and substitute as follows:

~~**2304.9.5 Fasteners in preservative-treated and fire-retardant-treated wood.** Fasteners for preservative-treated and fire-retardant-treated wood shall be of hot-dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper. The coating weights for zinc-coated fasteners shall be in accordance with ASTM A 153.~~

~~**Exception:** Fasteners other than nails, timber rivets, wood screws and lag screws shall be permitted to be of mechanically deposited zinc coated steel with coating weights in accordance with ASTM B 695, Class 55 minimum.~~

~~Fastenings for wood foundations shall be as required in AF&PA Technical Report No. 7.~~

<u>**2304.9.5 Fasteners in preservative-treated and fire-retardant-treated wood.** Fasteners in contact with preservative-treated wood and fire-retardant-treated wood shall be in accordance with this section.</u>

<u>**2304.9.5.1 Fasteners for preservative treated wood.** Fasteners for preservative-treated wood shall be of hot-dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper.</u>

<u>**Exceptions:**</u>

1. <u>One-half-inch (12.7 mm) diameter or greater steel bolts.</u>
2. <u>Fasteners other than nails, timber rivets, wood screws and lag screws shall be permitted to be of mechanically deposited zinc coated steel with coating weights in accordance with ASTM B 695, Class 55 minimum.</u>

<u>**2304.9.5.2 Fastenings for wood foundations.** Fastenings for wood foundations shall be as required in AF&PA Technical Report No. 7.</u>

<u>**2304.9.5.3 Fasteners for fire-retardant-treated wood used in exterior applications or wet or damp locations.** Fasteners for fire-retardant-treated wood used in exterior applications or wet or damp locations shall be of hot-dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper.</u>

2304.9.5.4 Fasteners for fire-retardant-treated wood used in interior applications. Fasteners for fire-retardant-treated wood used in interior locations shall be in accordance with the manufacturer's recommendations. In the absence of manufacturer's recommendations Section 2304.9.5.3 shall apply.

PART II – IRC

Delete and substitute as follows:

~~**R319.3 Fasteners.** Fasteners for pressure preservative and fire-retardant-treated wood shall be of hot-dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper. The coating weights for zinc-coated fasteners shall be in accordance with ASTM A 153.~~

~~**Exceptions:**~~

1. ~~One-half-inch (12.7mm) diameter or greater steel bolts.~~
2. ~~Fasteners other than nails, timber rivets, wood screws and lag screws shall be permitted to be of mechanically deposited zinc coated steel with coating weights in accordance with ASTM B 695, Class 55 minimum.~~

~~Fastenings for wood foundations shall be as required in AF&PA Technical Report No. 7.~~

R319.3 Fasteners in preservative-treated and fire-retardant-treated wood. Fasteners in contact with preservative-treated wood and fire-retardant-treated wood shall be in accordance with this section.

R319.3.1 Fasteners for Preservative Treated Wood. Fasteners for preservative-treated wood shall be of hot-dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper.

Exceptions:

1. One-half-inch (12.7 mm) diameter or greater steel bolts.
2. Fasteners other than nails, timber rivets, wood screws and lag screws shall be permitted to be of mechanically deposited zinc coated steel with coating weights in accordance with ASTM B 695, Class 55 minimum.

R319.3.2 Fastenings for wood foundations. Fastenings for wood foundations shall be as required in AF&PA Technical Report No. 7.

R319.3.3 Fasteners for fire-retardant-treated wood used in exterior applications or wet or damp locations. Fasteners for fire-retardant-treated wood used in exterior applications or wet or damp locations shall be of hot-dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper.

R319.3.4 Fasteners for fire-retardant-treated wood used in interior applications. Fasteners for fire-retardant-treated wood used in interior locations shall be in accordance with the manufacturer's recommendations. In the absence of manufacturer's recommendations Section R 319.3.3 shall apply.

Reason: 1. Bring in the exception for ½ bolts from the IRC into the IBC. 2. Recognize the different exposures for fire-retardant-treated wood and the fastener requirements for the exposure.

The interior exposure for FRTW is far less severe than the exposure for FRTW in wet, damp, or exterior locations. Until this year manufactures of FRTW used the code compliance report (BOCA, ICBO, NER, and SBCCI) to make their recommendations for the appropriate fastener for FRTW. With the consolidation of the code groups and the introduction of the ICC-ES system that is no longer allowed. The ICC-ES's position is the code over rules any testing a manufacturer has done for determining the appropriate fastener for FRTW.
Substantiation: FRTW for interior uses have not experienced problems with corrosion of fasteners used in contact with the wood. The manufacturers have satisfactorily used their recommendations for fasteners for more than 25 years. This change eliminates confusion between the code and the recommendations of the manufacturer.

Cost Impact: The code change proposal will not increase the cost of construction it will reduce the cost and allow the use of an appropriate fastener for the application.

Public Hearing Results

PART I — IBC
Committee Action: **Disapproved**

Committee Reason: This code change was disapproved because the proposed splitting of fire-retardant-treated wood requirements into interior and exterior was not substantiated.

Assembly Action: **None**

PART II — IRC
Committee Action: **Approved as Modified**

Modify the proposal as follows:

R319.3 Fasteners in preservative-treated and fire-retardant-treated wood. Fasteners in contact with preservative-treated wood and fire-retardant-treated wood shall be in accordance with this section. <u>The coating weights for zinc-coated fasteners shall be in accordance with ASTM A 153.</u>

(Portions of proposal not shown remain unchanged)

Committee Reason: This new language provides clarity to the code user on fasteners and their application when utilizing fire-retardant treated wood. The modification provides a needed reference to ASTM A 153 which was deleted in the original code change.

Assembly Action: **None**

Public Comments

Individual Consideration Agenda

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

Public Comment 1:

Joseph Holland, Hoover Treated Wood Products, requests Approval as Modified by this public comment for Part I.

Modify proposal as follows:

2304.9.5 Fasteners in preservative-treated and fire-retardant-treated wood. Fasteners for preservative treated and fire-retardant-treated wood shall be <u>in accordance with Section 2304.9.5.1 through 2304.9.5.4. 4</u> ~~of hot-dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper.~~ The coating weights for zinc-coated fasteners shall be in accordance with ASTM A 153.

> ~~**Exception:** Fasteners other than nails, timber rivets, wood screws and lag screws shall be permitted to be of mechanically deposited zinc coated steel with coating weights in accordance with ASTM B695, Class 55 minimum.f~~

~~Fastenings for wood foundations shall be as required in AF&PA Technical Report No. 7.~~

<u>**2304.9.5.1 Fasteners for preservative treated wood.** Fasteners for preservative-treated wood shall be of hot-dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper. Fasteners other than nails, timber rivets, wood screws and lag screws shall be permitted to be of mechanically deposited zinc coated steel with coating weights in accordance with ASTM B695, Class 55 minimum.</u>

<u>**2304.9.5.2 Fastenings for wood foundations.** Fastenings for wood foundations shall be as required in AF&PA Technical Report No. 7.</u>

<u>**2304.9.5.3 Fasteners for fire-retardant-treated wood used in exterior applications or wet or damp locations.** Fasteners for fire-retardant-treated wood used in exterior applications or wet or damp locations shall be of hot-dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper. Fasteners other than nails, timber rivets, wood screws and lag screws shall be permitted to be of mechanically deposited zinc coated steel with coating weights in accordance with ASTM B695, Class 55 minimum.</u>

<u>**2304.9.5.4 Fasteners for fire-retardant-treated wood used in interior applications.** Fasteners for fire-retardant-treated wood used in interior locations shall be in accordance with the manufacturer's recommendations. In the absence of manufacturer's recommendations Section 2304.9.5.3 shall apply.</u>

Commenter=s Reason: The original submission was taken from the 2003 IBC and the exception in the 2003 IRC. There were changes made to the 2006 editions not incorporated into the submission. This modification incorporates the 2006 language and drops the exception from the IRC.
In addition section 2304.9.5.3 incorporates the language from the exception for timber rivets, wood screws and lag screws for FRTW used in wet or damp locations and in exterior exposures.
This modification will recognize the fasteners being used for FRTW for more than 25 years.

Final Hearing Results

S76-06/07, Part I	**AMPC1**
S76-06/07, Part II	**AM**

Code Change No: S81-06/07

Original Proposal

Sections: 2304.11.2.5; IRC R319.1

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: Dennis Pitts, American Forest & Paper Association

PART I – IBC

Revise as follows:

2304.11.2.5 Wood siding. Clearance between wood siding and earth on the exterior of a building shall not be less than 6 inches (152 mm) or less than 2 inches(51 mm) from concrete steps, porch slabs, patio slabs, and similar horizontal surfaces exposed to the weather except where siding, sheathing, and wall framing are of naturally durable or preservative-treated wood.

PART II – IRC

Revise as follows:

R319.1 Location required. Protection from decay shall be provided in the following locations by the use of naturally durable wood or wood that is preservative treated in accordance with AWPA U1 for the species, product, preservative and end use. Preservatives shall be listed in Section 4 of AWPA U1.

1. Wood joists or the bottom of a wood structural floor when closer than 18 inches (457 mm) or wood girders when closer than 12 inches (305 mm) to the exposed ground in crawl spaces or unexcavated area located within the periphery of the building foundation.
2. All wood framing members that rest on concrete or masonry exterior foundation walls and are less than 8 inches (203 mm) from the exposed ground.
3. Sills and sleepers on a concrete or masonry slab that is in direct contact with the ground unless separated from such slab by an impervious moisture barrier.
4. The ends of wood girders entering exterior masonry or concrete walls having clearances of less than 0.5 inch (12.7 mm) on tops, sides and ends.
5. Wood siding, sheathing and wall framing on the exterior of a building having a clearance of less than 6 inches (152 mm) from the ground or less than 2 inches (51 mm) from concrete steps, porch slabs, patio slabs, and similar horizontal surfaces exposed to the weather.

No change to items 6 and 7.

Reason: The existing text should result in wood materials being at least 2" from the surface of typical 4"-thick concrete walks and porch slabs if the required minimum of 6" distance from the ground is maintained. In practice, however, it's not unusual to see less than 2" clearance. Without a minimum clearance water that may collect on the concrete can result in decay in the wood. Additionally, sufficient clearance to check for termite tubes may not be maintained.

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

Public Hearing Results

PART I — IBC
Committee Action: **Approved as Modified**

Modify the proposal as follows:

2304.11.2.5 Wood siding. Clearance between wood siding and earth on the exterior of a building shall not be less than 6 inches (152 mm) or less than 2 inches (51 mm) vertical from concrete steps, porch slabs, patio slabs, and similar horizontal surfaces exposed to the weather except where siding, sheathing, and wall framing are of naturally durable or preservative-treated wood.

Committee Reason: The proposal will minimize the exposure of wood siding to decay by elaborating on the minimum clearance requirements at horizontal concrete surfaces. The modification is intended to clarify that the clearance is measured vertically.

Assembly Action: **None**

PART II — IRC
Committee Action: **Approved as Modified**

Modify the proposal as follows:

R319.1 Location required. Protection from decay shall be provided in the following locations by the use of naturally durable wood or wood that is preservative treated in a accordance with AWPA U1 for the species, product, preservative and end use. Preservatives shall be listed in Section 4 of AWPA U1.

1. through 4. (No change to current text)
5. Wood siding, sheathing and wall framing on the exterior of a building having a clearance of less than 6 inches (152 mm) from the ground or less than 2 inches (51 mm) <u>measured vertically</u> from concrete steps, porch slabs, patio slabs and similar horizontal surfaces exposed to the weather.

Committee Reason: This additional language helps provide a usable measure criteria for material clearances. Without a minimum clearance water that collects on the concrete can result in decay of the wood. The modification specifically calls out that the measurement must be made vertically from concrete steps.

Assembly Action: **None**

Final Hearing Results

S81-06/07, Part I	**AM**
S81-06/07, Part II	**AM**

Code Change No: S82-06/07

Original Proposal

Sections: 2305, 1613.6.1, Table 2306.4.5

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

Revise as follows:

SECTION 2305
GENERAL DESIGN REQUIREMENTS FOR LATERAL-FORCE-RESISTING SYSTEMS

2305.1 General. Structures using wood shear walls and diaphragms to resist wind, seismic and other lateral loads shall be designed and constructed in accordance with <u>AF&PA SDPWS and the provisions of Section 2305, 2306, and 2307.</u> ~~the provisions of this section. Alternatively, compliance with the AF&PA SDPWS shall be permitted subject to the limitations therein and the limitations of this code.~~

~~**2305.1.1 Shear resistance based on principles of mechanics.** Shear resistance of diaphragms and shear walls are permitted to be calculated by principles of mechanics using values of fastener strength and sheathing shear resistance.~~

~~**2305.1.2 Framing.** Boundary elements shall be provided to transmit tension and compression forces. Perimeter members at openings shall be provided and shall be detailed to distribute the shearing stresses. Diaphragm and shear wall sheathing shall not be used to splice boundary elements. Diaphragm chords and collectors shall be placed in, or tangent to, the plane of the diaphragm framing unless it can be demonstrated that the moments, shears and deformations, considering eccentricities resulting from other configurations can be tolerated without exceeding the adjusted resistance and drift limits.~~

~~**2305.1.2.1 Framing members.** Framing members shall be at least 2 inch (51 mm) nominal width. In general, adjoining panel edges shall bear and be attached to the framing members and butt along their centerlines. Nails shall be placed not less than 3/8 inch (9.5 mm) from the panel edge, not more than 12 inches (305 mm) apart along intermediate supports, and 6 inches (152 mm) along panel edge bearings, and shall be firmly driven into the framing members.~~

~~**2305.1.3**~~ **2305.1.1 Openings in shear panels.** Openings in shear panels that materially affect their strength shall be fully detailed on the plans, and shall have their edges adequately reinforced to transfer all shearing stresses.

~~**2305.1.4 Shear panel connections.** Positive connections and anchorages capable of resisting the design forces shall be provided between the shear panel and the attached components. In Seismic Design Category D, E or F, the capacity of toenail connections shall not be used when calculating lateral load resistance to transfer lateral earthquake forces in excess of 150 pounds per foot (2189 N/m) from diaphragms to shear walls, drag struts (collectors) or other elements, or from shear walls to other elements.~~

~~**2305.1.5 Wood members resisting horizontal seismic forces contributed by masonry and concrete walls.** Wood shear walls, diaphragms, horizontal trusses and other members shall not be used to resist horizontal seismic forces contributed by masonry or concrete walls in structures over one story in height.~~

~~**Exceptions:**~~

1. ~~Wood floor and roof members are permitted to be used in horizontal trusses and diaphragms to resist horizontal seismic forces contributed by masonry or concrete walls, provided such forces do not result in torsional force distribution through the truss or diaphragm.~~
2. ~~Wood structural panel sheathed shear walls are permitted to be used to provide resistance to seismic forces contributed by masonry or concrete walls in two-story structures of masonry or concrete walls, provided the following requirements are met:~~
   - 2.1. ~~Story-to-story wall heights shall not exceed 12 feet (3658 mm).~~
   - 2.2. ~~Diaphragms shall not be designed to transmit lateral forces by rotation and shall not cantilever past the outermost supporting shear wall.~~
   - 2.3. ~~Combined deflections of diaphragms and shear walls shall not permit story drift of supported masonry or concrete walls to exceed the limit of Section 12.12.1 in ASCE 7.~~
   - 2.4. ~~Wood structural panel sheathing in diaphragms shall have unsupported edges blocked. Wood structural panel sheathing for both stories of shear walls shall have **2** unsupported edges blocked and, for the lower story, shall have a minimum thickness of 15/32 inch (11.9 mm).~~
   - 2.5. ~~There shall be no out-of-plane horizontal offsets between the first and second stories of wood structural panel shear walls.~~

~~**2305.1.6 Wood members resisting seismic forces from nonstructural concrete or masonry.** Wood members shall be permitted to resist horizontal seismic forces from nonstructural concrete, masonry veneer or concrete floors.~~

~~**2305.2 Design of wood diaphragms.**~~

~~**2305.2.1 General.** Wood diaphragms are permitted to be used to resist horizontal forces provided the deflection in the plane of the diaphragm, as determined by calculations, tests or analogies drawn therefrom, does not exceed the permissible deflection of attached distributing or resisting elements. Connections shall extend into the diaphragm a sufficient distance to develop the force transferred into the diaphragm.~~

~~**2305.2.2**~~ **2305.2 Diaphragm Deflection.** ~~Permissible deflection shall be that deflection up to which the diaphragm and any attached distributing or resisting element will maintain its structural integrity under design load conditions, such that the resisting element will continue to support design loads without danger to occupants of the structure. Calculations for diaphragm deflection shall account for the usual bending and shear components as well as any other factors, such as nail deformation, which will contribute to deflection.~~ The deflection (Δ) of a blocked wood structural panel diaphragm uniformly ~~nailed~~ fastened throughout is permitted to be calculated by using the following equation. If not uniformly ~~nailed~~ fastened, the constant 0.188 (For SI: 1/1627) in the third term must be modified accordingly.

$$\Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\sum(\Delta_c X)}{2b} \qquad \textbf{(Equation 23-1)}$$

For SI: $$\Delta = \frac{0.052vL^3}{EAb} + \frac{vL}{4Gt} + \frac{Le_n}{1627} + \frac{\sum(\Delta_c X)}{2b}$$

Where:
A = Area of chord cross section, in square inches (mm^2).
B = Diaphragm width, in feet (mm).
E = Elastic modulus of chords, in pounds per square inch (N/mm^2).
e_n = ~~Nail or staple~~ Staple deformation, in inches (mm) [see Table 2305.2.2(1)].
Gt = Panel rigidity through the thickness, in pounds per inch (N/mm) of panel width or depth [see Table 2305.2.2(2)].
L = Diaphragm length, in feet (mm).
v = Maximum shear due to design loads in the direction under consideration, in pounds per linear foot (plf) (N/mm).
Δ = The calculated deflection, in inches (mm).
$\Sigma(\Delta_c X)$ = Sum of individual chord-splice slip values on both sides of the diaphragm, each multiplied by its distance to the nearest support.

TABLE ~~2305.2.2(1)~~ 2305.2(1)
e_n VALUES (inches) FOR USE IN CALCULATING DIAPHRAGM DEFLECTION DUE TO FASTENER SLIP (Structural I)[ad]

LOAD PER FASTENER[c] (pounds)	FASTENER DESIGNATIONS[~~b~~]			
	~~6d~~	~~8d~~	~~10d~~	14-Ga staple x 2 inches long
60	~~0.01~~	~~0.00~~	~~0.00~~	0.011
80	~~0.02~~	~~0.01~~	~~0.01~~	0.018
100	~~0.03~~	~~0.01~~	~~0.01~~	0.028
120	~~0.04~~	~~0.02~~	~~0.01~~	0.04
140	~~0.06~~	~~0.03~~	~~0.02~~	0.053
160	~~0.10~~	~~0.04~~	~~0.02~~	0.068
180	~~-~~	~~0.05~~	~~0.03~~	-
200	-	~~0.07~~	~~0.47~~	-
220	-	~~0.09~~	~~0.06~~	-
240	-	-	~~0.07~~	-

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound = 4.448 N.

a. Increase e_n values 20 percent for plywood grades other than Structural I.
~~b. Nail values apply to common wire nails or staples identified.~~
c. Load per fastener = maximum shear per foot divided by the number of fasteners per foot at interior panel edges.
d. Decrease e_n values 50 percent for seasoned lumber (moisture content < 19 percent).

TABLE ~~2305.2.2(2)~~ 2305.2(2)
VALUES OF Gt FOR USE IN CALCULATING DEFLECTION OF WOOD STRUCTURAL PANEL SHEAR WALLS AND DIAPHRAGMS

(No change to table entries)

~~**2305.2.3 Diaphragm aspect ratios.** Size and shape of diaphragms shall be limited as set forth in Table 2305.2.3.~~

~~**TABLE 2305.2.3**~~
~~**MAXIMUM DIAPHRAGM DIMENSION RATIOS**~~
~~**HORIZONTAL AND SLOPED DIAPHRAGM**~~

~~**2305.2.4 Construction.** Wood diaphragms shall be constructed of wood structural panels manufactured with exterior glue and not less than 4 feet by 8 feet (1219mmby 2438 mm), except at boundaries and changes in framing where minimum sheet dimension shall be 24 inches (610 mm) unless all edges of the undersized sheets are supported by and fastened to framing members or blocking. Wood structural panel thickness for horizontal diaphragms shall not be less than the valves set forth in Tables 2304.7(3), 2304.7(4) and 2304.7(5) for corresponding joist spacing and loads.~~
~~**2305.2.4.1 Seismic Design Category F.** Structures assigned to Seismic Design Category F shall conform to the additional requirements of this section. Wood structural panel sheathing used for diaphragms and shear walls that are part of the seismic-force-resisting system shall be applied directly to the framing members.~~

> ~~**Exception:** Wood structural panel sheathing in a diaphragm is permitted to be fastened over solid lumber planking or laminated decking, provided the panel joints and lumber planking or laminated decking joints do not coincide.~~

~~**2305.2.5 Rigid diaphragms.** Design of structures with rigid diaphragms shall conform to the structure configuration requirements of Section 12.3.2 of ASCE 7 and the horizontal shear distribution requirements of Section 12.8.4 of ASCE 7. Open-front structures with rigid wood diaphragms resulting in torsional force distribution are permitted, provided the length, *l*, of the diaphragm normal to the open side does not exceed 25 feet (7620 mm), the diaphragm sheathing conforms to Section 2305.2.4 and the *l/w* ratio [as shown in Figure Figure 2305.2.5(1)] is less than 1 for one-story structures or 0.67 for structures over one story in height. **Exception:** Where calculations show that diaphragm deflections can be tolerated, the length, *l*, normal to the open end is permitted to be increased to a l/w ratio not greater than 1.5 where sheathed in compliance with Section 2305.2.4 or to 1 where sheathed in compliance with Section 2306.3.4 or 2306.3.5.~~

~~Rigid wood diaphragms are permitted to cantilever past the outermost supporting shearwall (or other vertical resisting element) a length, *l*, of not more than 25 feet (7620 mm) or two-thirds of the diaphragm width, *w*, whichever is smaller. Figure 2305.2.5(2) illustrates the dimensions of *l* and *w* for a cantilevered diaphragm.~~

~~Structures with rigid wood diaphragms having a torsional irregularity in accordance with Table 12.3-1, Item 1, of ASCE 7 shall meet the following requirements: the *l/w* ratio shall not exceed 1 for one-story structures or 0.67 for structures over one story in height, where *l* is the dimension parallel to the load direction for which the irregularity exists.~~

> ~~**Exception:** Where calculations demonstrate that the diaphragm deflections can be tolerated, the width is permitted to be increased and the *l/w* ratio is permitted to be increased to 1.5 where sheathed in compliance with Section 2305.2.4 or 1 where sheathed in compliance with Section 2306.3.4 or 2306.3.5.~~

~~**FIGURE 2305.2.5(1)**~~
~~**DIAPHRAGM LENGTH AND WIDTH FOR PLAN VIEW OF OPEN FRONT BUILDING**~~

~~**FIGURE 2305.2.5(2)**~~
~~**DIAPHRAGM LENGTH AND WIDTH FOR PLAN VIEW OF CANTILEVERED DIAPHRAGM**~~

~~**2305.3 Design of wood shear walls.**~~

~~**2305.3.1 General.** Wood shear walls are permitted to resist horizontal forces in vertical distributing or resisting elements, provided the deflection in the plane of the shear wall, as determined by calculations, tests or analogies drawn there from, does not exceed the more restrictive of the permissible deflection of attached distributing or resisting elements or the drift limits of Section 12.12.1 ofASCE7. Shear wall sheathing other than wood structural panels shall not be permitted in Seismic Design Category E or F (see Section 1613).~~

~~2305.3.2~~ <u>2305.3</u> <u>Shear wall</u> Deflection. ~~Permissible deflection shall be that deflection up to which the shear wall and any attached distributing or resisting element will maintain its structural integrity under design load conditions, i.e., continue to support design loads without danger to occupants of the structure.~~ The deflection (Δ) of a blocked wood structural panel shear wall uniformly fastened throughout is permitted to be calculated by the use of the following equation:

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b}$$ **(Equation 23-2)**

For SI: $$\Delta = \frac{vh^3}{3EAb} + \frac{vh}{Gt} + \frac{he_n}{407.6} + d_a\frac{h}{b}$$

where:

A = Area of boundary element cross section in square inches (mm^2) (vertical member at shear wall boundary).

b = Wall width, in feet (mm).

d_a = Vertical elongation of overturning anchorage (including fastener slip, device elongation, anchor rod elongation, etc.) at the design shear load (v).

E = Elastic modulus of boundary element (vertical member at shear wall boundary), in pounds per square inch (N/mm^2).

e_n = ~~Nail or staple~~ <u>Staple</u> deformation, in inches (mm) [see Table 2305.2.2(2)].

Gt = Panel rigidity through the thickness, in pounds per inch (N/mm) of panel width or depth [see Table 2305.2.2(2)].

H = Wall height, in feet (mm).
v = Maximum shear due to design loads at the top of the wall, in pounds per linear foot (N/mm).
Δ = The calculated deflection, in inches (mm).

~~**2305.3.3 Construction.** Wood shear walls shall be constructed of wood structural panels manufactured with exterior glue and not less than 4 feet by 8 feet (1219mmby 2438 mm), except at boundaries and at changes in framing. All edges of all panels shall be supported by and fastened to framing members or blocking.Wood structural panel thickness for shear walls shall not be less than set forth in Table 2304.6.1 for corresponding framing spacing and loads, except that 1/4 inch (6.4 mm) is permitted to be used where perpendicular loads permit.~~

~~**2305.3.4 Shear wall aspect ratios.** Size and shape of shear walls, perforated shear wall segments within perforated shear walls and wall piers within shear walls that are designed for force transfer around openings shall be limited as set forth in Table 2305.3.4. The height, *h*, and the width, *w*, shall be determined in accordance with Sections 2305.3.5 through 2305.3.5.2 and 2305.3.6 through 2305.3.6.2, respectively.~~

~~**TABLE 2305.3.4**~~
~~**MAXIMUM SHEAR WALL DIMENSION RATIOS**~~

~~**2305.3.5 Shear wall height definition.** The height of a shear wall, *h*, shall be defined as:~~

1. ~~The maximum clear height from the top of the foundation to the bottom of the diaphragm framing above; or~~
2. ~~The maximum clear height from the top of the diaphragm to the bottom of the diaphragm framing above [see Figure 2305.3.5(a)].~~

~~**2305.3.5.1 Perforated shear wall segment height definition.** The height of a perforated shear wall segment, *h*, shall be defined as specified in Section 2305.3.5 for shear walls.~~

~~**2305.3.5.2 Force transfer shear wall pier height definition.** The height, *h*, of a wall pier in a shear wall with openings designed for force transfer around openings shall be defined as the clear height of the pier at the side of an opening [see Figure 2305.3.5(b)].~~

~~**2305.3.6 Shear wall width definition.** The width of a shear wall, *w*, shall be defined as the sheathed dimension of the shear wall in the direction of application of force [see Figure 2305.3.5(a)].~~

~~**2305.3.6.1 Perforated shear wall segment width definition.** The width of a perforated shear wall segment, *w*, shall be defined as the width of full-height sheathing adjacent to openings in the perforated shear wall [see Figure 2305.3.5(a)].~~

~~**2305.3.6.2 Force transfer shear wall pier width definition.** The width, *w*, of a wall pier in a shear wall with openings designed for force transfer around openings shall be defined as the sheathed width of the pier at the side of an opening [see Figure 2305.3.5(b)].~~

~~**2305.3.7 Overturning restraint.** Where the dead load stabilizing moment in accordance with Chapter 16 allowable stress design load combinations is not sufficient to prevent uplift due to overturning moments on the wall, an anchoring device shall be provided. Anchoring devices shall maintain a continuous load path to the foundation.~~

~~**2305.3.8 Shear walls with openings.** The provisions of this section shall apply to the design of shear walls with openings. Where framing and connections around the openings are designed for force transfer around the openings, the provisions of Section 2305.3.8.1 shall apply. Where framing and connections around the openings are not designed for force transfer around the openings, the provisions of Section 2305.3.8.2 shall apply.~~

~~**2305.3.8.1 Force transfer around openings.** Where shear walls with openings are designed for force transfer around the openings, the limitations of Table 2305.3.4 shall apply to the overall shear wall, including openings, and to each wall pier at the side of an opening. Design for force transfer shall be based on a rational analysis. Detailing of boundary elements around the opening shall be provided in accordance with the provisions of this section [see Figure 2305.3.5(b)].~~

~~**2305.3.8.2 Perforated shear walls.** The provisions of Section 2305.3.8.2 shall be permitted to be used for the design of perforated shear walls. For the determination of the height and width of perforated shear wall segments, see Sections 2305.3.5.1 and 2305.3.6.1, respectively.~~

~~**2305.3.8.2.1 Limitations.** The following limitations shall apply to the use of Section 2305.3.8.2:~~

1. ~~A perforated shear wall segment shall be located at each end of a perforated shear wall. Openings shall be permitted to occur beyond the ends of the perforated shear wall, provided the width of such openings is not be included in the width of the perforated shear wall.~~
2. ~~The allowable shear set forth in Table 2306.4.1 shall not exceed 490 plf (7150 N/m).~~
3. ~~Where out-of-plane offsets occur, portions of the wall on each side of the offset shall be considered as separate perforated shear walls.~~
4. ~~Collectors for shear transfer shall be provided through the full length of the perforated shear wall.~~
5. ~~A perforated shear wall shall have uniform top of wall and bottom of wall elevations. Perforated shear walls not having uniform elevations shall be designed by other methods.~~
6. ~~Perforated shear wall height, *h*, shall not exceed 20 feet (6096 mm).~~

~~**2305.3.8.2.2 Perforated shear wall resistance.** The resistance of a perforated shear wall shall be calculated in accordance with the following:~~

1. ~~The percentage of full-height sheathing shall be calculated as the sum of the widths of perforated shear wall segments divided by the total width of the perforated shear wall, including openings.~~
2. ~~The maximum opening height shall be taken as the maximum opening clear height. Where areas above and below an opening remain unsheathed, the height of the opening shall be defined as the height of the wall.~~
3. ~~The unadjusted shear resistance shall be the allowable shear set forth in Table 2306.4.1 for height-to-width ratios of perforated shear wall segments that do not exceed 2:1 for seismic forces and 31/2:1 for other than seismic forces. For seismic forces, where the height-to-width ratio of any perforated shear wall segment used in the calculation of the sum of the widths of perforated shear wall segments, *∑Li*, is greater than 2:1 but does not exceed 31/2:1, the unadjusted shear resistance shall be multiplied by 2 *w/h*.~~
4. ~~The adjusted shear resistance shall be calculated by multiplying the unadjusted shear resistance by the shear resistance adjustment factors of Table 2305.3.8.2. For intermediate percentages of full-height sheathing, the values in Table 2305.3.8.2 are permitted to be interpolated.~~
5. ~~The perforated shear wall resistance shall be equal to the adjusted shear resistance times the sum of the widths of the perforated shear wall segments.~~

~~**2305.3.8.2.3 Anchorage and load path.** Design of perforated shear wall anchorage and load path shall conform to the requirements of Sections 2305.3.8.2.4 through 2305.3.8.2.8, or shall be calculated using principles of mechanics. Except as modified by these sections, wall framing, sheathing, sheathing attachment and fastener schedules shall conform to the requirements of Section 2305.2.4 and Table 2306.4.1.~~

~~**2305.3.8.2.4 Uplift anchorage at perforated shear wall ends.** Anchorage for uplift forces due to overturning shall be provided at each end of the perforated shear wall. The uplift anchorage shall conform to the requirements of Section 2305.3.7, except that for each story the minimum tension chord uplift force, *T*, shall be calculated in accordance with the following:~~

~~**(Equation 23-3)**~~

~~**TABLE 2305.3.8.2**~~
~~**SHEAR RESISTANCE ADJUSTMENT FACTOR, Co**~~
~~**WALL HEIGHT, H**~~

~~**FIGURE 2305.3.5**~~
~~**GENERAL DEFINITION OF SHEAR WALL HEIGHT, WIDTH AND HEIGHT-TO-WIDTH RATIO**~~

~~**2305.3.8.2.5 Anchorage for in-plane shear.** The unit shear force, *v*, transmitted into the top of a perforated shear wall, out of the base of the perforated shear wall at full height sheathing and into collectors connecting shear wall segments shall be calculated in accordance with the following:~~

~~**(Equation 23-4)**~~

~~**2305.3.8.2.6 Uplift anchorage between perforated shear wall ends.** In addition to the requirements of Section 2305.3.8.2.4, perforated shear wall bottom plates at full-height sheathing shall be anchored for a uniform uplift force, *t*, equal to the unit shear force, *v*, determined in Section 2305.3.8.2.5.~~

~~**2305.3.8.2.7 Compression chords.** Each end of each perforated shear wall segment shall be designed for acompression chord force, *C*, equal to the tension chord uplift force, *T*, calculated in Section 2305.3.8.2.4.~~

~~**2305.3.8.2.8 Load path.** Load path. A load path to the foundation shall be provided for each uplift force, T and t, for each shear force, V and v, and for each compression chord force, C. Elements resisting shearwall forces contributed by multiple stories shall be designed for the sum of forces contributed by each story.~~

~~**2305.3.8.2.9 Deflection of shear walls with openings.** The controlling deflection of a blocked shearwall with openings uniformly fastened throughout shall be taken as the maximum individual deflectionof the shear wall segments calculated in accordance with Section 2305.3.2, divided by the appropriateshear resistance adjustment factors of Table 2305.3.8.2.~~

~~**2305.3.9 Summing shear capacities.** The shear values for shear panels of different capacities applied to the same side of the wall are not cumulative except as allowed in Table 2306.4.1.~~

~~The shear values for material of the same type and capacity applied to both faces of the same wall are cumulative. Where the material capacities are not equal, the allowable shear shall be either two times the smaller shear capacity or the capacity of the stronger side, whichever is greater.~~

~~Summing shear capacities of dissimilar materials applied to opposite faces or to the same wall line is not allowed.~~

~~**Exception:** For wind design, the allowable shear capacity of shear wall segments sheathed with a combination of wood structural panels and gypsum wallboard on opposite faces, fiberboard structural sheathing and gypsum wallboard on opposite faces or hardboard panel siding and gypsum wallboard on opposite faces shall equal the sum of the sheathing capacities of each face separately.~~

~~**2305.3.10 Adhesives.** Adhesive attachment of shear wall sheathing is not permitted as a substitute for mechanical fasteners, and shall not be used in shear wall strength calculations alone, or in combination with mechanical fasteners in Seismic Design Category D, E or F.~~

~~**2305.3.11 Sill plate size and anchorage in Seismic Design Category D, E or F.** Anchor bolts for shear walls shall include steel plate washers, a minimum of 0.229 inch by 3 inches by 3 inches (5.82 mm by 76 mm by 76 mm) in size, between the sill plate and nut. The hole in the plate washer is permitted to be diagonally slotted with a width of up to 3/16 inch (4.76 mm) larger than the bolt diameter and a slot length not to exceed 13/4 inches (44 mm), provided a standard cut washer is placed between the plate washer and the nut. Sill plates resisting a design load greater than 490 plf (7154 N/m) using load and resistance factor design or 350 plf (5110 N/m) using allowable stress design shall not be less than a 3-inch (76 mm) nominal member. Where a single 3- inch (76 mm) nominal sill plate is used, 2- 20d box end nails shall be substituted for 2-16d common end nails found in line 8 of Table 2304.9.1.~~

~~**Exception:** In shear walls where the design load is greater than 490 plf (7151 N/m) but less than 840 plf (12264 N/m) using load and resistance factor design or greater than 350 plf (5110 N/m) but less than 600 plf (8760 N/m) using allowable stress design, the sill plate is permitted to be a 2-inch (51 mm) nominal member if the sill plate is anchored by two times the number of bolts required by design and 0.229-inch by 3-inch by 3-inch (5.82mm by 76mm by 76mm) plate washers are used.~~

1613.6.1 Assumption of flexible diaphragm. Add the following text at the end of Section 12.3.1.1 of ASCE 7: Diaphragms constructed of wood structural panels or untopped steel decking shall also be permitted to be idealized as flexible, provided all of the following conditions are met:

1. Toppings of concrete or similar materials are not placed over wood structural panel diaphragms except for nonstructural toppings no greater than 11/2 inches (38 mm) thick.
2. Each line of vertical elements of the lateral-force-resisting system complies with the allowable story drift of Table 12.12-1.
3. Vertical elements of the lateral-force-resisting system are light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets.
4. Portions of wood structural panel diaphragms that cantilever beyond the vertical elements of the lateral-force-resisting system are designed in accordance with Section ~~2305.2.5~~ 4.2.5.2 of AF & PA SDPWS ~~the International Building Code~~.

TABLE 2306.4.5
ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES FOR SHEAR WALLS OF LATH AND PLASTER OR GYPSUM BOARD WOOD FRAMED WALL ASSEMBLIES
(No change to table entries)

a. These shear walls shall not be used to resist loads imposed by masonry or concrete ~~construction (see Section 2305.1.5)~~ walls (see Section 4.1.5 of AF & PA SDPWS). Values shown are for short-term loading due to wind or seismic loading. Walls resisting seismic loads shall be subject to the limitations in Section 12.2.1 of ASCE 7. Values shown shall be reduced 25 percent for normal loading.
b. through k. (No change to current text)

Reason: Revision of Section 2305: Provisions being deleted from Section 2305 of the IBC are contained in *ANSI/AF&PA NDS Supplement* "*Special Design Provisions for Wind and Seismic" (SDPWS)* which is currently adopted by reference. These provisions are primarily for the building designer and duplication of the provisions not only is unnecessary, but duplication causes confusion. It is proper that all the design provisions be contained in a single document. Provisions of IBC-2006 Section 2305 are covered in the SDPWS-05 as shown in the following Table 2305.

Table 2305. Comparison of IBC-2006 Section 2305 and SDPWS-05

IBC-2006	SDPWS-05	Comment
2305.1.1	4.1.2	Same
2305.1.2	4.1.4	Same
2305.1.2.1	3.1.1, 4.2.7, 4.3.7	Same
2305.1.3	4.3.5	This sentence is retained because a specific requirement to detail on plans the reinforcing of holes in shear panels is not included in SDPWS. Requirements for force transfer for shear walls with openings are covered in SDPWS 4.3.5 and SDPWS includes general criteria by reference to NDS for ASD and LRFD which addresses effect of net section on design.
2305.1.4	4.1.7	Same
2305.1.5	4.1.5	Same
2305.1.6	4.1.6	Same
2305.2.1	4.2.1	Same
2305.2.2	4.2.2	Same in substance, however, SDPWS does not address deflection calculations for stapled diaphragms. Therefore, the diaphragm deflection equation and staple slip values are being retained. For nailed diaphragms, the SDPWS Simplified deflection equation has the same basis as Eq. 23-1. Use of Eq. 23-1 is permitted as an alternative and necessary equation inputs are provided in SDPWS Commentary. Stiffness properties for diaphragm construction other than wood structural panel are given in SDPWS for purposes of complying with drift and diaphragm flexibility requirements specified elsewhere in the building code.
2305.2.3	4.2.4	Same
Table 2305.2.3	Table 4.2.4	Same
2305.2.4	4.2.7	Same
2305.2.4.1	4.2.7.1	Same except attachment of sheathing directly to framing is generally required in SDPWS and not a special detail for SDC F. Expanded criteria are provided in SDPWS for wood structural panel over lumber decking.
2305.2.5	4.2.5	Same
2305.3.1	4.3.1	Same
2305.3.2	4.3.1, 4.3.2	Same in substance, however, SDPWS does not address deflection calculations for stapled shear walls. Therefore, the shear wall deflection equation and staple slip values are being retained. The SDPWS simplified deflection equation has the same basis as Eq. 23-2. Use of Eq. 23-2 is permitted as an alternative and necessary equation inputs are provided in SDPWS Commentary. Stiffness properties for shear wall construction other than wood structural panel are given in SDPWS for purposes of complying with drift and stiffness compatibility requirements specified elsewhere in the building code.
2305.3.3	4.3.7	Same
2305.3.4	4.3.4, 4.3.5	Same
Table 2305.3.4	Table 4.3.4	Same
2305.3.5	2.3	Same
2305.3.5.1	2.3	Same
2305.3.5.2	4.3.5.2	Same
2305.3.6	2.3	Same
2305.3.6.1	2.3	Same
2305.3.6.2	4.3.5.2	Same
2305.3.7	4.3.6.4.2	Same in substance except SDPWS language is applicable to designs in accordance with both ASD and LRFD methods.
2305.3.8	4.3.5	Same
2305.3.8.1	4.3.5.2	Same
2305.3.8.2	4.3.5.3	Same
2305.3.8.2.1	4.3.5.3	Same in substance except SDPWS language is applicable to designs in accordance with both ASD and LRFD methods. SDPWS language clarifies perforated shear wall sheathing limitations for one-sided and two-sided walls and for walls resisting wind and seismic.
2305.3.8.2.2	4.3.3.4, 4.3.4.1	Same
2305.3.8.2.3	4.3.6	Same
2305.3.8.2.4	4.3.6.1.2	Same
Table 2305.3.8.2	Table 4.3.2.1	Same
Figure 2305.3.5	Figure C4.3.5.1 and C4.3.5.2	Same
2305.3.8.2.5	4.3.6.4.1.1	Same
2305.3.8.2.6	4.3.6.4.2.1	Same
2305.3.8.2.7	4.3.6.1.2	Same
2305.3.8.2.8	4.3.6.4.4	Same
2305.3.8.2.9	4.3.2.1	Same in substance except SDPWS clarifies calculation method for perforated shear wall deflection.
2305.3.9	4.3.3.2,	Same in substance except SDPWS clarifies criteria for both ASD and LRFD methods. SDPWS also clarifies criteria for combination of materials on opposite sides of a two-sided wall for seismic. Currently, IBC states that they should not be summed.

2305.3.10	4.3.6.3.1	SDPWS limits use of adhesive shear wall systems to SDC A, B, and C and specifies R=1.5. In IBC, a reduced R is not specified for a system with adhesive.
2305.3.11	4.3.6.4.3	Same intent which is to minimize sill plate or bottom plate splitting; however, SDPWS specifies a minimum 2-1/2" square by ¼" washer for anchor bolts in all seismic design categories. To account for different bottom plate width and potential for cross-grain bending, SDPWS also requires the plate washer to extend to within ½" of the sheathed edge of the bottom plate. For SDC D, E and F only, IBC specifies 3x nominal sill plate with 3" square x 0.229" unless twice the number of anchor bolts are used. Where twice the number of anchor bolts are used, a 2x nominal sill plate is permitted provided the ASD design load is less than 600 plf.

Revision of Section 1613.6.1: The reference to Section 2305.2.5 of the IBC is replaced by reference to Section 4.2.5.2 of SDPWS containing the same limitations for cantilever diaphragms.

Revision of Table 2306.4.5 footnote a: The reference to Section 2305.1.5 of the IBC is replaced by reference to Section 4.1.5 of SDPWS containing the same limitations for wood members and systems resisting seismic forces contributed by masonry and concrete walls. The word "construction" is changed to "walls" to match language in both IBC and SDPWS.

Cost Impact: The cost change proposal will not increase the cost of construction. Provisions being deleted from Section 2305 of the IBC are contained in ANSI/AF&PA NDS Supplement "Special Design Provisions for Wind and Seismic" (SDPWS) which is currently adopted by reference.

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

SECTION 2305
GENERAL DESIGN REQUIREMENTS FOR
LATERAL-FORCE-RESISTING SYSTEMS

2305.1 General. Structures using wood shear walls and diaphragms to resist wind, seismic and other lateral loads shall be designed and constructed in accordance with AF&PA SDPWS and the provisions of Section 2305, 2306, and 2307.

2305.1.1 Openings in shear panels. Openings in shear panels that materially affect their strength shall be fully detailed on the plans, and shall have their edges adequately reinforced to transfer all shearing stresses.

2305.2 Diaphragm deflection. The deflection (Δ) of a blocked wood structural panel diaphragm uniformly fastened throughout with staples is permitted to be calculated by using the following equation. If not uniformly fastened, the constant 0.188 (For SI: 1/1627) in the third term ~~must~~ shall be modified accordingly.

$$\Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\Sigma(\Delta_c X)}{2b}$$

(Equation 23-1)

$$\Delta = \frac{.052vL^3}{EAb} + \frac{vL}{4Gt} + \frac{Le_n}{1627} + \frac{\Sigma(\Delta_c X)}{2b}$$

(Equation 23-2)

For SI:

Where:

A	=	Area of chord cross section, in square inches (mm^2).
b	=	Diaphragm width, in feet (mm).
E	=	Elastic modulus of chords, in pounds per square inch (N/mm^2).
e_n	=	Staple deformation, in inches (mm) [see Table ~~2305.2.2(1)~~ 2305.2(1)].
Gt	=	Panel rigidity through the thickness, in pounds per inch (N/mm) of panel width or depth [see Table ~~2305.2.2(2)~~ 2305.2(2)].
L	=	Diaphragm length, in feet (mm).
v	=	Maximum shear due to design loads in the direction under consideration, in pounds per linear foot (plf) (N/mm).
Δ	=	The calculated deflection, in inches (mm).
$\Sigma(\Delta_c X)$	=	Sum of individual chord-splice slip values on both sides of the diaphragm, each multiplied by its distance to the nearest support.

TABLE 2305.2(1)
e_n VALUES (inches) FOR USE IN CALCULATING DIAPHRAGM AND SHEAR WALL DEFLECTION DUE TO FASTENER SLIP (Structural I)[ac]

LOAD PER FASTENER[b] (pounds)	FASTENER DESIGNATIONS
	14-Ga staple x 2 inches long
60	0.011
80	0.018
100	0.028
120	0.04
140	0.053
160	0.068
180	-
200	-
220	-
240	-

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 pound = 4.448 N.
a. Increase e_n values 20 percent for plywood grades other than Structural I.
b. Load per fastener = maximum shear per foot divided by the number of fasteners per foot at interior panel edges.
c. Decrease e_n values 50 percent for seasoned lumber (moisture content < 19 percent).

TABLE 2305.2(2)
VALUES OF Gt FOR USE IN CALCULATING DEFLECTION OF WOOD STRUCTURAL PANEL SHEAR WALLS AND DIAPHRAGMS
(No change to table contents)

2305.3 Shear wall deflection. The deflection (Δ) of a blocked wood structural panel shear wall uniformly fastened throughout with staples is permitted to be calculated by the use of the following equation:

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b}$$

(Equation 23-2)

$$\Delta = \frac{vh^3}{3EAb} + \frac{vh}{Gt} + \frac{he_n}{407.6} + d_a\frac{h}{b}$$

For SI:

Where:

A = Area of boundary element cross section in square inches (mm^2) (vertical member at shear wall boundary).
b = Wall width, in feet (mm).
d_a = Vertical elongation of overturning anchorage (including fastener slip, device elongation, anchor rod elongation, etc.) at the design shear load (v).
E = Elastic modulus of boundary element (vertical member at shear wall boundary), in pounds per square inch (N/mm^2).
e_n = Staple deformation, in inches (mm) [see Table ~~2305.2.2(1)~~ 2305.2(1)].
Gt = Panel rigidity through the thickness, in pounds per inch (N/mm) of panel width or depth [see Table ~~2305.2.2(2)~~ 2305.2(2)].
h = Wall height, in feet (mm).
v = Maximum shear due to design loads at the top of the wall, in pounds per linear foot (N/mm).
Δ = The calculated deflection, in inches (mm).

1613.6.1 Assumption of flexible diaphragm. Add the following text at the end of Section 12.3.1.1 of ASCE 7: Diaphragms constructed of wood structural panels or untopped steel decking shall also be permitted to be idealized as flexible, provided all of the following conditions are met:

1. Toppings of concrete or similar materials are not placed over wood structural panel diaphragms except for nonstructural toppings no greater than 11/2 inches (38 mm) thick.
2. Each line of vertical elements of the lateral-force-resisting system complies with the allowable story drift of Table 12.12-1.
3. Vertical elements of the lateral-force-resisting system are light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets.
4. Portions of wood structural panel diaphragms that cantilever beyond the vertical elements of the lateral-force-resisting system are designed in accordance with Section 4.2.5.2 of AF & PA SDPWS.

TABLE 2306.4.5
ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES FOR SHEAR WALLS OF LATH AND PLASTER OR GYPSUM BOARD WOOD FRAMED WALL ASSEMBLIES

(No change to table contents)

a. These shear walls shall not be used to resist loads imposed by masonry or concrete walls (see Section 4.1.5 of AF & PA SDPWS). Values shown are for short-term loading due to wind or seismic loading. Walls resisting seismic loads shall be subject to the limitations in Section 12.2.1 of ASCE 7. Values shown shall be reduced 25 percent for normal loading.

b. through k. (No change to current text)

Committee Reason: This proposal substitutes a referenced standard for the provisions of Section 2305. The modification helps achieve the intent of the code change to retain IBC provisions pertaining to staple fasteners.

Assembly Action: **None**

Final Hearing Results

S82-06/07 **AM**

Code Change No: S83-06/07

Original Proposal

Section: 2306

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

Revise as follows:

SECTION 2306
ALLOWABLE STRESS DESIGN

2306.1 Allowable stress design. The structural analysis and construction of wood elements in structures using allowable stress design shall be in accordance with the following applicable standards:

American Forest & Paper Association.
NDS National Design Specification for Wood Construction
<u>SDPWS Special Design Provisions for Wind and Seismic</u>

American Institute of Timber Construction.
AITC 104 Typical Construction Details
AITC 110 Standard Appearance Grades for Structural Glued Laminated Timber
AITC 113 Standard for Dimensions of Structural Glued Laminated Timber
AITC 117 Standard Specifications for Structural Glued Laminated Timber of Softwood Species
AITC 119 Structural Standard Specifications for Glued Laminated Timber of Hardwood Species
AITC A190.1 Structural Glued Laminated Timber
AITC 200 Inspection Manual

American Society of Agricultural Engineers.
ASAE EP 484.2 Diaphragm Design of Metal -Clad, Post-Frame Rectangular Buildings
ASAE EP 486.1 Shallow Post Foundation Design
ASAE 559 Design Requirements and Bending Properties for Mechanically Laminated Columns

APA—The Engineered Wood Association.
Panel Design Specification
Plywood Design Specification Supplement 1 - Design & Fabrication of Plywood Curved Panel
Plywood Design Specification Supplement 2 - Design & Fabrication of Glued Plywood-Lumber Beams
Plywood Design Specification Supplement 3 - Design & Fabrication of Plywood Stressed-Skin Panels
Plywood Design Specification Supplement 4 - Design & Fabrication of Plywood Sandwich Panels
Plywood Design Specification Supplement 5 - Design & Fabrication of All-Plywood Beams
EWS T300 Glulam Connection Details
EWS S560 Field Notching and Drilling of Glued Laminated Timber Beams
EWS S475 Glued Laminated Beam Design Tables
EWS X450 Glulam in Residential Construction
EWS X440 Product and Application Guide: Glulam

EWS R540 Builders Tips: Proper Storage and Handling of Glulam Beams

Truss Plate Institute, Inc.
TPI 1 National Design Standard for Metal Plate Connected Wood Truss Construction

2306.1.1 Joists and rafters. The design of rafter spans is permitted to be in accordance with the *AF&PA Span Tables for Joists and Rafters*.

2306.1.2 Plank and beam flooring. The design of plank and beam flooring is permitted to be in accordance with the *AF&PA Wood Construction Data No. 4*.

2306.1.3 Treated wood stress adjustments. The allowable unit stresses for preservative-treated wood need no adjustment for treatment, but are subject to other adjustments. The allowable unit stresses for fire-retardant-treated wood, including fastener values, shall be developed from an approved method of investigation that considers the effects of anticipated temperature and humidity to which the fire-retardant-treated wood will be subjected, the type of treatment and the redrying process. Other adjustments are applicable except that the impact load duration shall not apply.

2306.1.4 Lumber decking. The capacity of lumber decking arranged according to the patterns described in Section 2304.8.2 shall be the lesser of the capacities determined for flexure and deflection according to the formulas in Table 2306.1.4.

TABLE 2306.1.4
ALLOWABLE LOADS FOR LUMBER DECKING
No change to table contents

~~**2306.2 Wind provisions for walls.**~~

~~**2306.2.1 Wall stud bending stress increase.** The AF&PA NDS fiber stress in bending (*Fb*) design values for sawn lumber wood studs resisting out of plane wind loads shall be increased by the factors in Table 2306.2.1, in lieu of the 1.15 repetitive member factor. These increases take into consideration the load sharing and composite actions provided by the wood structural panels as defined in Section 2302.1. The increases shall apply where the studs are designed for bending and are spaced no more than 16 inches (406 mm) o.c., covered on the inside with a minimum of 1/2-inch (12.7 mm) gypsum board fastened in accordance with Table 2306.4.5 and sheathed on the exterior with a minimum of 3/8-inch (9.5mm) wood structural panel sheathing. All panel joints shall occur over studs or blocking and shall be attached using a minimum of 8d common nails spaced a maximum of 6 inches o.c. (152 mm) at panel edges and 12 inches o.c. (305mm) at intermediate framing members.~~

~~**TABLE 2306.2.1**~~
~~**WALL STUD BENDING STRESS INCREASE FACTORS**~~

2306. 3 Wood diaphragms.

2306. 3.1 Wood structural panel diaphragms. Wood structural panel diaphragms shall be designed and constructed in accordance with AF&PA SDPWS. Wood structural panel diaphragms are permitted to resist horizontal forces using the allowable shear capacities set forth in Table 2306.3.1 or Table 2306.3.2. The allowable shear capacities in Table 2306.3.1 and Table 2306.3.2 are permitted to be increased 40 percent for wind design. ~~calculated by principles of mechanics without limitations by using values for fastener strength in the AF&PA NDS, structural design properties for wood structural panels based on DOC PS-1 and DOC PS-2 or wood structural panel design properties given in the *APA Panel Design Specification* (PDS).~~

TABLE 2306.3.1
RECOMMENDED SHEAR FOR WOOD STRUCTURAL PANEL DIAPHRAGMS WITH FRAMING OF DOUGLAS FIR LARCH OR SOUTHERN PINE FOR WIND AND SEISMIC LOADING.
(No change to table contents)

TABLE 2306.3.2
ALLOWABLE SHEAR IN POUNDS PER SUQRE FOOT FOR HORIZONTAL BLOCKED DIAPHRAGMS UTILIZING MULTIPLE ROWS OF FASTENERS WITH FRAMING OF DOUGLAS FIR LARCH OR SOUTHERN PINE FOR WIND OR SEISMIC LOADING.
(No change to table contents)

~~**2306.3.2 Shear capacities modifications.** The allowable shear capacities in Tables 2306.3.1 and 2306.3.2 for horizontal wood structural panel diaphragms shall be increased 40 percent for wind design.~~

~~**2306.3.3 Diagonally sheathed lumber diaphragms.** Diagonally sheathed lumber diaphragms shall be nailed in accordance with Table 2306.3.3.~~

~~**TABLE 2306.3.3**~~
~~**DIAGONALLY SHEATHED LUMBER DIAPHRAGM NAILING SCHEDULE**~~

2306. 3.4 Single diagonally sheathed lumber diaphragms. Single diagonally sheathed lumber diaphragms shall be designed and constructed in accordance with AF&PA SDPWS. ~~Single diagonally sheathed lumber diaphragms shall be constructed of minimum 1-inch (25 mm) thick nominal sheathing boards laid at an angle of approximately 45 degrees (0.78 rad) to the supports. The shear capacity for single diagonally sheathed lumber diaphragms of southern pine or Douglas fir-larch shall not exceed 300 plf (4378N/m) of width. The shear capacities shall be adjusted by reduction factors of 0.82 for framing members of species with a specific gravity equal to or greater than 0.42 but less than 0.49 and 0.65 for species with a specific gravity of less than 0.42, as contained in the AF&PA NDS.~~

~~**2306.3.4.1 End joints.** End joints in adjacent boards shall be separated by at least one stud or joist space and there shall be at least two boards between joints on the same support.~~

~~**2306.3.4.2 Single diagonally sheathed lumber diaphragms.** Single diagonally sheathed lumber diaphragms made up of 2-inch (51 mm) nominal diagonal lumber sheathing fastened with 16d nails shall be designed with the same shear capacities as shear panels using 1-inch (25 mm) boards fastened with 8d nails, provided there are not splices in adjacent boards on the same support and the supports are not less than 4 inch (102mm) nominal depth or 3 inch (76 mm) nominal thickness.~~

2306.3.5 Double diagonally sheathed lumber diaphragms. Double diagonally sheathed lumber diaphragms shall be designed and constructed in accordance with AF&PA SDPWS. ~~Double diagonally sheathed lumber diaphragms shall be constructed of two layers of diagonal sheathing boards at 90 degrees (1.57 rad) to each other on the same face of the supporting members. Each chord shall be considered as a beam with uniform load per foot equal to 50 percent of the unit shear due to diaphragm action. The load shall be assumed as acting normal to the chord in the plan of the diaphragm in either direction. The span of the chord or portion thereof shall be the distance between framing members of the diaphragm, such as the joists, studs and blocking that serve to transfer the assumed load to the sheathing. The shear capacity of double diagonally sheathed diaphragms of Southern pine or Douglas fir-larch shall not exceed 600 plf (8756 kN/m) of width. The shear capacity shall be adjusted by reduction factors of 0.82 for framing members of species with a specific gravity equal to or greater than 0.42 but less than 0.49 and 0.65 for species with a specific gravity of less than 0.42, as contained in the AF&PA NDS. Nailing of diagonally sheathed lumber diaphragms shall be in accordance with Table 2306.3.3.~~

2306.3.6 Gypsum board diaphragm ceilings. Gypsum board diaphragm ceilings shall be in accordance with Section 2508.5.

~~**2306.4 Shear walls.** Panel sheathing joints in shear walls shall occur over studs or blocking. Adjacent panel sheathing joints shall occur over and be nailed to common framing members (see Section 2305.3.1 for limitations on shear wall bracing materials).~~

2306.4.1 Wood structural panel shear walls. Wood structural panel shear walls shall be designed and constructed in accordance with AF&PA SDPWS. Wood structural panel shear walls are permitted to resist horizontal forces using ~~The~~ the allowable shear capacities set forth in ~~for wood structural panel shear walls shall be in accordance with~~ Table 2306.4.1. ~~These~~ Allowable capacities in Table 2306.4.1 are permitted to be increased 40 percent for wind design. ~~Shear walls are permitted to be calculated by principles of mechanics without limitations by using values for nail strength given in the AF&PA NDS and wood structural panel design properties given in the *APA Panel Design Specification*.~~

TABLE 2306.4.1
ALLOWABLE SHEAR (POUNDS PER FOOT) FOR WOOD STRUCTURAL PANEL SHEAR WALLS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE FOR WIND OR SEISMIC LOADING
(No change to table contents)

2306.4.2 Lumber sheathed shear walls. Single and double diagonally sheathed lumber ~~diaphragms~~ shear walls shall be designed and constructed in accordance with AF&PA SDPWS. ~~are permitted using the construction and allowable load provisions of Sections 2306.3.4 and 2306.3.5.~~ Single and double diagonally sheathed lumber walls shall not be used to resist seismic loads in structures in Seismic Design Category E or F.

2306.4.3 Particleboard shear walls. Particleboard shear walls shall be designed and constructed in accordance with AF&PA SDPWS. Particleboard shear walls shall be permitted to resist horizontal forces using ~~The design~~ the allowable shear ~~capacity of~~ capacities ~~particleboard shear walls shall be in accordance with~~ set forth in Table 2306.4.3. Allowable capacities in Table 2306.4.3 are permitted to be increased 40 percent for wind design. ~~Shear panels shall be constructed with particleboard sheets not less than 4 feet by 8 feet (1219 mm by 2438 mm), except at boundaries and changes in framing. Particleboard panels shall be designed to resist shear only, and chords, collector members and boundary elements shall be connected at all corners. Panel edges shall be backed with 2-inch (51 mm) nominal or wider framing. Sheets are permitted to be installed either horizontally or vertically. For 3/8-inch (9.5 mm) particleboard sheets installed with the long dimension parallel to the studs spaced 24 inches (610 mm) o.c, nails shall be spaced at 6 inches (152 mm) o.c. along intermediate framing members. For all other conditions, nails of the same size shall be spaced at 12 inches (305mm)o.c. along intermediate framing members. Particleboard panels less than 12 inches (305 mm) wide shall be blocked.~~ Particleboard shall not be used to resist seismic forces in structures in Seismic Design Category D, E or F.

TABLE 2306.4.3
ALLOWABLE SHEAR FOR PARTICLEBOARD SHEAR WALL SHEATHING
(No change to table contents)

2306.4.4 Fiberboard shear walls. Fiberboard shear walls shall be designed and constructed in accordance with AF&PA SDPWS. Fiberboard shear walls are permitted to resist horizontal forces using ~~The design~~ the allowable shear ~~capacity of~~ capacities ~~fiberboard shear walls shall be in accordance with~~ set forth in Table 2306.4.4. Allowable capacities in Table 2306.4.4 are permitted to be increased 40 percent for wind design. ~~The fiberboard sheathing shall be applied vertically or horizontally to wood studs not less than 2 inch (51 mm) in nominal thickness spaced 16 inches (406 mm) o.c. Blocking not less than 2 inch (51 mm) nominal in thickness shall be provided at horizontal joints.~~ Fiberboard shall not be used to resist seismic forces in structures in Seismic Design Category D, E or F.

TABLE 2306.4.4
ALLOWABLE SHEAR VALUES (plf) FOR WIND OR SEISMIC LOADING ON VERTICAL DIAPHRAGMS OF FIBERBOARD SHEATHING BOARD CONSTRUCTION FOR TYPE V CONSTRUCTION ONLY
(No change to table contents)

2306.4.5 Shear walls sheathed with other materials. Shear walls sheathed with portland cement plaster, gypsum lath, gypsum sheathing, or gypsum board shall be designed and constructed in accordance with AF&PA SDPWS. Shear walls sheathed with these materials are permitted to resist horizontal forces using the allowable ~~Shear~~ shear capacities ~~for walls sheathed with lath, plaster or gypsum board shall be in accordance with~~ set forth in Table 2306.4.5. ~~Shear walls sheathed with lath, plaster or gypsum board shall be constructed in accordance with Chapter 25 and Section 2306.4.5.1. Walls resisting seismic loads shall be subject to the limitations in Section 12.2.1 of ASCE 7.~~ Shear walls sheathed with portland cement plaster, gypsum lath, gypsum sheathing, or gypsum board shall not be used to resist seismic loads in structures in Seismic Design Category E or F.

TABLE 2306.4.5
ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES FOR SHEAR WALLS OF LATH AND PLASTER OR GYPSUM BOARD WOOD FRAMED WALL ASSEMBLIES
(No change to table contents)

~~**2306.4.5.1 Application of gypsum board or lath and plaster to wood framing.**~~
~~**2306.4.5.1.1 Joint staggering.** End joints of adjacent courses of gypsum board shall not occur over the same stud.~~

~~**2306.4.5.1.2 Blocking.** Where required in Table 2306.4.5, wood blocking having the same cross-sectional dimensions as the studs shall be provided at joints that are perpendicular to the studs.~~

~~**2306.4.5.1.3 Fastening.** Studs, top and bottom plates and blocking shall be fastened in accordance with Table 2304.9.1.~~

~~**2306.4.5.1.4 Fasteners.** The size and spacing of fasteners shall be set forth in Table 2306.4.5. Fasteners shall be spaced not less than 3/8 inch (9.5 mm) from edges and ends of gypsum boards or sides of studs, blocking and top and bottom plates.~~

~~**2306.4.5.1.5 Gypsum lath.** Gypsum lath shall be applied perpendicular to the studs. Maximum allowable shear values shall be as set forth in Table 2306.4.5.~~

~~**2306.4.5.1.6 Gypsum sheathing.** Four-foot-wide (1219 mm) pieces of gypsum sheathing shall be applied parallel or perpendicular to studs. Two-foot-wide (610 mm) pieces of gypsum sheathing shall be applied perpendicular to the studs. Maximum allowable shear values shall be as set forth in Table 2306.4.5.~~

~~**2306.4.5.1.7 Other gypsum boards.** Gypsum board shall be applied parallel or perpendicular to studs. Maximum allowable shear values shall be as set forth in Table 2306.4.5.~~

Reason: Provisions being deleted from Section 2306 of the IBC are also contained in the AF&PA *Special Design Provisions for Wind and* Seismic (AF&PA SDPWS) which is currently adopted by reference. Deleted provisions are primarily for the building designer and duplication of the provisions is not necessary and causes confusion. However; this proposed change retains tabulated values of ASD unit shear capacity for shear walls and diaphragms as the building code has been the primary source of this information for many years. ASD unit shear capacities for shear walls and diaphragms can also be obtained directly from the SDPWS-05. Over time, it is desired that all the design provisions, including tabulated ASD unit shear capacities, be obtained by reference to the SDPWS. Provisions of the IBC Section 2306 are covered in SDPWS-05 as shown in Table 2306.

Table 2306. Comparison of IBC Section 2306 and SDPWS-05

IBC Section 2306	SDPWS-05	Comment
2306.2.1	3.1.1.1	Same
Table 2306.2.1	Table 3.1.1.1	Same
2306.3.1	4.1.2	Same
2306.3.2	Table 4.2A-C	Same except increase for wind is incorporated in SDPWS design value tables.
2306.3.3	4.2.7.2, 4.2.7.3	Same
2306.3.4	4.2.7.2	Same except 40% increase is recognized for wind design consistent with SDPWS.
2306.3.4.1	4.2.7.2	Same
2306.3.4.2	4.2.7.2	Same
2306.3.5	4.2.7.3	Same except 40% increase is recognized for wind design consistent with SDPWS.
2306.4	4.3.7	Same
2306.4.1	4.1.2	Same
2306.4.3	4.3.7.2	Same except 40% increase is recognized for wind design consistent with SDPWS.
2306.4.4	4.3.7.3	Same except 40% increase ins recognized for wind design consistent with SDPWS.
2306.4.5.1	4.3.7.4	Same
2306.4.5.1.1	4.3.7.4	Same
2306.4.5.1.2	4.3.7.4	Same
2306.4.5.1.3	4.3.7.4	Same
2306.4.5.1.4	4.3.7.4	Same
2306.4.5.1.5	4.3.7.4.3	Same
2306.4.5.1.6	4.3.7.4.2	Same
2306.4.5.1.7	4.3.7.4	Same

With removal of duplicate information, it is suggested that remaining sections be numbered as follows:

SECTION 2306
ALLOWABLE STRESS DESIGN
2306.1 Allowable stress design.
2306.1.1 Joists and rafters.
2306.1.2 Plank and beam flooring.
2306.1.3 Treated wood stress adjustments.
2306.1.4 Lumber decking.
2306.2 Wood diaphragms.
2306.2.1 Wood structural panel diaphragms.
2306.2.2 Single diagonally sheathed lumber diaphragms.
2306.2.3 Double diagonally sheathed lumber diaphragms.
2306.2.4 Gypsum board diaphragm ceilings.
2306.3 Shear walls.
2306.3.1 Wood structural panel shear walls.
2306.3.2 Lumber sheathed shear walls.
2306.3.3 Particleboard shear walls.
2306.3.4 Fiberboard shear walls.
2306.3.5 Shear walls sheathed with other materials.

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: Relying on a referenced standard for the technical provisions for allowable stress design of wood is consistent with the action taken on S82-06/07.

Assembly Action: **None**

Final Hearing Results

S83-06/07 **AS**

Code Change No: S90-06/07

Original Proposal

Sections: 2305.3.11, 2308.6, 2308.12.8, 2308.12.9; IRC 403.1.6.1, R602.11.1

Proponent: Randall Shackelford, Simpson Strong-Tie Co.

PART I – IBC STRUCTURAL

Revise as follows:

2305.3.11 Sill plate size and anchorage in Seismic Design Category D, E or F. Shear wall sill plates shall be anchored with a ~~A~~nchor bolts ~~for shear walls shall include~~ with steel plate washers, between the sill plate and nut or with approved anchor straps load rated in accordance with section 1715.1 and spaced to provide equivalent anchorage. Steel plate washers shall be a minimum of 0.229 inch by 3 inches by 3 inches (5.82 mm by 76 mm by 76 mm) in size~~, between the sill plate and nut~~. The hole in the plate washer is permitted to be diagonally slotted with a width of up to 3/16 inch (4.76 mm) larger than the bolt diameter and a slot length not to exceed 13/4 inches (44 mm), provided a standard cut washer is placed between the plate washer and the nut. Sill plates resisting a design load greater than 490 plf (7154 N/m) using load and resistance factor design or 350 plf (5110 N/m) using allowable stress design shall not be less than a 3-inch (76 mm) nominal member. Where a single 3- inch (76 mm) nominal sill plate is used, 2- 20d box end nails shall be substituted for 2-16d common end nails found in line 8 of Table 2304.9.1.

> **Exception:** In shear walls where the design load is greater than 490 plf (7151 N/m) but less than 840 plf (12 264 N/m) using load and resistance factor design or greater than 350 plf (5110 N/m) but less than 600 plf (8760 N/m) using allowable stress design, the sill plate is permitted to be a 2-inch (51 mm) nominal member if the sill plate is anchored by two times the number of bolts or anchor straps required by design and 0.229-inch by 3-inch by 3-inch (5.82mmby 76mmby 76mm) plate washers are used.

2308.6 Foundation plates or sills. Foundations and footings shall be as specified in Chapter 18. Foundation plates or sills resting on concrete or masonry foundations shall comply with Section 2304.3.1. Foundation plates or sills shall be bolted or anchored to the foundation with not less than 1/2-inch-diameter (12.7 mm) steel bolts or approved anchors spaced to provide equivalent anchorage as the steel bolts. Bolts shall be embedded at least 7 inches (178 mm) into concrete or masonry, and spaced not more than 6 feet (1829 mm) apart. There shall be a minimum of two bolts or anchor straps per piece with one bolt or anchor strap located not more than 12 inches (305 mm) or less than 4 inches (102 mm) from each end of each piece. A properly sized nut and washer shall be tightened on each bolt to the plate.

2308.12.8 ~~Steel plate washers~~ Sill plate anchorage. Sill plates shall be anchored with anchor bolts with steel plate washers ~~shall be placed~~ between the foundation sill plate and the nut, or approved anchor straps load rated in accordance with Section 1715.1. Such washers shall be a minimum of 0.229 inch by 3 inches by 3 inches (5.82 mm by 76 mm by 76 mm) in size. The hole in the plate washer is permitted to be diagonally slotted with a width of up to 3/16 inch (4.76 mm) larger than the bolt diameter and a slot length not to exceed 1 ¾ inches (44 mm), provided a standard cut washer is placed between the plate washer and the nut.

2308.12.9 Sill plate anchorage in Seismic Design Category E. Steel bolts with a minimum nominal diameter of ⅝ inch (15.9 mm) or approved foundation anchor straps load rated in accordance with Section 1715.1 and spaced to provide equivalent anchorage shall be used in Seismic Design Category E.

PART II – IRC

Revise as follows:

R403.1.6.1 Foundation anchorage in Seismic Design Categories C, D_0, D_1 and D_2. In addition to the requirements of Section R403.1.6, the following requirements shall apply to wood light-frame structures in Seismic Design Categories D_0, D_1 and D_2 and wood light-frame townhouses in Seismic Design Category C.

1. Plate washers conforming to Section R602.11.1 shall be provided for all anchor bolts over the full length of required braced wall lines <u>except where approved anchor straps are used</u>. Properly sized cut washers shall be permitted for anchor bolts in wall lines not containing braced wall panels.
2. Interior braced wall plates shall have anchor bolts spaced at not more than 6 feet (1829 mm) on center and located within 12 inches (305 mm) of the ends of each plate section when supported on a continuous foundation.
3. Interior bearing wall sole plates shall have anchor bolts spaced at not more than 6 feet (1829 mm) on center and located within 12 inches (305 mm) of the ends of each plate section when supported on a continuous foundation.
4. The maximum anchor bolt spacing shall be 4 feet (1219 mm) for buildings over two stories in height.
5. Stepped cripple walls shall conform to Section R602.11.3.
6. Where continuous wood foundations in accordance with Section R404.2 are used, the force transfer shall have a capacity equal to or greater than the connections required by Section R602.11.1 or the braced wall panel shall be connected to the wood foundations in accordance with the braced wall panel-to-floor fastening requirements of Table R602.3(1).

R602.11.1 Wall anchorage. Braced wall line sills shall be anchored to concrete or masonry foundations in accordance with Sections R403.1.6 and R602.11. For all buildings in Seismic Design Categories D_0, D_1 and D_2 and townhouses in Seismic Design Category C, plate washers, a minimum of 0.229 inch by 3 inches by 3 inches (5.8mm by 76mm by 76 mm) in size, shall be installed between the foundation sill plate and the nut<u>, except where approved anchor straps are used</u>. The hole in the plate washer is permitted to be diagonally slotted with a width of up to 3/16 inch (5 mm) larger than the bolt diameter and a slot length not to exceed 1 3/4 inches (44 mm), provided a standard cut washer is placed between the plate washer and the nut.

Reason: (IBC) Revise the code to allow strap anchors in higher seismic regions, or clarify code that strap anchors are permitted in higher seismic regions, depending on how you look at it.

Recent cyclic testing of foundation anchor straps on long and short walls by Simpson Strong-Tie has shown that they perform very well under cyclic loading. This is partly because they wrap around the sill plate at the sheathing nailing location, thereby helping to prevent cross-grain bending of the sill plate. Since prevention of cross-grain bending is the primary reason for using plate washers, anchor straps can be substituted for anchor bolts with plate washers. However, most anchor straps are not a one-for-one substitution for anchor bolts, so it is necessary to add the wording “spaced as required to provide equivalent anchorage’. For shear walls, the designer will determine the required spacing based on the tested allowable load of the strap anchor. For conventional construction, builders can refer to manufacturer’s literature for equivalent spacing to anchor bolts.

(IRC) Some building officials have interpreted this section as prohibiting the use of anchor straps to anchor sill plates when the 3 by 3 washer is required. Recent cyclic testing of foundation anchor straps on long and short shear walls by Simpson Strong-Tie has shown that they perform very well under cyclic loading. This is partly because they wrap around the sill plate at the sheathing nailing location, thereby helping to prevent cross-grain bending of the sill plate. Since prevention of cross-grain bending is the primary reason for using plate washers, anchor straps can be substituted for anchor bolts with plate washers. Although it is true that most anchor straps are not a one-for-one substitution for anchor bolts, the IRC already contains the wording “spaced as required to provide equivalent anchorage to ½-inch diameter anchor bolts” in Section R403.1.6. Builders and building officials can refer to manufacturer’s literature for equivalent spacing to anchor bolts. This change limits the permission to “anchor straps” because that is what has been tested.

Cost Impact: The code change proposal will not increase the cost of construction. It will allow additional options.

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

~~**2305.3.11 Sill plate size and anchorage in Seismic Design Category D, E or F.** Shear wall sill plates shall be anchored with a Anchor bolts with steel plate washers, between the sill plate and nut or with approved anchor straps load rated in accordance with section 1715.1 and spaced to provide equivalent anchorage. Steel plate washers shall be a minimum of 0.229 inch by 3 inches by 3 inches (5.82 mm by 76 mm by 76 mm) in size. The hole in the plate washer is permitted to be diagonally slotted with a width of up to 3/16 inch (4.76 mm) larger than the bolt diameter and a slot length not to exceed 13/4 inches (44 mm), provided a standard cut washer is placed between the plate washer and the nut. Sill plates resisting a design load greater than 490 plf (7154 N/m) using load and resistance factor design or 350 plf (5110 N/m) using allowable stress design shall not be less than a 3-inch (76 mm) nominal member. Where a single 3- inch (76 mm) nominal sill plate is used, 2- 20d box end nails shall be substituted for 2-16d common end nails found in line 8 of Table 2304.9.1.~~

~~**Exception:** In shear walls where the design load is greater than 490 plf (7151 N/m) but less than 840 plf (12 264 N/m) using load and resistance factor design or greater than 350 plf (5110 N/m) but less than 600 plf (8760 N/m) using allowable stress design, the sill plate is permitted to be a 2-inch (51 mm) nominal member if the sill plate is anchored by two times the number of bolts or anchor straps required by design and 0.229-inch by 3-inch by 3-inch (5.82mmby 76mmby 76mm) plate washers are used.~~

2308.6 Foundation plates or sills. Foundations and footings shall be as specified in Chapter 18. Foundation plates or sills resting on concrete or masonry foundations shall comply with Section 2304.3.1. Foundation plates or sills shall be bolted or anchored to the foundation with not less than 1/2-inch-diameter (12.7 mm) steel bolts or approved anchors spaced to provide equivalent anchorage as the steel bolts. Bolts shall be embedded at least 7 inches (178 mm) into concrete or masonry, and spaced not more than 6 feet (1829 mm) apart. There shall be a minimum of two bolts or anchor straps per piece with one bolt or anchor strap located not more than 12 inches (305 mm) or less than 4 inches (102 mm) from each end of each piece. A properly sized nut and washer shall be tightened on each bolt to the plate.

2308.12.8 Sill plate anchorage. Sill plates shall be anchored with anchor bolts with steel plate washers between the foundation sill plate and the nut, or approved anchor straps load rated in accordance with Section 1715.1. Such washers shall be a minimum of 0.229 inch by 3 inches by 3 inches (5.82 mm by 76 mm by 76 mm) in size. The hole in the plate washer is permitted to be diagonally slotted with a width of up to 3/16 inch (4.76 mm) larger than the bolt diameter and a slot length not to exceed 1 ¾ inches (44 mm), provided a standard cut washer is placed between the plate washer and the nut.

2308.12.9 Sill plate anchorage in Seismic Design Category E. Steel bolts with a minimum nominal diameter of e inch (15.9 mm) or approved foundation anchor straps load rated in accordance with Section 1715.1 and spaced to provide equivalent anchorage shall be used in Seismic Design Category E.

Committee Reason: The code change allows the use of anchor straps as an alternative for foundation anchorage. The modification is for consistency with the action taken on S82-06/07.

Assembly Action: **None**

PART II — IRC
Committee Action: **Approved as Submitted**

Committee Reason: This change, allowing the use of anchor straps, provides a technique that adds versatility to the code.

Assembly Action: **None**

Final Hearing Results

S90-06/07, Part I	**AM**
S90-06/07, Part II	**AS**

Code Change No: S95-06/07

Original Proposal

Table 2306.3.1, Table 2306.4.1

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

Revise as follows:

TABLE 2306.3.1
ALLOWABLE SHEAR (POUNDS PER FOOT) FOR WOOD STRUCTURAL PANEL DIAPHRAGMS WITH FRAMING OF DOUGLAS-FIR-LARCH, OR SOUTHERN PINE [a] FOR WIND OR SEISMIC LOADING [h]

c. Framing at adjoining panel edges shall be 3 inches nominal or wider, and nails shall be staggered where ~~nails are spaced~~ panel edge nailing is specified at ~~2 inches o.c. or~~ 2-1/2 inches o.c. or less.
d. Framing at adjoining panel edges shall be 3 inches nominal or wider, and nails shall be staggered where both of the following conditions are met: (1) 10d nails having penetration into framing of more than 1-1/2 inches and (2) ~~nails are spaced~~ panel edge nailing is specified at 3 inches o.c. or less.

(Portions of table and footnotes not shown remain unchanged)

TABLE 2306.4.1
ALLOWABLE SHEAR (POUNDS PER FOOT) FOR WOOD STRUCTURAL PANEL SHEAR WALLS WITH FRAMING OF DOUGLAS-FIR-LARCH, OR SOUTHERN PINE [a] FOR WIND OR SEISMIC LOADING [b, h, i, j, l]

e. Framing at adjoining panel edges shall be 3 inches nominal or wider, and nails shall be staggered where ~~nails are spaced~~ panel edge nailing is specified at 2 inches on center or less.
f. Framing at adjoining panel edges shall be 3 inches nominal or wider, and nails shall be staggered where both of the following conditions are met: (1) 10d (3" x 0.148") nails having penetration into framing of more than 1-1/2 inches and (2) ~~nails are spaced~~ panel edge nailing is specified at 3 inches on center or less.

(Portions of table and footnotes not shown remain unchanged)

Reason: Substitute revised material for current provision of the Code.
The purpose of the proposal is to establish technically sound language in the footnotes that require the staggering of nails based on their spacing. The allowable shear values in Tables 2306.3.1 and 2306.4.1 are based on specified spacing of panel edge nailing. The in-place spacing, however, can vary substantially. The requirements for staggering are intended to be based on the specified spacing from which allowable shear values are determined. The current language in the footnotes, however, implies that the requirements are based on the in-place spacing. The proposed revisions will establish that the requirements are based on the specified spacing.
The term "or less" is added to Footnotes (e) and (f) of Table 2306.4.1 for consistency with Footnote (d) of Table 2306.3.1. It is also done to eliminate the possibility of specifying, for example, panel edge nailing at 1.9 inches o.c. to avoid the requirements in Footnote (e) of Table 2306.4.1 for 3-inch nominal framing members and staggered nailing. Footnote (c) of Table 2306.3.1 is revised for a similar reason: eliminate the possibility of specifying panel edge nailing at 2.4 or 1.9 inches o.c. to avoid the requirements for 3-inch nominal framing members and staggered nailing.

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

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: This proposal was disapproved based on the approval of code change S83-06/07.

Assembly Action: **None**

Public Comments

Individual Consideration Agenda

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

Public Comment:

Edwin T. Huston, P.E., S.E., Smith & Huston, Inc., Consulting Engineers, representing National Council of Structural Engineering Associations, Code Advisory Committee, requests Approval as Submitted.

Commenter=s Reason: At the Orlando hearings, disapproval of Proposal S95 was requested due to committee action on Proposals S82 and S83, which were for approval as amended and approval as submitted, respectively. Proposals S82 and S83, however, do not delete Tables 2306.3.1 and 2306.4.1. Consequently, the proposed revisions to Tables 2306.3.1 and 2306.4.1 in Proposal S95 remain valid.

Final Hearing Results

S95-06/07 **AS**

Code Change No: S97-06/07

Original Proposal

Table 2306.3.1, Table 2306.4.1, 2307.1.1

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

Revise as follows:

TABLE 2306.3.1
ALLOWABLE SHEAR (POUNDS PER FOOT) FOR WOOD STRUCTURAL PANEL DIAPHRAGMS WITH FRAMING OF DOUGLAS-FIR-LARCH, OR SOUTHERN PINE [a] FOR WIND OR SEISMIC LOADING [h]

c. Framing at adjoining panel edges shall be 3 inches nominal or wider, and nails at all panel edges shall be staggered where nails are spaced 2 inches o.c. or 2-1/2 inches o.c.
d. Framing at adjoining panel edges shall be 3 inches nominal or wider, and nails at all panel edges shall be staggered where both of the following conditions are met: (1) 10d nails having penetration into framing of more than 1-1/2 inches and (2) nails are spaced 3 inches o.c. or less.

(Portions of table and footnotes not shown remain unchanged)

TABLE 2306.4.1
ALLOWABLE SHEAR (POUNDS PER FOOT) FOR WOOD STRUCTURAL PANEL SHEAR WALLS WITH FRAMING OF DOUGLAS-FIR-LARCH, OR SOUTHERN PINE [a] FOR WIND OR SEISMIC LOADING [b, h, i, j, l]

e. Framing at adjoining panel edges shall be 3 inches nominal or wider, and nails at all panel edges shall be staggered where nails are spaced 2 inches on center.
f. Framing at adjoining panel edges shall be 3 inches nominal or wider, and nails at all panel edges shall be staggered where both of the following conditions are met: (1) 10d (3" x 0.148") nails having penetration into framing of more than 1-1/2 inches and (2) nails are spaced 3 inches on center.
h. Where panels are applied on both faces of a wall and nail spacing is less than 6 inches o.c. on either side, panel joints shall be offset to fall on different framing members. Or framing shall be 3-inch nominal or thicker at adjoining panel edges and nails ~~on each side~~ at all panel edges shall be staggered.
i. In Seismic Design Category D, E or F, where shear design values exceed 350 pounds per lineal foot, all framing members receiving edge nailing from abutting panels shall not be less than a single 3-inch nominal member, or two 2-inch nominal members fastened together in accordance with Section 2306.1 to transfer the design shear value between framing members. Wood structural panel joint and sill plate nailing shall be staggered ~~in all cases~~ at all panel edges. See Section 2305.3.11 for sill plate size and anchorage requirements.

(Portions of table and footnotes not shown remain unchanged)

2307.1.1 Wood structural panel shear walls. In Seismic Design Category D, E or F, where shear design values exceed 490 pounds per foot (7154 N/m), all framing members receiving edge nailing from abutting panels shall not be less than a single 3-inch (76 mm) nominal member or two 2-inch (51 mm) nominal members fastened together in accordance with AF&PA NDS to transfer the design shear value between framing members. Wood structural panel joint and sill plate nailing shall be staggered ~~in all cases~~ at all panel edges. See Section 2305.3.11 for sill plate size and anchorage requirements.

Reason: Substitute revised material for current provision of the code. There is confusion among designers, code officials and contractors concerning application of the requirement for staggering of nails at the panel edges of wood structural panel sheathing. The intent is to stagger the nails transversely and longitudinally along each panel edge at a recommended spacing of 3/8 to 1/2 inch, thus creating two lines of resistance along each panel edge. The confusion comes from the mistaken assumption that the staggering can occur transversely back and forth at the edges of abutting panels, rather than along each panel edge. The proposed revisions will clarify that the staggering of the nails is required at each panel edge.

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

Public Hearing Results

Committee Action: **Disapproved**

Committee Reason: This proposal was disapproved based on the approval of code change S83-06/07.

Assembly Action: **None**

Public Comments

Individual Consideration Agenda

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

Public Comment:

Edwin T Huston, P.E., S.E., Smith & Huston, Inc., Consulting Engineers, representing National Council of Structural Engineering Associations, Code Advisory Committee, requests Approval as Submitted.

Commenter=s Reason: At the Orlando hearings, disapproval of Proposal S97 was requested due to committee action on Proposals S82 and S83, which were for approval as amended and approval as submitted, respectively. Proposals S82 and S83, however, do not delete Tables 2306.3.1 and 2306.4.1 and Section 2307.1.1. Consequently, the proposed revisions to Tables 2306.3.1 and 2306.4.1 and Section 2307.1.1 in Proposal S97 remain valid.

Final Hearing Results

S97-06/07 **AS**

Code Change No: S98-06/07

Original Proposal

Table 2306.4.5

Proponent: Ben L. Schmid, Consulting Structural Engineer

Revise as follows:

TABLE 2306.4.5
ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES FOR SHEAR WALLS OF LATH AND PLASTER OR GYPSUM BOARD WOOD FRAMED WALL ASSEMBLIES

TYPE OF MATERIAL	THICKNESS OF MATERIAL	WALL CONSTRUCTION	FASTENER SPACING[b] MAXIMUM (inches)	SHEAR VALUE[a, e] (plf)	MINIMUM FASTNER SIZE[c,d,j,k]
Gypsum lath, plain or perforated	⅜" lath and ½" plaster	Unblocked	5	~~100~~ 180	No. 13 gage, 1⅛" long, $^{19}/_{64}$" head, plasterboard nail ~~16 Ga. Galv. Staple, a ⅛" long~~ 0.120" Nail, min ⅜" head, 1¼" long

(Portions of table not shown do not change)

Reason: The purpose of this proposal is to revise the outdated Shear Value for 3/8" gypsum lath and ½" plaster from 100 (plf) to 180 (plf).

All the sheathing material listed in Table 2306.4.5 with shear values for Seismic Force-Resisting Systems are contained in ASCE 7-05 Table 12.2.1 Bearing Wall Systems A.14-Light-framed walls with shear panels of all other materials, resulting with a Response Modification Coefficient, R of 2 (Limited Ductility).

The current code shear value for 3/8" Gypsum lath and ½" plaster is greatly undervalued and has been for over 50 years. Comparison is made of ½" thick Gypsum board, unblocked, with nails at 4 inches on center with a shear value of 125 (plf), listed as Item 4 Table 2306.4.5 with Item 2: 7/8" thick Gypsum lath and plaster, with similar nails at 5 inches on center. In an 8 foot by 8 foot panel, there would be 180 nails in the ½" thick Gypsum board versus 260 similar nails in the 7/8" thick Gypsum lath and plaster.

Factoring $\frac{260}{180}$ nails x $\frac{0.375"}{0.50"}$ nail bearing x $\frac{0.875"}{0.50"}$ gypsum thickness x 125 (plf) for ½" Gypsum board results in 236 (plf) for 7/8" thick Gypsum lath and plaster.

As an accredited expert on wood frame structures, the Author has observed and reported on damage to such sheathed wood framed structures in the past 9 earthquakes in California. The 1971 San Francisco and 1994 Northridge Earthquakes provided comparison of damage to one and two story residences sheathed with stucco exterior walls and ½" Gypsum board versus 7/8" Gypsum lath and plaster on interior walls due to each earthquake. In each geographic area with equal Modified Mercalli (MMI) shaking of between 7 and 9 intensity, the ½" Gypsum board sheathed residences has extensive and severe damage as compared to 7/8" lath and plaster. Combined with the required Cornerite 2" x 2" expanded metal reinforcing at each intersecting wall and ceiling, each room becomes a box and supports its tributary lateral loading including walls, ceilings or floor joists and roof area due to the continuity furnished by 7/8" lath and plaster.

Immediately after the 1994 Northridge Earthquake, the City and County of Los Angeles funded cyclic load testing of plywood, stucco and Gypsum board sheathing at the Civil Engineering Laboratory at the University of California an Irvine. The tests verified that the Allowable Stress Design values should have been reduced 50 per cent per note 1 in Table 25-I in the 1994 UBC.

The Board of Directors of the Structural Engineers Association of Southern California authorized and funded cyclic testing, using the testing protocol developed for the sheathing testing at UC Irvine, for 7/8" Gypsum lath and plaster. The three 8 foot by 8 foot tested panels developed an average of 205 (plf). The testing was done at the Specialized Testing Laboratory, ICC-ES certified. Ben Schmid, S.E. and Ted Christensen, S.E. continuously observed the tests. The test allowed the conclusion that the existing Shear Value shown in Table 2306.4.5 is overly restrictive.

Data for cyclic tests and resulting Load/Deflection curves are submitted for 7/8" thick Gypsum lath and plaster, ½" and 5/8" Gypsum Board. Allowable Stress Design (ASD) values are developed from the curve data at effective Strength Limit State multiplied by 0.65 for Load and Resistance Factor Design (LRFD), then divided by 1.4 for ASD.

Bibliography:

1. Gypsum Board tests number 7 and 8 reported in Final Report of a Testing Program of Light-framed Walls with Wood Sheathed Shear Panels by the Structural Engineers Association of Southern California and Department of Civil and Environmental Engineering, University of California, Irvine, dated December, 2001. Research Award Number "FEMA-DR-1008-8011."

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

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

TABLE 2306.4.5
ALLOWABLE SHEAR FOR WIND OR SEISMIC FORCES FOR SHEAR WALLS OF LATHAND PLASTER OR GYPSUM BOARD WOOD FRAMED WALL ASSEMBLIES

TYPE OF MATERIAL	THICKNESS OF MATERIAL	WALL CONSTRUCTION	FASTENER SPACINGb MAXIMUM (inches)	SHEAR VALUEa, e (plf)	MINIMUM FASTNER SIZEc,d,j,k
2. Gypsum lath, plain or perforated with vertical joints staggered	d" lath and ½" plaster	Unblocked	5	180	No. 13 gage, 1c" long,19/64" head, plasterboard nail 0.120" Nail, min d" head, 1¼" long
3. Gypsum lath, plain or perforated	d" lath and ½" plaster	Unblocked	5	100	16 Ga. Galv. Staple, 1 c" long 0.120" Nail, min d" head, 1¼" long

(Portions of table not shown remain unchanged)

Committee Reason: The proposal adds a valuable option for shear wall buildings. The proponent provided adequate testing and verification for increasing the allowable shear. The modification clarifies that the vertical joints must be staggered since that is what was tested in justifying the higher shear values. In addition the modification retains the current allowable shear values since these would still be permissible if joints are not staggered.

Assembly Action: **None**

Final Hearing Results

S98-06/07 **AM**

Code Change No: S99-06/07

Original Proposal

Section: 2307.1

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

Revise as follows:

2307.1 Load and resistance factor design. The structural analysis and construction of wood elements and structures using load and resistance factor design shall be in accordance with AF&PA NDS <u>and AF&PA SDPWS</u>.

Reason: Adds reference to the AF&PA Special Design Provisions for Wind and Seismic (SDPWS) to Section 2307 pertaining to load and resistance factor design (LRFD) for wood elements and structures. The AF&PA SDPWS is currently adopted by reference.

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This change adds a cross-reference to the standard that was adopted for lateral design by S82-06/07.

Assembly Action: **None**

Final Hearing Results

S99-06/07 **AS**

Code Change No: S100-06/07

Original Proposal

Sections: 2308.2; IRC R301.3

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: Kirk Grundahl, Wood Truss Council of America, representing the Structural Building Components Industry

PART I – IBC

Revise as follows:

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

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

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

2. Bearing wall ~~floor-to-floor heights~~ story height shall not exceed a stud height of 10 feet (3048 mm) plus a height of floor framing not to exceed 16 inches (406 mm). Floor framing height shall be permitted to exceed this limit provided the story height limit is not exceeded.

(No changes to items 3 through 7)

PART II – IRC

Revise as follows:

R301.3 Story height. Buildings constructed in accordance with these provisions shall be limited to story heights of not more than the following:

1. For wood wall framing, the laterally unsupported bearing wall stud height permitted by Table R602.3(5) plus a height of floor framing not to exceed 16 inches.

   Exception: For wood framed wall buildings with bracing in accordance with Table R602.10.1, the wall stud clear height used to determine the maximum permitted story height may be increased to 12 feet without requiring an engineered design for the building wind and seismic force resisting systems provided that the length of bracing required by Table R602.10.1 is increased by multiplying by a factor of 1.20.Wall studs are still subject to the requirements of this section.

2. For steel wall framing, a stud height of 10 feet, plus a height of floor framing not to exceed 16 inches.
3. For masonry walls, a maximum bearing wall clear height of 12 feet plus a height of floor framing not to exceed 16 inches.

   Exception: An additional 8 feet is permitted for gable end walls.

4. For insulating concrete form walls, the maximum bearing wall height per story as permitted by Section 611 tables plus a height of floor framing not to exceed 16 inches.

Individual walls or walls studs shall be permitted to exceed these limits as permitted by Chapter 6 provisions, provided story heights are not exceeded. Floor framing height shall be permitted to exceed these limits provided the story height limit is not exceeded An engineered design shall be provided for the wall or wall framing members when they exceed the limits of Chapter 6. Where the story height limits are exceeded, an engineered design shall be provided in accordance with the International Building Code for the overall wind and seismic force resisting systems.

Reason: (IBC) The purpose of this proposed code change is to clarify the code language regarding story height limitations and how stud height limitations and floor framing limitations are to be interpreted.

The current story height requirements were introduced in the 2003 code cycle into the IRC by BCCS (RB39-02). The final proposal was extensively modified from the original proposal which dealt only with the stud height issue. The committee reason for accepting RB39-02 as modified was given as follows:

Committee Reason: Based on proponent's published reason. This code change has been modified from that originally submitted to the ICC in order to coordinate with the material limits currently contained in (and, in the case of wood, currently being added to) the IRC. The exception to Item 1 has been added at the request of other interested parties to accommodate the high ceilings that are currently common in some parts of the country.

No technical supporting documentation was provided on the 16 inch limitation on floor framing. Our assumption is that the intent of the code modification was to arrive at a maximum story height by setting a maximum floor framing depth to be used with a maximum stud height. However, the way the current language has been interpreted, there becomes a limit of 16 inches on floor framing regardless of stud height. Both I-joist and floor truss depths can exceed a 16 inches. Unless technical justification can be provided otherwise, there is no reason to limit the depth of the floor framing if the story height is not exceeded.

The IRC 16 inch floor framing limitation was brought into the IBC Conventional Light-Frame Construction section in the 2006 code change cycle in S191 with the following reasoning and was accepted as submitted:

Reason: Table 2308.9.1 of the IBC contains the spacing requirement for 10 foot studs, and it is assumed that 10 foot studs can be used for walls in buildings built under the conventional construction provisions of Section 2308. However, the current wording of Section 2308.2 of the code limits the floor-to-floor height to 10 feet, which precludes the use of 10-foot studs in bearing walls. The proposed change will make the scope of the section consistent with the requirements contained therein and also limit floor-to-floor heights for conventional light-frame construction provisions. A similar limitation can be found in Section R301.3 of the International Residential Code.

As in the case of the IRC, the intention was to limit story height not to limit floor framing height. Chapter 5 defines story height as follows: HEIGHT, STORY. The vertical distance from top to top of two successive finished floor surfaces; and, for the topmost story, from the top of the floor finish to the top of the ceiling joists or, where there is not a ceiling, to the top of the roof rafters.

There are no general limits on floor framing height elsewhere in the IBC, nor are there any limitations on floor framing height in chapter 23.

(IRC) Current limitations are based on a floor height of 16 inches. The current story height requirements were introduced in the 2003 code cycle by BCCS (RB39-02). The final proposal was extensively modified from the original proposal which dealt only with the stud height issue. The committee reason for accepting RB39-02 as modified was given as follows:

> Committee Reason: Based on proponent's published reason. This code change has been modified from that originally submitted to the ICC in order to coordinate with the material limits currently contained in (and, in the case of wood, currently being added to) the IRC. The exception to Item 1 has been added at the request of other interested parties to accommodate the high ceilings that are currently common in some parts of the country.

No technical supporting documentation was provided for the 16 inch limitation on floor framing. Our assumption is that the intent of the code modification was to arrive at a maximum story height by setting a maximum floor framing depth to be used with a maximum stud height. However, the way the current language has been interpreted, a limit of 16 inches on floor framing regardless of stud height. Both I-joist and floor truss depths can exceed a 16 inches. Unless technical justification can be provided otherwise, there is no reason to limit the floor framing if the story height is not exceeded.

Chapter 2 defines story height as follows: HEIGHT, STORY. The vertical distance from top to top of two successive tiers of beams or finished floor surfaces; and, for the topmost story, from the top of the floor finish to the top of the ceiling joists or, where there is not a ceiling, to the top of the roof rafters.

Chapter 5 includes no limits on floor framing height, nor are there any limitations on floor framing in any of the individual material requirement sections.

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

Public Hearing Results

PART I — IBC
Committee Action: **Approved as Modified**

Modify proposal as follows:

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

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

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

2. Maximum floor-to-floor height shall not exceed 11 feet 7 inches (mm). Bearing wall ~~story~~ height shall not exceed a stud height of 10 feet (3048 mm). ~~plus a height of floor framing not to exceed 16 inches (406 mm). Floor framing height shall be permitted to exceed this limit provided the story height limit is not exceeded.~~

(No changes to items 3 through 7)
Committee Reason: This proposal clarifies the limitation on story height under the conventional construction provisions. The modification states the maximum floor to floor height directly.

Assembly Action: **None**

PART II — IRC
Committee Action: **Approved as Modified**

Modify the proposal as follows:

R301.3 Story height. Buildings constructed in accordance with these provisions shall be limited to story heights of not more than the following:

(No change to items 1 - 4)

Individual walls or walls studs shall be permitted to exceed these limits as permitted by Chapter 6 provisions, provided story heights are not exceeded. Floor framing height shall be permitted to exceed these limits provided the story height ~~limit is not exceeded.~~ does not exceed 11'-7". An engineered design shall be provided for the wall or wall framing members when they exceed the limits of Chapter 6. Where the story height limits are exceeded, an engineered design shall be provided in accordance with the International Building code for the overall wind and seismic force resisting systems.

Committee Reason: This change passed IBC Structural. Passing this keeps the code language the same in the IBC and the IRC. The added language allows more design flexibility and would allow I joists in garages and others where a longer uninterrupted span is desired or required.

Assembly Action: **None**

Final Hearing Results

S100-06/07, Part I	**AM**
S100-06/07, Part II	**AM**

Code Change No: S101-06/07

Original Proposal

Section: 2308.2

Proponent: Randall Shackelford, Simpson Strong-Tie Co.

Revise as follows:

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

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

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

2. Bearing wall floor-to-floor heights shall not exceed a stud height of 10 feet (3048 mm) plus a height of floor framing not to exceed 16 inches (406 mm).
3. Loads as determined in Chapter 16 shall not exceed the following:
   3.1. Average dead loads shall not exceed 15 psf (718 N/m^2) for combined roof and ceiling, exterior walls, floors and partitions.

   Exceptions:

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

   3.2. Live loads shall not exceed 40 psf (1916 N/m^2) for floors.
   3.3. Ground snow loads shall not exceed 50 psf (2395 N/m^2).

4. Wind speeds shall not exceed 100 miles per hour (mph) (44 m/s) (3-second gust).

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

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

Reason: The purpose of this proposal is to revise wind limitation in the IBC to match the IRC

Studies conducted by the Institute for Business and Home Safety show that the conventional construction requirements of the IBC and IRC are frequently inadequate for wood buildings built where the design windspeed exceeds 100 mph. The IRC was revised to reflect this last code change cycle, but the IBC was not. This change will make the IRC and IBC have the same limitations.

Cost Impact: The code change proposal will increase the cost of construction in areas between 100 and 110 miles per hour if the buildings are currently being built without consideration of wind forces.

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This code change aligns the wind limitations for the IBC conventional construction provisions with those in the IRC.

Assembly Action: **None**

Final Hearing Results

S101-06/07 **AS**

Code Change No: S102-06/07

Original Proposal

Sections: 2308.11.2, 2308.12.2

Proponent: David W. Ware, Owens Corning

Revise as follows:

2308.11.2 Concrete or masonry. Concrete or masonry walls and stone or masonry veneer shall not extend above the basement.

Exceptions:

1. Stone and masonry veneer is permitted to be used in the first two stories above grade plane or the first three stories above grade plane where the lowest story has concrete or masonry walls in Seismic Design Category B, provided that structural use panel wall bracing is used and the length of bracing provided is one- and one-half times the required length as determined in Table 2308.9.3(1).
2. Stone and masonry veneer is permitted to be used in the first story above grade plane or the first two stories above grade plane where the lowest story has concrete or masonry walls in Seismic Design Category B or C.
3. Stone and masonry veneer is permitted to be used in the first two stories above grade plane in Seismic Design Categories B and C, provided the following criteria are met:
   - 3.1. Type of brace per Section 2308.9.3 shall be Method 3 and the allowable shear capacity in accordance with Table 2306.4.1 shall be a minimum of 350 plf (5108 N/m).
   - 3.2. The bracing of the top story shall be located at each end and at least every 25 feet (7620 mm) o.c. but not less than 40 percent of the braced wall line. The bracing of the first story shall be located at each end and at least every 25 feet (7620 mm) o.c. but not less than 35 percent of the braced wall line.
   - 3.3. Hold-down connectors shall be provided at the ends of braced walls for the second floor to first floor wall assembly with an allowable design of 2,000 pounds (8896 N). Hold-down connectors shall be provided at the ends of each wall segment of the braced walls for the first floor to foundation with an allowable design of 3,900 pounds (17 347 N). In all cases, the hold-down connector force shall be transferred to the foundation.
   - 3.4. Cripple walls shall not be permitted.

2308.12.2 Concrete or masonry. Concrete or masonry walls and stone or masonry veneer shall not extend above the basement.

Exception: ~~M~~ Stone and masonry veneer is permitted to be used in the first story above grade plane in Seismic Design Category D, provided the following criteria are met:

1. Type of brace in accordance with Section 2308.9.3 shall be Method 3 and the allowable shear capacity in accordance with Table 2306.4.1 shall be a minimum of 350 plf (5108 N/m).
2. The bracing of the first story shall be located at each end and at least every 25 feet (7620 mm) o.c. but not less than 45 percent of the braced wall line.
3. Hold-down connectors shall be provided at the ends of braced walls for the first floor to foundation with an allowable design of 2,100 pounds (9341 N).
4. Cripple walls shall not be permitted.

Reason: This proposal ensures that requirements for masonry and stone materials are equally applied to all veneer type materials and provides consistency of terminology throughout relevant code sections. The intent of this code change is coordination and clarification.
Past code language for stone and masonry materials has not clearly delineated specific provisions applying to adhered versus anchored systems. These proposed code changes build on approved modifications to these and ancillary code sections approved last year.
The type of veneer is broadened to include stone and masonry, consistent with IBC Chapter 14 and IRC Section 703; and is consistent with changes approved last year to IBC Section 2308.2.

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This proposal clarifies the conventional construction provisions pertaining to the use of stone or masonry veneer. The necessary weight limitations are currently stated in Section 2308.2.

Assembly Action: **None**

Final Hearing Results

S102-06/07 **AS**

Code Change No: S105-06/07

Original Proposal

Sections: 2406.1.1, 2406.2, Chapter 35; IRC R308.3, R308.3.1

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: William E. Koffel, P.E., Koffel Associates, Inc., representing Glazing Industry Code Committee

PART I – IBC

1. Revise as follows:

2406.1.1 ~~CPSC 16 CFR 1201~~. <u>Impact test.</u> Except as provided in Sections 2406.1.2 through 2406.1.4, all glazing shall pass the <u>impact</u> test requirements of ~~CPSC 16 CFR 1201, listed in Chapter 35~~ <u>Section 2406.2</u>. ~~Glazing shall comply with the CPSC 16 CFR, Part 1201 criteria, for Category I or II as indicated in Table 2406.1.~~

2. Add new text as follows:

<u>**2406.2 Impact test.** Where required by other sections of the Code, glazing shall be tested in accordance with CPSC 16 CFR 1201. Glazing shall comply with the test criteria for Category I or II as indicated in Table 2406.2(1)</u>

<u>**Exception:** Glazing not in doors or enclosures for hot tubs, whirlpools, saunas, steam rooms, bathtubs and showers shall be permitted to be tested in accordance with ANSI Z97.1. Glazing shall comply with the test criteria for Class A or B as indicated in Table 2406.2(2).</u>

3. Revise table as follows:

TABLE ~~2406.1~~ 2406.2(1)
MINIMUM CATEGORY CLASSIFICATION OF GLAZING USING CPSC 16 CFR 1201

(No change to table entries)

4. Add new table as follows:

TABLE 2406.2(2)
MINIMUM CATEGORY CLASSIFICATION OF GLAZING USING ANSI Z97.1

EXPOSED SURFACE AREA OF ONE SIDE OF ONE LITE	GLAZING IN STORM OR COMBINATION DOORS (Category class)	GLAZING IN DOORS (Category class)	GLAZED PANELS REGULATED BY ITEM 7 OF SECTION 2406.3 (Category class)	GLAZED PANELS REGULATED BY ITEM 6 OF SECTION 2406.3 (Category class)	DOORS AND ENCLOSURES REGULATED BY ITEM 5 OF SECTION 2406.3 (Category class)	SLIDING GLASS DOORS PATIO TYPE (Category class)
9 square feet or less	B	B	No requirement	B	A	A
More than 9 square feet	A	A	A	A	A	A

5. Revise Chapter 35 as follows:

ANSI
Z97.1-~~84 (R1994)~~ 04

PART II – IRC

1. Revise as follows:

R308.3 Human impact loads. Individual glazed areas, including glass mirrors in hazardous locations such as those indicated as defined in Section R308.4, shall pass the test requirements of ~~CPSC 16 CFR, Part 1201~~ Section 308.3.1. ~~Glazing shall comply with CPSC 16 CFR, Part 1201 criteria for Category I or Category II as indicated in Table R308.3.~~

Exception: Louvered windows and jalousies shall comply with Section R308.2.

2. Add new text as follows:

R308.3.1 Impact Test. Where required by other sections of the Code, glazing shall be tested in accordance with CPSC 16 CFR 1201. Glazing shall comply with the test criteria for Category I or II as indicated in Table R308.3.1(1).

Exception: Glazing not in doors or enclosures for hot tubs, whirlpools, saunas, steam rooms, bathtubs and showers shall be permitted to be tested in accordance with ANSI Z97.1. Glazing shall comply with the test criteria for Class A or B as indicated in Table R308.3.1 (2).

3. Revise table as follows:

TABLE ~~R308.3~~ R308.3.1(1)
MINIMUM CATEGORY CLASSIFICATION OF GLAZING USING CPSC 16 CFR 1201

(Portions of table not shown do not change)

3. Add new table as follows:

TABLE R308.3.1(2)
MINIMUM CATEGORY CLASSIFICATION OF GLAZING USING ANSI Z97.1

EXPOSED SURFACE AREA OF ONE SIDE OF ONE LITE GLAZING IN	GLAZING IN STORM OR COMBINATION DOORS (Category class)	GLAZING IN DOORS (Category class)	GLAZED PANELS REGULATED BY ITEM 7 OF SECTION 2406.3 (Category class)	GLAZED PANELS REGULATED BY ITEM 6 OF SECTION 2406.3 (Category class)	DOORS AND ENCLOSURES REGULATED BY ITEM 5 OF SECTION 2406.3 (Category class)	SLIDING GLASS DOORS PATIO TYPE (Category class)
9 square feet or less	B	B	No requirement	B	A	A
More than 9 square fee	A	A	A	A	A	A

For SI: 1 square foot = 0.0929 m^2.

Reason: In 1977, the U.S. Consumer Products Safety Commission ("CPSC") adopted as a mandatory federal safety regulation *Safety Standard for Architectural Glazing Materials*, codified at 16 CFR Part 1201. The CPSC amended its *Safety Standard for Architectural Glazing Materials* on several occasions subsequent to its initial adoption, the last time on June 28, 1982.

Set forth below are the more significant differences between these two standards, both standards applicable to safety glazing materials used in architectural applications. This reason statement makes no attempt to summarize all pertinent provisions of the two standards, only their significant differences.

The principal differences between the CPSC's 16 CFR 1201 standard and the ANSI Z97.1-2004 standard relate to their scope and function. The CPSC standard is not only a test method and a procedure for determining the safety performance of architectural glazing, but also a federal standard that mandates where and when safety glazing materials must be used in architectural applications and preempts any non-identical state or local standard. In contrast, ANSI Z97 is only a voluntary safety performance specification and test method. It does not purport to indicate where and when safety glazing materials must be used, leaving those determinations up to the building codes and to glass and fenestration specifiers. In this instance, the IBC provides the requirements regarding the safety performance of architectural glazing beyond that which is covered by the federal standard.

The CPSC requires the installation of safety glazing materials meeting 16 CFR 1201 only in storm doors, combination doors, entrance-exit doors, sliding patio doors, closet doors, and shower and tub doors and enclosures. Other than that, meeting CPSC's requirements is necessary only when and if a building code authority or other jurisdiction adopting safety glazing laws specifically mandates that safety glazing comply with the CPSC standard, 16 CFR 1201 -- and most building codes do. ANSI Z97, as a voluntary standard, applies only when, where, and if it is adopted by a building code authority or is specified in the approved plans and specifications of the architect, building contractor, or other glass specifier.

Test Specimens: For impact testing, the CPSC requires only one specimen of each nominal thickness be submitted for testing and specifies it must be the largest size the manufacturer produces up to a maximum of size of 34″ by 76″. ANSI Z97 requires that four specimens of each nominal thickness and size must be impact-tested. The manufacturer has the option of testing either 34″ by 76″ specimens or the largest size it commercially produces less than 34″ by 76″, but with a minimum size of 24″ by 30″. A nominal thickness is defined as +/- ⅛-inch.

Types of Glass: The CPSC standard has no performance tests for plastics or for bent glass. ANSI Z97 has specific tests for both.

The CPSC standard does not prohibit the use of ordinary annealed glass in hazardous locations as long as it passes the appropriate impact tests, consistent with the concept of a performance based impact test. (Thick, heavy annealed glass is likely to pass the CPSC 18-inch drop-height and 48-inch drop-height impact tests for Category I and II locations.) ANSI Z97.1-2004 contains an express limitation on annealed glass: "Monolithic annealed in any thickness is not considered safety glazing material under this standard."

Asymmetrical Glazing Material: The CPSC standard requires all asymmetrical glazing materials to be impacted on both sides of each specimen and then evaluated under the pass-fail criteria. There is no exception. ANSI Z97 requires that, with the exception of mirror glazing, all asymmetrical glass specimens must be impacted on both sides, two on one side and two on the other. With respect to mirror glazing products using reinforced or non-reinforced organic adhesive backing, all four specimens must be impacted only on the non-reinforced side "and with no other material applied."

Impact Categories or Levels: The CPSC standard has two distinct impact levels or categories, Category I and Category II, and specifies which defined hazardous location must contain Category II safety glazing materials and which may use Category I glazing materials. Glazing material successfully passing the impact test of a 48-inch drop height, a 400 foot-pound impact, is classified as "Category II" glass. Glazing material passing the 18-inch drop height, a 150 foot-pounds impact, is classified as "Category I" glass. ANSI Z97 has adopted three separate impact categories or classes, based upon impact performance. ANSI Z97's Class A glazing materials are comparable to the CPSC's Category II glazing materials, passing a 48-inch drop height test, and its Class B glazing materials are comparable to the CPSC's Category I glazing materials, passing the 18-inch drop height test. ANSI Z97 also has a product-specific Class C impact test, a 12-inch drop height test, applicable only for fire-resistant glazing materials. However, the proposed code change does not identify Class C as an acceptable product for use in hazardous locations.

Pass-Fail Impact Criteria: The CPSC standard, like the ANSI standard, offers alternative criteria for evaluating whether a test specimen passes the impact test. The CPSC standard considers the specimen a pass if a 3-inch diameter solid steel ball, weighing 4 lbs., will not pass through the opening when placed on the specimen for one second. ANSI uses the 3-inch sphere measure, but does not require the sphere be a steel ball and does not specify the weight of the 3-inch sphere, but does require that the sphere not pass freely through the opening when a force of 4 lbs. is applied to the sphere. There is no time element associated with this alternative.

A second alternative pass-fail criterion under the CPSC standard involves weighing the 10 largest particles selected within five minutes after the impact test -- they must weigh no more than the equivalent weight of 10 square inches of the original specimen. The ANSI standard has an almost identical criterion, except the 10 largest particles must be "crack-free." It also includes additional product-specific qualifications applicable solely to selecting the 10 largest particles of tempered glass and offers a formula for determining the weight of 10 square inches of the original specimen.

The CPSC standard has no separate pass-fail impact criteria for the scenario in which the glass specimen separates from the frame after impact and breaks or produces a hole in the glass. The ANSI standard has a special criterion for that scenario -- to pass, the glass is subjected to the same 3-inch sphere measure or to the weight criterion for the 10 largest crack-free particles.

The CPSC standard involves impact-testing of only a single specimen of each nominal glass thickness. Accordingly, if that specimen passes, all glass of that type and thickness is deemed to pass. Under the ANSI standard, four specimens of each type, size, and thickness must be impact tested, and if any one of the four specimens fails, there is a failure of that specific type, thickness, and size.

Impact Testing Apparatus: Relatively minor technical differences exist between the test frames and impactors specified in the CPSC standard and those in ANSI Z97.1. The ANSI standard prescribes special test frame and subframe configurations for impact-testing bent glass; the CPSC standard has no provisions for testing bent glass. The ANSI standard includes detailed specifications for the impactor suspension device and traction and release system and for their operation; the CPSC standard does not.

Weathering Tests: The CPSC standard requires a weathering test only for organic coated glass. ANSI requires a weathering test for laminated glass and plastics as well as for organic coated glass.

The CPSC accelerated weathering test (only for organic coated glass) uses the xenon arc Weather-Ometer. The ANSI standard gives the manufacturer the choice of one of three weathering exposure alternatives, the xenon arc exposure, the enclosed twin carbon arc exposure, or the one-year outdoor exposure in South Florida. The ANSI prescribed xenon arch apparatus and procedure are the more current versions of the pertinent ASTM standards, ASTM G 155 and ASTM D 2565-92A, than the versions referenced in the CPSC standard. The CPSC's xenon arc procedure for interpreting results of the adhesion test requires an average adhesion value or pull force of no less than 90% of the average of the unexposed organic-coated glass specimens in order to "pass," whereas the ANSI standard requires no less than 75% of the average of the unexposed specimens.

Indoor Aging Tests: The CPSC standard does not prescribe any indoor aging test; the ANSI standard requires specified indoor aging tests for plastics and organic coated glass intended for indoor-use only, followed by impact tests.

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

Public Hearing Results

PART I – IBC
Committee Action: Approved as Modified

Modify proposal as follows:

2406.1.1 Impact test. Except as provided in Sections 2406.1.2 through 2406.1.4, all glazing shall pass the impact test requirements of Section 2406.2.

2406.2 Impact test. Where required by other sections of the Code, glazing shall be tested in accordance with CPSC 16 CFR 1201. Glazing shall comply with the test criteria for Category I or II as indicated in Table 2406.2(1)

Exception: Glazing not in doors or enclosures for hot tubs, whirlpools, saunas, steam rooms, bathtubs and showers shall be permitted to be tested in accordance with ANSI Z97.1. Glazing shall comply with the test criteria for Class A or B as indicated in Table 2406.2(2).

TABLE 2406.2(1)
MINIMUM CATEGORY CLASSIFICATION OF GLAZING USING CPSC 16 CFR 1201

(No change to table contents)

TABLE 2406.2(2)
MINIMUM CATEGORY CLASSIFICATION OF GLAZING USING ANSI Z97.1

EXPOSED SURFACE AREA OF ONE SIDE OF ONE LITE	~~GLAZING IN STORM OR COMBINATION DOORS (Category class)~~	~~GLAZING IN DOORS (Category class)~~	GLAZED PANELS REGULATED BY ITEM 7 OF SECTION 2406.3 (Category class)	GLAZED PANELS REGULATED BY ITEM 6 OF SECTION 2406.3 (Category class)	DOORS AND ENCLOSURES REGULATED BY ITEM 5 OF SECTION 2406.3[a] (Category class)	~~SLIDING GLASS DOORS PATIO TYPE (Category class)~~
9 square feet or less	~~B~~	~~B~~	No requirement	B	A	~~A~~
More than 9 square feet	~~A~~	~~A~~	A	A	A	~~A~~

a. Use is only permitted by the Exception to Section 2406.2.

Chapter 35:

ANSI
Z97.1- 04

Committee Reason: This proposal updates the code to include an exception for Class A and B glazing in accordance with the ANSI standard. The modification clarifies the intention by removing table columns that could lead to misapplication of the code.

Assembly Action: None

PART II – IRC
Committee Action: Disapproved

Committee Reason: There was a modification proposed to this code change proposal when it was heard by the structural committee. The IRC B/E committee voted to disapprove the proposed change since the modification was not also brought before this committee. The proponent was not present to answer questions or provide the modification.

Assembly Action: None

Public Comments

Individual Consideration Agenda

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

Public Comment:

William E. Koffel, P.E., Koffel Associates, Inc., representing Glazing Industry Code Committee, requests Approval as Modified by this Public Comment for Part II.

Modify proposal as follows:

TABLE 308.3.1(2)
MINIMUM CATEGORY CLASSIFICATION OF GLAZING USING ANSI Z97.1

EXPOSED SURFACE AREA OF ONE SIDE OF ONE LITE	~~GLAZING IN STORM OR COMBINATION DOORS (Category class)~~	~~GLAZING IN DOORS (Category class)~~	GLAZED PANELS REGULATED BY ITEM 7 OF SECTION R308.4 (Category class)	GLAZED PANELS REGULATED BY ITEM 6 OF SECTION R308.4 (Category class)	DOORS AND ENCLOSURES REGULATED BY ITEM 5 OF SECTION R308.4[a] (Category class)	~~SLIDING GLASS DOORS PATIO TYPE (Category class)~~
9 square feet or less	~~B~~	~~B~~	No requirement	B	A	~~A~~
More than 9 square fee	~~A~~	~~A~~	A	A	A	~~A~~

For SI: 1 square foot = 0.0929 m^2.
a. Use is only permitted by the exception to Section R308.3.1.

(Portions of the proposal not shown remain unchanged)

Commenter=s Reason: The modification proposed is technically the same as the modification accepted in Part I of the code change proposal. As noted in the Committee Reason, the modification was not submitted during the hearing discussion on Part II of the code change proposal. Approval of the code change as modified by the Public Comment will be consistent with the action taken on Part II of the code change.

Final Hearing Results

S105-06/07, Part I	**AM**
S105-06/07, Part II	**AMPC1**

Code Change No: S106-06/07

Original Proposal

Sections: 2406.2.1, 2407.1, 2408.2.1, 2408.3; IRC R308.1.1

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: William E. Koffel, P.E., Koffel Associates, Inc., representing Glazing Industry Code Committee

PART I – IBC

Revise as follows:

2406.2.1 Multilight assemblies. Multilight glazed assemblies having individual lights not exceeding 1 square foot (0.09 m2) in exposed areas shall have at least one light in the assembly marked as indicated in Section 2406.2. Other lights in the assembly shall be marked "CPSC 16 CFR 1201" <u>"or ANSI Z97.1," as appropriate.</u>

2407.1 Materials. Glass used as a handrail assembly or a guard section shall be constructed of either single fully tempered glass, laminated fully tempered glass or laminated heat-strengthened glass. Glazing in railing in-fill panels shall be of an approved safety glazing material that conforms to the provisions of Section 2406.1.1. For all glazing types, the minimum nominal thickness shall be 1/4 inch (6.4 mm). Fully tempered glass and laminated glass shall comply with Category II of CPSC 16 CFR 1201, or Class A of ANSI Z97.1, listed in Chapter 35.

2408.2.1 Testing. Test methods and loads for individual glazed areas in racquetball and squash courts subject to impact loads shall conform to those of CPSC 16 CFR~~, Part~~ 1201 or ANSI Z97.1, listed in Chapter 35, with impacts being applied at a height of 59 inches (1499 mm) above the playing surface to an actual or simulated glass wall installation with fixtures, fittings and methods of assembly identical to those used in practice.

Glass walls shall comply with the following conditions:

1. A glass wall in a racquetball or squash court, or similar use subject to impact loads, shall remain intact following a test impact.
2. The deflection of such walls shall not be greater than 11/2 inches (38 mm) at the point of impact for a drop height of 48 inches (1219 mm).

Glass doors shall comply with the following conditions:

1. Glass doors shall remain intact following a test impact at the prescribed height in the center of the door.
2. The relative deflection between the edge of a glass door and the adjacent wall shall not exceed the thickness of the wall plus 1/2 inch (12.7 mm) for a drop height of 48 inches (1219 mm).

2408.3 Gymnasiums and basketball courts. Glazing in multipurpose gymnasiums, basketball courts and similar athletic facilities subject to human impact loads shall comply with Category II of CPSC 16 CFR 1201, or Class A of ANSI Z97.1, listed in Chapter 35.

PART II – IRC

Revise as follows:

R308.1.1 Identification of multiple assemblies. Multipane assemblies having individual panes not exceeding 1 square foot (0.09 m^2) in exposed area shall have at least one pane in the assembly identified in accordance with Section R308.1. All other panes in the assembly shall be labeled "CPSC 16 CFR 1201" or "ANSI Z97.1" as appropriate.

Reason: (IBC) For the most part the proposal is a companion to the GICC proposal to recognize ANSI Z97.1 as an alternative test procedure to CPSC 16 CFR 1201 for products not regulated by the federal standard. However, the proposal also addresses some other editorial issues.
Section 2406.2.1 – returns to the language in the 2003 Edition of the IBC recognizing both test standards.
Section 2407.1 – recognizes the ANSI Z97.1 test standard.
Section 2408.2.1 – editorial clean-up with respect to the reference to the CPSC standard for consistency purposes and recognizes the ANSI Z97.1 test standard.
Section 2408.1 – recognizes the ANSI Z97.1 test standard.

It should be noted that Section 2409 already recognizes both test standards so a change was not necessary.

(IRC) The proposal is a companion to the GICC proposal to recognize ANSI Z97.1 as an alternative test procedure to CPSC 16 CFR 1201 for products not regulated by the federal standard. The proposal also inserts the letters "CPSC" in the mark to be consistent with the requirements in the IBC.

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

Public Hearing Results

PART I – IBC
Committee Action: **Approved as Submitted**

Committee Reason: The code change adds an appropriate standard reference and is consistent with the action on S105-06/07.

Assembly Action: **None**

PART II – IRC
Committee Action: **Disapproved**

Committee Reason: The two standards; CPSC 16 CFR and the standard proposed to be added ANSI Z97.1 are not the same and should not be listed as alternatives for one another.

Assembly Action: **None**

Public Comments

Individual Consideration Agenda

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

Public Comment:

William E. Koffel, P.E., Koffel Associates, Inc., representing Glazing Industry Code Committee, requests Approval as Submitted for Part II.

Commenter's Reason: S106-06/07, Part II was disapproved because S105-06/07, Part II was disapproved. A separate Public Comment has been submitted to address the modification that was not available during the discussion of S105-06/07. If the result of the Final Action Hearings is to Approve S105-06/07 as Modified by the Public Comment, approval of S106-06/07, Part II is merely correlative with that action.

Final Hearing Results

S106-06/07, Part I	**AS**
S106-06/07, Part II	**AS**

Code Change No: S108-06/07

Original Proposal

Section: 2407.1.2

Proponent: William E. Koffel, P.E., Koffel Associates, Inc., representing Glazing Industry Code Committee

Revise as follows:

2407.1.2 Support. Each handrail or guard section shall be supported by a minimum of three glass balusters or shall be otherwise supported to remain in place should one baluster panel fail. Glass balusters shall not be installed without an attached handrail or guard.

> **<u>Exception:</u>** <u>A top rail shall not be required where the glass balusters are laminated glass with two or more glass plies of equal thickness and the same glass type. The panels shall be designed to withstand the loads specified in Section 1607.7.</u>

Reason: At the time the provisions of Section 2407.1.2 were developed the dominant glass used for baluster panels was single tempered glass. This glass was structurally adequate and had been successfully used. The required top rail was to provide a degree of protection should one baluster fail for any reason. Tempered glass characteristically may fail in a manner where it evacuates the opening.

In some applications the use of a top rail is an undesirable visual barrier. A typical example is the guard at the front of the spectator levels of sport arenas and theaters. In a number of these installations the top rail has been eliminated. The balusters have been laminated heat-strengthened or tempered glass complying with the IBC structural requirements for top rails. Variances from Section 2407.1.2 have been historically granted by building officials.

If one ply of the laminated glass breaks, the glass will remain in place. Unlike single tempered glass, it will not evacuate the opening. Even in the rare instance where both plies may simultaneously fail, the glass will remain in place.

It should be noted that the GICC has submitted another code change which proposes to delete Section 2407.1.2 in favor of reference two ASTM standards. If the section is deleted as recommended in the other proposal, the proposed exception is not required and this proposal should be recommended for Disapproval.

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

Public Hearing Results

Committee Action: **Approved as Modified**

Modify proposal as follows:

2407.1.2 Support. Each handrail or guard section shall be supported by a minimum of three glass balusters or shall be otherwise supported to remain in place should one baluster panel fail. Glass balusters shall not be installed without an attached handrail or guard.

> **Exception:** A top rail shall not be required where the glass balusters are laminated glass with two or more glass plies of equal thickness and the same glass type <u>when approved by the building official</u>. The panels shall be designed to withstand the loads specified in Section 1607.7.

Committee Reason: The code change adds an option to a top rail that is now often permitted as an alternative method. The modification requires the approval of the building official and is intended to address a concern that in some uses such as schools and hospitals, glass breakage is not acceptable.

Assembly Action: **None**

Final Hearing Results

S108-06/07 **AM**

Code Change No: S110-06/07

Original Proposal

Section: 2508.4, Chapter 35

Proponent: Marcelo M. Hirschler, GBH International, representing American Fire Safety Council

1. Revise as follows:

2508.4 Joint treatment. Gypsum board fire-resistance-rated assemblies shall have joints and fasteners treated.

Exception<u>s:</u>

<u>1.</u> Joint and fastener treatment need not be provided where any of the following conditions occur:
- <u>1</u>.1. Where the gypsum board is to receive a decorative finish such as wood paneling, battens, acoustical finishes or any similar application that would be equivalent to joint treatment.
- <u>1</u>.2. On single-layer systems where joints occur over wood framing members.
- <u>1</u>.3. Square edge or tongue-and-groove edge gypsum board (V-edge), gypsum backing board or gypsum sheathing.
- <u>1</u>.4. On multilayer systems where the joints of adjacent layers are offset from one to another.
- <u>1</u>.5. Assemblies tested without joint treatment.

<u>2. Fire-resistance rated gypsum board assemblies shall be permitted to be fastened with a listed elastomeric joint material instead of being fastened with joint compound and joint tape where the following apply:</u>
- <u>2.1. The complete assembly, with the elastomeric joint material, meets a one hour fire resistance rating</u>
- <u>2.2. When tested in accordance with ASTM E 119, the elastomeric joint material complies with ASTM C 920, and</u>
- <u>2.3. The elastomeric joint material exhibits a modulus of 20 pounds per square inch (psi) or less at 100 percent elongation, when tested in accordance with ASTM C 1523 (both before and after artificial weathering).</u>

2. Add standards to Chapter 35 as follows:

ASTM

<u>C 920-05</u> <u>Standard Specification for Elastomeric Joint Sealants</u>
<u>C 1523-04</u> <u>Standard Test Method for Determining Modulus, Tear and Adhesion Properties of Precured Elastomeric Joint Sealants.</u>

Reason: Elastomeric joint compound materials exist which can replace traditional joint compound and joint tape (traditional mud and tape joint) and generate a gypsum board assembly with a 1 hour fire resistance rating which outperforms (in terms of fire resistance rating) the traditional joint system. Test results from a screening test conducted at a nationally recognized test lab show that heat transfer to the unexposed side (as evidenced by temperature rise) takes longer with some elastomeric materials than with the traditional system (report is attached for information). Full scale ASTM E 119 tests are underway. The elastomeric systems have been in use for many years in residential environments because the use of a single component system makes application simpler. In recent years types of elastomeric compound have been developed which can meet the fire performance requirement needed to create 1 hour fire resistance rated assemblies. However, they cannot be used in applications where a 1 hour fire resistance rating is required, unless a change is made to the IBC.

The additional properties are also important for a successful elastomeric sealant to be able to meet the full range of needs of the drywall industry. Today, in residential construction, successful elastomeric sealants already are used to replace the typical "mud" and tape joint materials for residential wood stud framing. However, sealants that meet the fire resistance requirements should also provide great resistance to cracking if moderate movement should occur in the drywall – which is becoming a more prevalent problem than in decades past due to the growing use in the construction trade of fast-growth lumber, which is less dimensionally stable than the old-growth lumber that was prevalent in years or decades past. In order for a joint material to resist cracking successfully, it has been found that the more resilient the sealant, at low modulus, the better. It is important, given the tendency of drywall paper to tear or delaminate under stress, that a sealant exhibit a modulus not exceeding 20 psi, at 100% elongation. The lower the modulus the less adhesive stress is applied to the bond-line of the drywall/sealant interface when movement occurs and the less chance the drywall paper will fail. In order for an elastomeric sealant to successfully resist cracking for the longest possible period of time after installation, it is important for the sealant not to lose its initial elastomeric and low modulus properties over time. Thus, the modulus should remain the same even after weathering or aging. Sealants that are formulated with no plasticizers, which readily migrate from sealants that contain them and leave them relatively rigid and higher in modulus over time, are far superior and are able to perform over many years without failure. It is also likely that the common plasticizers used in elastomeric sealants make those sealants less fire resistant because such plasticizers are low molecular weight organic oils that readily burn.

It has been reported that numerous drywall contractors around the US have used low modulus, high performance latex sealants for several years to seal the joints in drywall. This has been done by those contractors to prevent the kind of cracking they have otherwise experienced when they have used the traditional joint tape and mud in many situations where relatively extreme shrinkage movement has occurred in the underlying framing lumber. Now that it is possible to provide not only crack resistance but also fire resistance in such a low modulus, high performance sealant, the drywall finishing trade has a new means of providing high quality drywall finishing, with no compromise in fire safety.

The new referenced standards are: ASTM C 920, Standard Specification for Elastomeric Joint Sealants, and ASTM C 1523, Standard Test Method for Determining Modulus, Tear and Adhesion Properties of Precured Elastomeric Joint Sealants. The ASTM C 920 specification does not include a test method for modulus, which is critical for long-term performance. The ASTM C 1523 test method contains the test method for modulus as well as a weathering test method, which needs to be used to assess whether the modulus is still suitably high after aging of the assembly. ASTM C 1442 (weathering practice) and ASTM C 717 (terminology) are also attached for information.

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

Public Hearing Results

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

Analysis: Review of proposed new standard indicated that, in the opinion of ICC staff, the standard did comply with ICC criteria for referenced standards.

Committee Action: Disapproved

Committee Reason: This proposal appears to be out of place in this section since this is not an exception where "joint treatment" is not required but is instead an alternate product. Additionally the text addresses gypsum board being fastened with these materials. Item 2.3 would appear to be more appropriate for determining the acceptance under the standard and does not seem to be needed within the code. The committee did recognize that this is somewhat of a "chicken or the egg" issue. This can not go into the code because there is no standard, but because the code does not address it, there is no standard developed to test it. While conceptually fine, this proposal would create confusion regarding which test and product are acceptable when testing. The proposal should be coordinated with Table 2506.2 so that a conflict does not develop with the existing code requirements for gypsum board.

Assembly Action: None

Public Comments

Individual Consideration Agenda

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

Public Comment:

Marcelo M. Hirschler, GBH International, representing Sashco, requests Approval as Modified by this public comment.

Replace proposal with the following:

TABLE 2506.2
GYPSUM BOARD MATERIALS AND ACCESSORIES

MATERIAL	STANDARD
Accessories for gypsum board	ASTM C 1047
Adhesives for fastening gypsum wallboard	ASTM C 557
Elastomeric joint sealants	ASTM C 920
Exterior soffit board	ASTM C 931
Fiber-reinforced gypsum panels	ASTM C 1278
Glass mat gypsum backing panel	ASTM C 1178
Glass mat gypsum substrate	ASTM C 1188
Gypsum backing board and gypsum shaftliner board	ASTM C 442
Gypsum ceiling board	ASTM C 1395
Gypsum sheathing	ASTM C 79
Gypsum wallboard	ASTM C 36
Joint reinforcing tape and compound	ASTM C 474; C 475
Nails for gypsum boards	ASTM C 514, F 547, F 1667
Predecorated gypsum board	ASTM C 960
Steel screws	ASTM C 954, C 1002
Steel studs, load bearing	ASTM C 955
Steel studs, nonload bearing	ASTM C 645
Standard specification for gypsum board	ASTM C 1396
Testing gypsum and gypsum products	ASTM C 22, C 472, C 473
Water-resistant gypsum backing board	ASTM C 630

Add standard to Chapter 35 as follows:

ASTM
C 920-05 Standard Specification for Elastomeric Joint Sealants

Commenter=s Reason: As explained in the original proposal, “elastomeric joint sealant materials” exist now to seal gypsum board assemblies, replacing the traditional “joint reinforcing tape and compound”. Some of those assemblies have been tested and meet the 1 hour fire resistance rating usually considered necessary for gypsum board assemblies. In fact, tests were conducted at Southwest Research Institute on two gypsum board assemblies and they showed that the assembly tested with the joints sealed with the elastomeric joint sealant material performed at least as well as the one tested with the traditional joint reinforcing tape and compound. The assembly with the elastomeric joint sealant material achieved a 1 hour fire resistance rating (including passing the hose strean test). This comment proposes a modification to the proposal to do exactly what the technical committee tacitly recommended, namely adding a row to Table 2506.2, with the corresponding standard specification for elastomeric joint sealants (ASTM C 920). If the proposal is accepted as modified by this comment, the IBC will recognize the elastomeric joint sealant materials as an acceptable gypsum board accessory material. The use of the material will then have to comply with the appropriate requirements in other parts of the code. ICC staff analysis already indicated at the proposal stage that the standard specification (ASTM C 920) complies with the ICC criteria for referenced standards.

Final Hearing Results

S110-06/07 AMPC1

Code Change No: S112-06/07

Original Proposal

Sections: 2509.2; IRC R702.4.2

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: John Mulder, James Hardie Building Products, Inc.

PART I – IBC

Revise as follows:

2509.2 Base for tile. Glass mat gypsum, ~~Cement,~~ fiber-cement or glass mat gypsum backers in compliance with ASTM C 1178, C 1288 or C 1325 and installed in accordance with manufacturer recommendations shall be used as a base for wall tile in tub and shower areas and wall and ceiling panels in shower areas. Water-resistant gypsum backing board shall be used as a base for tile in water closet compartment walls when installed in accordance with GA-216 or ASTM C 840 and manufacturer recommendations. Regular gypsum wallboard is permitted under tile or wall panels in other wall and ceiling areas when installed in accordance with GA-216 or ASTM C 840.

PART II – IRC

Revise as follows:

R702.4.2. ~~Cement~~ Fiber-mat reinforced, fiber-cement and glass mat gypsum backers. ~~Cement~~ Fiber-mat reinforced, fiber-cement or glass mat gypsum backers in compliance with ASTM C 1325, C 1288, ~~C1325~~ or C 1178 and installed in accordance with manufacturers' recommendations shall be used as backers for wall tile in tub and shower areas and wall panels in shower areas.

Reason: The current Code language does not adequately describe the "cement" backer within the context of its published Standard definition: "fiber-mat reinforced products, n – manufactured thin section composites of hydraulic cementitious matrices and non-asbestos fibers in two-dimensional scrim(s)" [published definition in ASTM C 1154-02]. Additionally, the order of the list of compliance specifications does not coincide with the order of the recognized product listing.

Description of product per ASTM C 1325-04, Non-Asbestos Fiber-mat Reinforced Cement Substrate Sheets. Definition per ASTM C 1154-02, Standard Terminology for Fiber-Reinforced Cement Products.

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

Public Hearing Results

Errata: Revise Part I of code change to read as follows:

2509.2 Base for tile. Glass mat gypsum, ~~Cement,~~ fiber-cement or ~~glass mat gypsum~~ fiber-mat reinforced cement backers in compliance with ASTM C 1178, C 1288 or C 1325 and installed in accordance with manufacturer recommendations shall be used as a base for wall tile in tub and shower areas and wall and ceiling panels in shower areas. Water-resistant gypsum backing board shall be used as a base for tile in water closet compartment walls when installed in accordance with GA-216 or ASTM C 840 and manufacturer recommendations. Regular gypsum wallboard is permitted under tile or wall panels in other wall and ceiling areas when installed in accordance with GA-216 or ASTM C 840.

PART I — IBC Committee Action: Approved as Modified

Modify the proposal as follows:

2509.2 Base for tile. Glass mat water-resistant gypsum backing panels, discrete non-asbestos fiber-cement interior substrate sheets or non-asbestos fiber-mat reinforced cement substrate sheets ~~backers~~ in compliance with ASTM C 1178, C 1288 or C 1325 and installed in accordance with manufacturer recommendations shall be used as a base for wall tile in tub and shower areas and wall and ceiling panels in shower areas. Water-resistant gypsum backing board shall be used as a base for tile in water closet compartment walls when installed in accordance with GA-216 or ASTM C 840 and manufacturer recommendations. Regular gypsum wallboard is permitted under tile or wall panels in other wall and ceiling areas when installed in accordance with GA-216 or ASTM C 840.

Committee Reason: This code change provides clarity on the materials to be used as a base for tile. The modification makes the wording more consistent with the referenced standard for these materials.

Assembly Action: None

PART II — IRC
Committee Action: Disapproved

Committee Reason: Based upon the action on code change S11106/07, Part II.

Assembly Action: None

Final Hearing Results

S112-06/07, Part I	**AM**
S112-06/07, Part II	**D**

Code Change No: S113-06/07

Original Proposal

Table 1405.2, Table 2511.1.1, 2512.2, Table 2512.6, 2513.3

Proponent: Stephen V. Skalko, P.E., Portland Cement Association

Revise as follows:

TABLE 1405.2
MINIMUM THICKNESS OF WEATHER COVERINGS

COVERING TYPE	MINIMUM THICKNESS (INCHES)
Stucco or exterior ~~portland~~ cement plaster	

(Portions of table not shown do not change)

TABLE 2511.1.1
INSTALLATION OF PLASTER CONSTRUCTION

MATERIAL	STANDARD
Lathing and furring (cement plaster)	ASTM C 1063
~~Portland~~ Cement plaster	ASTM C 926
Steel framing	ASTM C 754; C 1007

(Portions of table not shown do not change)

2512.2 Plasticity agents. Only approved plasticity agents and approved amounts thereof shall be added to portland cement <u>or blended cements</u>. When plastic cement or masonry cement is used, no additional lime or plasticizers shall be added. Hydrated lime or the equivalent amount of lime putty used as a plasticizer is permitted to be added to cement plaster or cement and lime plaster in an amount not to exceed that set forth in ASTM C 926.

TABLE 2512.6
CEMENT PLASTERS[a]

c. Finish coat plaster is permitted to be applied to interior ~~portland~~ cement <u>plaster</u> base coats after a 48-hour period.

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

2513.3 Bedding coat proportions. The bedding coat for interior or exterior surfaces shall be composed of one-part portland cement~~,~~ <u>and</u> one-part Type S lime<u>; or one-part blended cement and one-part Type S lime; or masonry cement; or plastic cement,</u> and a maximum of three parts of graded white or natural sand by volume. The bedding coat for interior surfaces shall be composed of 100 pounds (45.4 kg) of neat gypsum plaster and a maximum of 200 pounds (90.8 kg) of graded white sand. A factory-prepared bedding coat for interior or exterior use is permitted. The bedding coat for exterior surfaces shall have a minimum compressive strength of 1,000 pounds per square inch (psi) (6895 kPa).

Reason: This proposal is to provide consistency within Chapters 14 and 25 of the IBC on the use of cements for interior and exterior plaster (stucco) work. The changes can be summarized as follows:

The word portland is being deleted from Table 1405.2 for type of weathering covering since cement plaster is a defined term in Chapter 25 and can be comprised of portland cement, blended cement, masonry cement and plastic cement.

The word portland is being deleted from Table 2511.1 and Footnote (c) to Table 2512.6 for the type of plaster since cement plaster is a defined term in Chapter 25 and can be comprised of portland cement, blended cement, masonry cement and plastic cement.

ASTM C926 also permits blended cements to be used in cement plaster mixes in combination with plasticity agents. This change adds these acceptable cementitious materials to the list of cementitious materials regulated by Section 2512.2

ASTM C926 also permits blended cements, masonry cements and plastic cements to be used in cement plaster mixes. This change adds these acceptable cementitious materials to the list of cementitious materials regulated by Section 2513.3

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This code change improves the terminology for cement plaster provisions by making it consistent with ASTM C926.

Assembly Action: **None**

Final Hearing Results

S113-06/07 **AS**

Code Change No: S114-06/07

Original Proposal

Section: 2512.1

Proponent: Stephen V. Skalko, P.E., Portland Cement Association

Revise as follows:

2512.1 General. Plastering with cement plaster shall be not less than three coats when applied over metal lath or wire fabric lath <u>or gypsum board backing as specified in Section 2510.5</u> and shall be not less than two coats when applied over masonry~~,~~ <u>or</u> concrete ~~or gypsum board backing as specified in Section 2510.5~~. If the plaster surface is to be completely covered by veneer or other facing material, or is completely concealed by another wall, plaster application need only be two coats, provided the total thickness is as set forth in ASTM C 926.

Reason: ASTM C 926 specifies that two-coat plaster is only to be used over surfaces of solid bases that are rigid such as masonry, stone or concrete. Three-coat finishes are intended to be applied over other less rigid bases such as gypsum board, wood or rigid foam-board type products. This proposal makes the code consistent with ASTM C926.

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This change makes the number of cement plaster coats that are required over gypsum backing board consistent with ASTM C 926.

Assembly Action: **None**

Final Hearing Results

S114-06/07 **AS**

Code Change No: S115-06/07

Original Proposal

Chapter 35

Proponent: Standards Writing Organization

Revise standards as follows:

AA

The Aluminum Association
1525 Wilson Blvd, Suite 600
Arlington, VA 22209

Standard reference number	Title
ADM1 2005 ~~00~~	Aluminum Design Manual: Part I-A Specification for Aluminum Structures - Allowable Stress Design; and Part I-B Specification for Aluminum Structures - Building Load and Resistance Factor Design

APA

APA-Engineered Wood Association
P. O. Box 11700
Tacoma,WA 98411-0700

Standard reference number	Title
EWS R540-02 96	Builders Tips: Proper Storage and Handling of Glulam Beams
EWS-T300-05 ~~02~~	Glulam Connection Details
EWS X440-03 ~~00~~	Product Guide - Glulam

ASTM

ASTM International
100 Barr Harbor Drive
West Conshohocken, PA 19428-2959

Standard reference number	Title
A 6/A 6M-05a ~~04a~~	Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling
A 36/A 36M-05 ~~04~~	Specification for Carbon Structural Steel
A 82/A 2M-05a ~~02~~	Specification for Steel Wire, Plain, for Concrete Reinforcement
A 153/A153M-05 ~~03~~	Specification for Zinc Coating (Hot Dip) on Iron and Steel Hardware
A 185/A 185M-05a ~~02~~	Specification for Steel Welded and Seamless Steel Pipe Piles
A 307-04~~03~~	Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength
A 421/A 421M-05 ~~02~~	Specification for Uncoated Stress-relieved Steel Wire for Prestressed Concrete
A 480A/A 480M~~–~~05 ~~02~~	Specification for General Requirements for Flt-rolled Stainless and Heat-resisting Steel Plate, Sheet and Strip
A 496/A 496M-05 ~~02~~	Specification for Steel Wire, Deformed for Concrete Reinforcement
A 497 A 497M- 05a ~~01~~	Specification for Steel Welded Reinforcement Deformed, for Concrete
A 568/A 568M-05a ~~03~~	Specification for Steel, Sheet, Carbon, and High-Strength, Low-Allow, Hot-rolled and Cold-rolled, General Requirements for
A 588/A 588M-05 ~~04~~	Specification for High-strength Low-allow Structural Steel with 50 ksi (345 Mpa) Minimum Yield Point to 4 inches (100mm) Thick
A 615/A 615M-05a ~~04a~~	Specification for Deformed and Plain Billet-steel Bars for Concrete Reinforcement
A 690/A 690M-05 ~~00a~~	Standard Specification for High Strength Low-allow Steel H-Piles and Sheet Piling for Use in Marine Environments

A 706/A 706M-05a ~~04a~~	Specification for Low-alloy Steel Deformed and Plain Bars for Concrete Reinforcement
A 722/A 722M-05 ~~98(2003)~~	Specification for Uncoated High-strength Steel Bar for Prestressing Concrete
A 767/A 767M-05 ~~00b~~	Specification for Zinc-coated (Galvanized) Steel Bars for Concrete Reinforcement
A 775/A 775M-04a	Specification for Epoxy-coated Steel Reinforcing Bars
A 884-04 ~~02~~	Specification for Epoxy-coated Steel Wire and Welded Wire Fabric for Reinforcement
A 992/A 992M-04a	Standard Specification for Structural Shapes
A 996/A 996M-05a ~~04~~	Specification for Rail-steel and Axle-steel Deformed Bars for Concrete Reinforcement
A 1008/A 1008M-05b ~~04b~~	Specification for Steel, Sheet, Cold-rolled, Carbon, Structural, High-strength Low-allow and High-strength Low-alloy with Improved Formability
B 695-04 ~~00~~	Standard Specification for Coatings of Zinc Mechanically Deposited on Iron and Steel
C 22/C-22M-00(2005)e01	Specification for Gypsum
C 28/C-28M-00~~e01~~(2005)	Specification for Gypsum Plasters
C 31/31M-03a ~~98~~	Practice for Making and Curing concrete Test Specimens in the Field
C 35-01(2005 ~~4~~) ~~95~~	Specification for Inorganic Aggregates for Use in Gypsum Plaster
C 56-~~96(2001)~~ 05	Specification for Structural Clay Non-Load-Bearing Tile
C 62-05 ~~041~~	Specification for Building Brick (Solid Masonry Units Made From Clay or Shale)
C 67-05 ~~03ae01~~	Test Methods of Sampling and Testing Brick and Structural Clay Tile
C 90-06 ~~03a~~	Specification for Loadbearing Concrete Masonry Units
C 91-05 ~~03a~~	Specification for Masonry Cement
C 109/C 109M-05 ~~02~~	Standard Test Method for Compressive Strength of Hydraulic Cement Mortars (Using 2-in. or [50-mm] Cube Specimens)
C 126-99(2005)	Specification for Ceramic Glazed Structural Clay Facing Tile, Facing Brick, and Solid Masonry Units
C 150-05 ~~04a~~	Specification for Portland Cement
C 199-84 (2005 ~~0~~)	Test Method for Pier Test for Refractory Mortars
C 207-06 ~~04~~	Specification for Hydrated Lime for Masonry Purposes
C 216-05a ~~04a~~	Specification for Facing Brick (Solid Masonry Units Made From Clay or Shale)
C 270-05a ~~04~~	Specification for Mortar for Unit Masonry
C 317/C 317M-00(2005)	Specification for Gypsum Concrete
C 473-05 ~~03~~	Test Methods for Physical Testing of Gypsum Panel Products
C 503-05 ~~03~~	Specification for Marble Dimension Stone (Exterior)
C 578-05a ~~04~~	Standard Specification for Rigid, Cellular Polystyrene Thermal Insulation
C 587-04 ~~02~~	Specification for Gypsum Veneer Plaster
C 595-05 ~~03~~	Specification for Blended Hydraulic Cements
C 631-95a(200~~0~~ 4)	Specification for Bonding Compounds for Interior Gypsum Plastering
C 645-04a	Specification for Nonstructural Steel Framing Members
C 652-05a ~~04a~~	Specification for Hollow Brick (Hollow Masonry Units Made from Clay or Shale
C 744-05 ~~99~~	Specification for Prefaced Concrete and Calcium Silicate Masonry Units
C 840-05 ~~04~~	Specification for Application and Finishing of Gypsum Board
C 844-04 ~~99~~	Specification for Application of Gypsum Base to Receive Gypsum Veneer Plaster
C 887-05 ~~79a(2001)~~	Specification for Packaged, Dry, Combined Materials for Surface Bonding Mortar
C 897-05 ~~00~~	Specification for Aggregate for Job-Mixed Portland Cement-Based Plasters
C 926-98a(2005)	Specification for Application of Portland Cement-Based Plaster
C 932-05 ~~03~~	Specification for Surface-Applied Bonding Agents for Exterior Plastering

C 933-05 ~~04~~	Specification for Welded Wire Lath
C 954-04 ~~00~~	Specification for Steel Drill Screws for the Application of Gypsum Panel Products or Metal Plaster Bases to Steel Studs from 0.033 inch (0.84 mm) to 0.112 inch (2.84 mm) in Thickness
C 1002-04 ~~01~~	Specification for Steel Self-Piercing Tapping Screws for the Application of Gypsum Panel Products or Metal Plaster Bases to Wood Studs or Steel Studs
C1019-05 ~~03~~	Test Method for Sampling and Testing Grout
C 1047-05 ~~99~~	Specification for Accessories for Gypsum Wallboard and Gypsum Veneer Base
C 1072-05b ~~00a~~	Standard Text Method for Measurement of Masonry Flexural Bond Strength
C 1177/C 1177M-04e01	Specification for Glass Mat Gypsum Substrate for Use as Sheathing
C 1178/C 1178M-04e01	Specification for Glass Mat Water-Resistant Gypsum Backing Panel
C 1261-05 ~~04~~	Specification for Firebox Brick for Residential Fireplaces
C 1278/C 1278M-03e01	Specification for Fiber-Reinforced Gypsum Panels
C 1288-99(2004) ~~01~~	Standard Specification for Discrete Non-Asbestos Fiber-Cement Interior Substrate Sheets
C 1328-05 ~~03a~~	Specification for Plastic (Stucco Cement)
C 1329-05 ~~04~~	Specification for Mortar Cement
C 1395/C 139~~5~~6M-04	Specification for Gypsum Ceiling Board
C 1405-00a05a	Standard Specification for Glazed Brick (Single Fired, Solid Brick Units)
D 25-99~~e01~~(2005)	Specification for Round Timber Piles
D 1557-02e01	Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lb/ft3(2,700kN-m/m3))
D 2166-00e01	Test Method for Unconfined Compressive Strength of Cohesive Soil
D 2216-05 ~~98~~	Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
D 2843-99(2004)e01	Test for Density of Smoke from the Burning or Decomposition of Plastics
D 3200-74(2005)	Standard Specification and Test Method for Establishing Recommended Design Stresses for Round Timber Construction Poles
D 3737-05 ~~03~~	Practice for Establishing Allowable Properties for Structural Glued Laminated Timber (Glulam)
D 4318-05 ~~00~~	Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
D 5055-05 ~~04~~	Specification for Establishing and Monitoring Structural Capacities of Prefabricated Wood I-Joists
D 5456-05a ~~03~~	Specification for Evaluation of Structural Composite Lumber Products
E 72-05 ~~02~~	Standard Test Methods of Conducting Strength Tests of Panels for Building Construction
E 605-93(2006) ~~00~~	Test Method for Thickness and Density of Sprayed Fire-Resistive Material (SFRM) Applied to Structural Members
E 736-00(2006)	Test Method for Cohesion/Adhesion of Sprayed Fire-Resistive Materials Applied to Structural Members
E 1886-05 ~~04~~	Test Method for Performance of Exterior Windows, Curtain Walls, Doors and Storm Shutters Impacted by Missiles and Exposed to Cyclic Pressure Differentials
E 1996-05b ~~04~~	Specification for Performance of Exterior Windows, Curtain Walls, Doors and Storm Shutters Impacted by Windborne Debris in Hurricanes

AWPA

American Wood-Preservers' Association
P. O. Box 361784
Birmingham, AL 35236-1784

Standard reference number	Title
M4-~~02~~ 06	Standard for Care of Preservative-Treated Wood Products
U1-~~04~~ 06	USE CATEGORY SYSTEM: User Specification for Treated wood except Section 7 Commodity Specification H

DOC

U.S. Department of Commerce
National Institute of Standards and Technology
100 Bureau Drive Stop 3460
Gaithersburg, MD 20899

Standard reference number	Title
PS 2-04 ~~95~~	Performance Standard for Wood-Based Structural-Use Panels

HPVA

Hardwood Plywood Veneer Association
1825 Michael Faraday Drive
Reston, VA 20190-5350

Standard reference number	Title
HP-1-2004 ~~2000~~	Standard for Hardwood and Decorative Plywood

UL

Underwriters Laboratories,Inc.
333 Pfingsten Road
Northbrook, IL 60062-2096

Standard reference number	Title
641-95	Type L Low-temperature Venting Systems-with Revisions through August 2005 ~~April 1999~~

Reason: The *ICC Code Development Process for the International Codes* (Procedures) Section 4.5* requires the updating of referenced standards to be accomplished administratively, and be processed as a Code Proposal. In May 2005, a letter was sent to each developer of standards that are referenced in the I-Codes, asking them to provide ICC with a list of their standards in order to update to the current edition. Above is the list received of the referenced standards under the maintenance responsibility of the IRC Committee.

> ***4.5 Updating Standards:** The updating of standards referenced by the Codes shall be accomplished administratively by the appropriate code development committee in accordance with these full procedures except that multiple standards to be updated may be included in a single proposal.

Public Hearing Results

Errata: Add the following standard update

DOC
PS 20-~~99~~ 05 American Softwood Lumber Standard

Committee Action: **Approved as Modified**

Modify the proposal as follows:

ASTM

ASTM International100 Barr Harbor DriveWest Conshohocken, PA 19428-2959Standard reference number Title
A 615/A 615M-~~05a~~ Specification for Deformed and Plain Billet04a steel Bars for Concrete Reinforcement
A 706/A 706M-~~05a~~ Specification for Low-alloy Steel Deformed and 04a Plain Bars for Concrete Reinforcement

(Portions of proposal not shown remain unchanged)

Committee Reason: The proposal makes necessary updates to existing referenced standards. The modification retains the current edition of ASTM standards that are also referenced by ACI 318 for consistency with that standard.

Assembly Action: **None**

Final Hearing Results

S115-07/07 **AM**

Code Change No: S116-06/07

Original Proposal

Sections: 1607.11.2, 1613.6.1

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

Revise as follows:

1. 1607.11.2 Reduction in roof live loads. The minimum uniformly distributed roof live loads, L_o, in Table 1607.1 are permitted to be reduced ~~according to the following provisions~~ in accordance with Section 1607.11.2.1 or 1607.11.2.2.

1607.11.2.1 Flat, pitched and curved roofs. No change to text.

1607.11.2.2 Special-purpose roofs. No change to text.

~~1607.11.2.3~~ 1607.11.3 Landscaped roofs. No change to text.

~~1607.11.2.4~~ 1607.11.4 Awnings and canopies. No change to text.

2. 1613.6.1 Assumption of flexible diaphragm. Add the following text at the end of Section 12.3.1.1 of ASCE 7.

Diaphragms constructed of wood structural panels or untopped steel decking shall also be permitted to be idealized as flexible, provided all of the following conditions are met:

1. Toppings of concrete or similar materials are not placed over wood structural panel diaphragms except for nonstructural toppings no greater than 1-1/2 inches (38 mm) thick.
2. Each line of vertical elements of the ~~lateral~~ seismic-force-resisting system complies with the allowable story drift of Table 12.12-1.
3. Vertical elements of the ~~lateral~~ seismic-force-resisting system are light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets.
4. Portions of wood structural panel diaphragms that cantilever beyond the vertical elements of the lateral-force-resisting system are designed in accordance with Section 2305.2.5 of the International Building Code.

Reason: Item 1 clarifies which provisions are intended to be referenced. Also, current Sections 1607.1.2.3 and 1607.11.2.4 do not contain provisions for the reduction of roof live loads and are renumbered accordingly. Item 2 rewording is needed because ASCE 7-05 does not define or use the term "lateral-force-resisting system."

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

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This code change makes editorial changes that improve the provisions for roof live load reductions as well as flexible diaphragms under seismic loads.

Assembly Action: **None**

Final Hearing Results

S116-06/07 **AS**

Code Change No: S117-06/07

Original Proposal

Table 2306.4.4

Errata: The following proposal was not published in the monograph:

Proponent: Louis Wagner, American Fiberboard Association

Delete Table 2306.4.4 and substitute as follows:

TABLE 2306.4.4ALLOWABLE SHEAR VALUES (plf) FOR WIND OR SEISMIC LOADING ON SHEAR WALLS OF FIBERBOARD SHEATHING BOARDCONSTRUCTION FOR TYPE V CONSTRUCTION ONLY [a,b,c,d,e]

THICKNESS AND GRADE	FASTENER SIZE	ALLOWABLE SHEAR VALUE (pounds per linear foot) nail spacing at panel edges (inches)a		
		4	3	2
1/2” or 25/32” Structural	No. 11 gage galvanized roofing nail 1-1/2” long for ½”, 1-3/4” for 25/32” with 3/8” head	170	230	260
	No. 16 gage galvanized staple, 7/16” crown f	150	200	225
	No. 16 gage galvanized staple, 1” crown f	220	290	325

For SI: 1 inch = 25.4 mm, 1 pound per foot = 14.5939 N/m

a. Fiberboard sheathing shall not be used to brace concrete or masonry walls.

b. Panel edges shall be backed with 2 inch or wider framing of Douglas fir-larch or Southern pine. For framing of other species: (1) Find specific gravity for species of framing lumber in AF&PA NDS. (2) For staples, multiply the shear value from the table above by 0.82 for species with specific gravity of 0.42 or greater, or 0.65 for all other species. (3) For nails, multiply the shear value from the table above by the following adjustment factor: Specific Gravity Adjustment Factor = [1-(0.5-SG)], where SG = Specific gravity of the framing lumber.

c. Values shown are for fiberboard sheathing on one side only with long panel dimension either parallel or perpendicular to studs. d Fastener shall be spaced 6 inches on center along intermediate framing members.

e. Values are not permitted in Seismic Design Category D, E, or F.

f. Staple length shall not be less than 1-1/2” for 25/32-inch sheathing or 1-1/4” for 1/2-inch sheathing.

Reason: This change incorporates revisions consistent with those implemented in the reference document SDPWS-05 for nailed fiberboard shear walls. Nailed values are based on requirements in ASTM C208 for fiberboard and test values in PFS Test Report #96-60 such that the minimum target ratio of test load to allowable load is 2.8. Test results for 2 inch edge nail spacing are adjusted to 3” and 4” edge nail spacing assuming load per nail for 2 inch edge nailing is 75% of that for less dense 3 inch and 4 inch edge nail patterns. The ratio of 75% is based on minimum requirements of ASTM C208 for 3 inch edge nail spacing. During a prior change submittal, cyclic data was not available for fiberboard shear walls. Cyclic testing has been conducted and results are reported in WMEL-2002-03 (see page 56). Results confirm adequacy in resisting cyclic loads as the ratio of cyclic to monotonic strength values were equivalent to those for wood structural panel control walls. Stapled values are derived from tests (PFS Test Report #96-60) in a consistent manner to nailed values such that the minimum target ratio of test load to allowable load will be 2.8. Values are not permitted for lateral resistance in Seismic Design Categories D, E, or F consistent with provisions of the SDPWS-05 for nailed fiberboard shear walls.

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

Bibliography:
Special Design Provisions for Wind and Seismic (SDPWS) 2005 Edition American Forest & Paper Association Available at http://www.awc.org/pdf/windsiesmicsupp.pdf Monotonic and Cyclic Tests of Shear Walls With Gypsum Wallboard, Fiberboard and Hardboard Siding Report No. WMEL-2002-03 Dolan and Toothman Available at www.fiberboard.org
PFS Test Report#96-60 Available at www.fiberboard.org

Public Hearing Results

Committee Action: **Approved as Submitted**

Committee Reason: This proposal provides updated allowable shear values for fiberboard sheathing that should be included since they are based on cyclic testing.

Assembly Action: **None**

Final Hearing Results

S117-06/07 **AS**