S55–06/07
1908.1.3 (New)

Proponent: James S. Lai, LAI Associates

Add new text as follows:

1908.1.3 ACI 318, section 14.8.4. Modify ACI 318, section 14.8.4 to read as follows:

14.8.4 – Δ_s maximum deflection due to service loads, including PΔ effects, shall not exceed $l_c/150$. At mid-height, Δ_s shall be calculated by:

$$\Delta_s = 0.67\Delta_{cr} + (M - 0.67M_{cr})(\Delta_n - 0.67\Delta_{cr}) \div (M_n - 0.67M_{cr});\quad \text{for } M > 0.67M_{cr} \qquad (14\text{-}8)$$

$$\Delta_s = 5\, M_s\, l_c^2 \div (48E_c\, I_g)\ ;\ \text{for } M \le 0.67M_{cr} \qquad (14\text{-}9)$$

Where

$\Delta_{cr} = 5(M_{cr})\, l_c^2 \div (48\, E_c I_g)$

$\Delta_n = 5(M_n)\, l_c^2 \div (48\, E_c I_{cr})$

M = Maximum unfactored moment due to service loads including PΔeffects

Reason: To replace part of code provisions referenced to ACI 318-05. This proposed change is in advance of code change under consideration by American Concrete Institute Committee 318. The original equations (14-8) and (14-9) significantly underestimate service load deflection in comparison to the original development of the alternate slender wall procedure. The corresponding code change, when approved by ACI, will not be published until 2008 and will delay further its inclusion in IBC until succeeding code cycle.

The original code development on alternate slender wall design was introduced into the 1987 UBC Supplement through efforts of SEAOC Building Code Committee. The provision was based on findings of Joint SCCACI- SEAOSC Task Committee on Slender Walls pursuant to full scale tests conducted in the early 1980's on twelve 4 feet wide by 24 feet high concrete wall panels of varying height to thickness ratios ranging from 30 to 60. [Refer to "Test Report on Slender Walls", aka "Green Book"]. The design procedure is predicated on control of out-of-plane deflection for serviceability under code prescribed forces in addition to required moment strength.

The ACI procedure was adopted by IBC 2000 and subsequent code editions. As quoted in ACI 318R-05 Commentary Section R14.8, Section 14.8 is based on the corresponding requirements in 1997 UBC and experimental research of the Test Report by SCCACI-SEAOSC. The ACI Commentary further alleged that the procedure, as prescribed in UBC, has been converted from working stress to factored load design. This could also imply that the ACI procedure as written is a direct conversion of UBC procedure.

Based on close examination of the original 1980 test data, lateral deflection increases rapidly when the moment exceeds 2/3 of M_{cr}, where M_{cr} is as defined in ACI 318. The analytical result follows closely with the bilinear load deflection characteristic. In the original development of the alternate slender wall design procedure, a linear interpolation between Δ_{cr} and Δ_n is permitted in obtaining Δ_s in order to simplify the slender wall panel design for M > $^2/_3$ M_{cr} in lieu of using more comprehensive analysis. It is important to note that the test results did not support the ACI $7.5\sqrt{f_c'}$ for modulus of rupture in any of the test panels. Also, the ACI procedure does not appear to correlate with the 1980 test results. ACI needs to improve its methodology in computing M_u and I_e so that computed results would follow a bilinear load deflection characteristic.

In computing service load deflection the current ACI procedure employs effective moment of inertia and a magnified moment for the combined moment due to lateral force and eccentric vertical load, also know as the P-Δ effect. Because the effective moment of inertia and magnified moment are dependent upon each other, some iteration is necessary. Furthermore, there has been question on the validity of using the Branson equation in calculating I_e for slender panel out-of-plane deflection calculation specifically when the ratio of I_g/I_{cr} exceeds 3. ACI procedure significantly under-estimates service load deflection in comparison to the original development of the alternate slender wall design procedure. For any given wall panel with reinforcement approaching the upper limit and with increase lateral force and/ or axial load, ACI procedure significantly under-estimates the service load deflection in comparison to the UBC procedure. Analytical studies have shown that calculation of service load deflection using ACI 318 equations (14-8) and (14-9) yields deflection below the service load deflection Δ_{cr} in a wide range of axial and lateral force combinations.

Based on a comprehensive comparative study by the 2005 Slender Wall Panel Task Group of the Structural Engineers Association of Southern California, this proposed code change modifies the current ACI 318 design procedure in the calculation of service load deflection and is a direct conversion of the 1997 UBC. ACI 318 defines M_{cr} at a modulus of rupture of $7.5\sqrt{f_c'}$. In order to effect a lower limit for slender wall design at $5\sqrt{f_c'}$, the permissible moment at cracking should be limited to 2/3 M_{cr}. The corresponding deflection can be set at 2/3 Δ_{cr}. This proposal effectively restores the original service load deflection limits since it was introduced and published in 1987 UBC Supplement through 1997 UBC.

Bibliography:

1. "UBC 97 and ACI 318-02 Code Comparison Summary Report," SEAOSC Slender Wall Task Group, SEAOSC, Whittier, CA, January 2006, (published as part of seminar notes.)
2. Mehran Pourzanjani, Design of Slender Walls – a comparative study of ACI vs UBC procedure," January, 2006 SEAOSC Seminar, SEAOSC, Whittier, CA.

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

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

S56–06/07
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 $0.75\varphi N_n$ and $0.75\varphi V_n$, where φ 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:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S57–06/07
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:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S58–06/07
2102, 2104.3 through 2104.4.3, Table 1704.5.1, Table 1704.5.3

Proponent: Phillip Samblanet, The Masonry Society/Jason J. Thompson, Masonry Alliance for Codes and Standards

1. Delete without substitution:

SECTION 2102
DEFINITIONS AND NOTATIONS

~~**MEAN DAILY TEMPERATURE.** The average daily temperature of temperature extremes predicted by a local weather bureau for the next 24 hours.~~

~~**2104.3 Cold weather construction.** The cold weather construction provisions of ACI 530.1/ASCE 6/TMS 602, Article 1.8 C, or the following procedures shall be implemented when either the ambient temperature falls below 40°F (4°C) or the temperature of masonry units is below 40°F (4°C).~~

~~**2104.3.1 Preparation.**~~

1. ~~Temperatures of masonry units shall not be less than 20°F (-7°C) when laid in the masonry. Masonry units containing frozen moisture, visible ice or snow on their surface shall not be laid.~~
2. ~~Visible ice and snow shall be removed from the top surface of existing foundations and masonry to receive new construction. These surfaces shall be heated to above freezing, using methods that do not result in damage.~~

~~**2104.3.2 Construction.** The following requirements shall apply to work in progress and shall be based on ambient temperature.~~

~~**2104.3.2.1Construction requirements for temperatures between 40°F (4°C) and 32°F (0°C).** The following construction requirements shall be met when the ambient temperature is between 40°F (4°C) and 32°F (0°C):~~

1. ~~Glass unit masonry shall not be laid.~~
2. ~~Water and aggregates used in mortar and grout shall not be heated above 140°F (60°C).~~
3. ~~Mortar sand or mixing water shall be heated to produce mortar temperatures between 40°F (4°C) and 120°F (49°C) at the time of mixing. When water and aggregates for grout are below 32°F(0°C), they shall be heated.~~

~~**2104.3.2.2 Construction requirements for temperatures between 32°F (0°C) and 25°F (-4°C).** The requirements of Section 2104.3.2.1 and the following construction requirements shall be met when the ambient temperature is between 32°F (0°C) and 25°F (-4°C):~~

1. ~~The mortar temperature shall be maintained above freezing until used in masonry.~~
2. ~~Aggregates and mixing water for grout shall be heated to produce grout temperature between 70°F (21°C) and 120°F (49°C) at the time of mixing. Grout temperature shall be maintained above 70°F (21°C) at the time of grout placement.~~
3. ~~Heat AAC masonry units to a minimum temperature of 40°F (4°C) before installing thin-bed mortar.~~

~~**2104.3.2.3 Construction requirements for temperatures between 25°F (-4°C) and 20°F (-7°C).** The requirements of Sections 2104.3.2.1 and 2104.3.2.2 and the following construction requirements shall be met when the ambient temperature is between 25°F (-4°C) and 20°F (-7°C):~~

1. ~~Masonry surfaces under construction shall be heated to 40°F (4°C).~~
2. ~~Wind breaks or enclosures shall be provided when the wind velocity exceeds 15 miles per hour (mph) (24 km/h).~~
3. ~~Prior to grouting, masonry shall be heated to a minimum of 40°F (4°C).~~

~~**2104.3.2.4 Construction requirements for temperatures below 20°F (-7°C).** The requirements of Sections 2104.3.2.1, 2104.3.2.2 and 2104.3.2.3 and the following construction requirement shall be met when the ambient temperature is below 20°F (-7°C): Enclosures and auxiliary heat shall be provided to maintain air temperature within the enclosure to above 32°F (0°C).~~

~~**2104.3.3 Protection.** The requirements of this section and Sections 2104.3.3.1 through 2104.3.3.5 apply after the masonry is placed and shall be based on anticipated minimum daily temperature for grouted masonry and anticipated mean daily temperature for ungrouted masonry.~~

~~**2104.3.3.1 Glass unit masonry.** The temperature of glass unit masonry shall be maintained above 40°F (4°C) for 48 hours after construction.~~

~~**2104.3.3.2 AAC masonry.** The temperature of AAC masonry shall be maintained above 32°F (0°C) for the first 4 hours after thin-bed mortar application.~~

~~**2104.3.3.3 Protection requirements for temperatures between 40°F (4°C) and 25°F (-4°C).** When the temperature is between 40°F (4°C) and 25°F (-4°C), newly constructed masonry shall be covered with a weather-resistive membrane for 24 hours after being completed.~~

~~**2104.3.3.4 Protection requirements for temperatures between 25°F (-4°C) and 20°F (-7°C).** When the temperature is between 25°F (-4°C) and 20°F (-7°C), newly constructed masonry shall be completely covered with weather-resistive insulating blankets, or equal protection, for 24 hours after being completed. The time period shall be extended to 48 hours for grouted masonry, unless the only cement in the grout is Type III portland cement.~~

~~**2104.3.3.5 Protection requirements for temperatures below20°F (-7°C).** When the temperature is below 20°F (-7°C), newly constructed masonry shall be maintained at a temperature above 32°F (0°C) for at least 24 hours after being completed by using heated enclosures, electric heating blankets, infrared lamps or other acceptable methods. The time period shall be extended to 48 hours for grouted masonry, unless the only cement in the grout is Type III portland cement.~~

~~**2104.4 Hot weather construction.** The hot weather construction provisions of ACI 530.1/ASCE 6/TMS 602, Article 1.8 D, or the following procedures shall be implemented when the temperature or the temperature and wind-velocity limits of this section are exceeded.~~

~~**2104.4.1 Preparation.** The following requirements shall be met prior to conducting masonry work.~~

~~**2104.4.1.1 Temperature.** When the ambient temperature exceeds 100°F (38°C), or exceeds 90°F (32°C) with a wind velocity greater than 8 mph (3.5 m/s):~~

1. ~~Necessary conditions and equipment shall be provided to produce mortar having a temperature below 120°F (49°C).~~
2. ~~Sand piles shall be maintained in a damp, loose condition.~~

~~**2104.4.1.2 Special conditions.** When the ambient temperature exceeds 115°F (46°C), or 105°F (40°C) with a wind velocity greater than 8 mph (3.5 m/s), the requirements of Section 2104.4.1.1 shall be implemented, and materials and mixing equipment shall be shaded from direct sunlight.~~

~~**2104.4.2 Construction.** The following requirements shall be met while masonry work is in progress.~~

~~**2104.4.2.1 Temperature.** When the ambient temperature exceeds 100°F (38°C), or exceeds 90°F (32°C) with a wind velocity greater than 8 mph (3.5 m/s):~~

1. ~~The temperature of mortar and grout shall be maintained below 120°F (49°C).~~
2. ~~Mixers, mortar transport containers and mortar boards shall be flushed with cool water before they come into contact with mortar ingredients or mortar.~~
3. ~~Mortar consistency shall be maintained by retempering with cool water.~~
4. ~~Mortar shall be used within 2 hours of initial mixing.~~
5. ~~Thin-bed mortar shall be spread no more than 4 feet (1219 mm) ahead of AAC masonry units.~~
6. ~~AAC masonry units shall be placed within one minute after spreading thin-bed mortar.~~

~~**2104.4.2.2 Special conditions.** When the ambient temperature exceeds 115°F (46°C), or exceeds 105°F (40°C) with a wind velocity greater than 8 mph (3.5 m/s), the requirements of Section 2104.4.2.1 shall be implemented and cool mixing water shall be used for mortar and grout. The use of ice shall be permitted in the mixing water prior to use. Ice shall not be permitted in the mixing water when added to the other mortar or grout materials.~~

~~**2104.4.3 Protection.** When the mean daily temperature exceeds 100°F (38°C) or exceeds 90°F (32°C) with a wind velocity greater than 8 mph (3.5 m/s), newly constructed masonry shall be fog sprayed until damp at least three times a day until the masonry is three days old.~~

(Renumber subsequent sections)

2. Revise table as follows:

TABLE 1704.5.1
LEVEL 1 SPECIAL INSPECTION

INSPECTION TASK	FREQUENCY OF INSPECTION		REFERENCE FOR CRITERIA		
	Continuous during task listed	Periodically during task listed	IBC Section	ACI 530/ASCE5/TMS 402[a]	ACI 530.1/ASCE 6/TMS 602[a]
2. The inspection program shall verify:					
e. Protection of masonry during cold weather (temperature below 40°F) or hot weather (temperature above 90°F).			~~Sec. 2104.3, 2104.4~~		Art. 1.8C, 1.8D

(Portions of table not shown do not change)

TABLE 1704.5.3
LEVEL 2 SPECIAL INSPECTION

INSPECTION TASK	FREQUENCY OF INSPECTION		REFERENCE FOR CRITERIA		
	Continuous during task listed	Periodically during task listed	IBC Section	ACI 530/ASCE5/TMS 402[a]	ACI 530.1/ASCE 6/TMS 602[a]
2. The inspection program shall verify:					
e. Protection of masonry during cold weather (temperature below 40°F) or hot weather (temperature above 90°F).			~~Sec. 2104.3, 2104.4~~		Art. 1.8C, 1.8D

(Portions of table not shown do not change)

Reason: This change removes transcribed provisions from the referenced standard ACI 530.1.1/ASCE 6/TMS 602, thus simplifying the IBC and avoiding the chance that the provisions in the IBC and the referenced standard vary unnecessarily.

Since the development of the IBC, the sponsoring organizations of the ACI 530.1/ASCE 6/TMS 602 have been concerned about portions of that standard that were transcribed into the IBC directly because the provisions in the IBC could begin to digress from those in the referenced standard. If this happened, two differing sets of requirements would exist, creating unnecessary confusion in the design, construction and compliance communities. However, at the request of the masonry industry, these provisions were maintained in the IBC.

During the development of the 2006 IBC however, concerns about maintaining accurate transcription were again raised, and considerable confusion was caused in trying to make sure the IBC matched the referenced standard. Because of this, and because designers, builders, and inspectors must have the referenced standard for other portions of masonry construction, the transcribed provisions are proposed for removal from the IBC.

The sections proposed for deletion are included directly in Articles 1.8 C and 1.8 D of ACI 530.1/ASCE 6/TMS 602..

In the future, updates to cold and hot weather construction requirements will be appropriately made and balloted in the proper consensus forum – that being the Masonry Standards Joint Committee, which is charged with updating the ACI 530.1/ASCE 6/TMS 602.

As noted, the IBC provisions in Section 2104.3 and 2104.4 are taken directly from Articles 1.8 C and 1.8 D of the ACI 530.1/ASCE 6/TMS 602. The Commentary to ACI 530.1/ASCE 6/TMS 602 provides substantiation to the provisions that are in Articles 1.8 C and 1.8 D of that standard, and which currently exist in IBC Sections 2104.3 and 2104.4.

Cost Impact: The code change proposal will not increase the cost of construction because no design, construction, or compliance requirements have changed.

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

S59–06/07

2109.1.1, 2109.2 through 2109.2.2, 2109.2.3, 2109.3 through 2109.7.4, 2109.8

Proponent: Phillip Samblanet, The Masonry Society and Jason J. Thompson, Masonry Alliance for Codes and Standards

1. Revise as follows:

2109.1 General. Empirically designed masonry shall conform to this chapter or Chapter 5 of ACI 530/ASCE 5/TMS 402.

2109.1.1 Limitations. The use of empirical design of masonry shall be limited ~~as follows:~~ <u>in accordance with Section Section 5.1.2 of ACI 530/ASCE 5/TMS 402.</u>

1. ~~Empirical design shall not be used for buildings assigned to Seismic Design Category D, E or F as specified in Section 1613, nor for the design of the seismic-force-resisting system for buildings assigned to Seismic Design Category B or C.~~
2. ~~Empirical design shall not be used for masonry elements that are part of the lateral-force-resisting system where the basic wind speed exceeds 110 mph (79 m/s).~~
3. ~~Empirical design shall not be used for interior masonry elements that are not part of the lateral-force- resisting system in buildings other than enclosed buildings as defined in Chapter 6 of ASCE 7 in:~~
   - ~~3.1. Buildings over 180 feet (55 100 mm) in height.~~
   - ~~3.2. Buildings over 60 feet (18 400 mm) in height where the basic wind speed exceeds 90 mph (40 m/s).~~
   - ~~3.3. Buildings over 35 feet (10 700 mm) in height where the basic wind speed exceeds 100 mph (45 m/s).~~
   - ~~3.4. Where the basic wind speed exceeds 110 mph (79 m/s).~~
4. ~~Empirical design shall not be used for exterior masonry elements that are not part of the lateral-force- resisting system and that are more than 35 feet (10 700 mm) above ground:~~
   - ~~4.1. Buildings over 180 feet (55 100 mm) in height.~~
   - ~~4.2. Buildings over 60 feet (18 400 mm) in height where the basic wind speed exceeds 90 mph (40 m/s).~~
   - ~~4.3. Buildings over 35 feet (10 700 mm) in height where the basic wind speed exceeds 100 mph (45 m/s).~~
5. ~~Empirical design shall not be used for exterior masonry elements that are less than or equal to 35 feet (10 700 mm) above ground where the basic wind speed exceeds 110 mph (79 m/s).~~
6. ~~Empirical design shall only be used when the resultant of gravity loads is within the center third of the wall thickness and within the central area bounded by lines at one-third of each cross-sectional dimension of foundation piers.~~
7. ~~Empirical design shall not be used for AAC masonry.~~

In buildings that exceed one or more of the ~~above~~ limitations in Section 5.1.2 of ACI 530/ASCE 5/TMS 402, masonry shall be designed in accordance with the engineered design provisions of Section 2107 or 2108 or the foundation wall provisions of Section 1805.5.

2. Delete without substitution:

~~2109.2 Lateral stability.~~

~~2109.2.1 Shear walls.~~ ~~Where the structure depends upon masonry walls for lateral stability, shear walls shall be provided parallel to the direction of the lateral forces resisted.~~

~~2109.2.1.1 Cumulative length of shear walls.~~ ~~In each direction in which shear walls are required for lateral stability, shear walls shall be positioned in two separate planes. The minimum cumulative length of shear walls provided shall be 0.4 times the long dimension of the building. Cumulative length of shear walls shall not include openings or any element with a length that is less than one-half its height.~~

~~2109.2.1.2 Maximum diaphragm ratio.~~ ~~Masonry shear walls shall be spaced so that the length-to-width ratio of each diaphragm transferring lateral forces to the shear walls does not exceed the values given in Table 2109.2.1.2.~~

~~TABLE 2109.2.1.2~~
~~DIAPHRAGM LENGTH-TO-WIDTH RATIOS~~

~~2109.2.2 Roofs.~~ ~~The roof construction shall be designed so as not to impart out-of-plane lateral thrust to the walls under roof gravity load.~~

3. Revise as follows:

~~2109.2.3~~ 2109.2 Surface-bonded walls. Dry-stacked, surface- bonded concrete masonry walls shall comply with the requirements of Chapter 5 of ACI 530/ASCE 5/TMS 402~~this code for masonry wall construction~~, except where otherwise noted in this section.

(Renumber subsequent sections)

4. Delete without substitution:

~~2109.3 Compressive stress requirements.~~

~~2109.3.1 Calculations.~~ ~~Compressive stresses in masonry due to vertical dead plus live loads, excluding wind or seismic loads, shall be determined in accordance with Section 2109.3.2.1. Dead and live loads shall be in accordance with Chapter 16, with live load reductions as permitted in Section 1607.9.~~

~~**2109.3.2 Allowable compressive stresses.** The compressive stresses in masonry shall not exceed the values given in Table 2109.3.2. Stress shall be calculated based on specified rather than nominal dimensions.~~

~~**2109.3.2.1 Calculated compressive stresses.** Calculated compressive stresses for single wythe walls and for multiwythe composite masonry walls shall be determined by dividing the design load by the gross cross-sectional area of the member. The area of openings, chases or recesses in walls shall not be included in the gross cross-sectional area of the wall.~~

~~**2109.3.2.2 Multiwythe walls.** The allowable stress shall be as given in Table 2109.3.2 for the weakest combination of the units used in each wythe.~~

~~**2109.4 Lateral support.**~~

~~**2109.4.1 Intervals.** Masonry walls shall be laterally supported in either the horizontal or vertical direction at intervals not exceeding those given in Table 2109.4.1.~~

~~**TABLE 2109.4.1**~~
~~**WALL LATERAL SUPPORT REQUIREMENTS**~~

~~**2109.4.2 Thickness.** Except for cavity walls and cantilever walls, the thickness of a wall shall be its nominal thickness measured perpendicular to the face of the wall. For cavity walls, the thickness shall be determined as the sum of the nominal thicknesses of the individual wythes. For cantilever walls, except for parapets, the ratio of height-to-nominal thickness shall not exceed 6 for solid masonry or 4 for hollow masonry. For parapets, see Section 2109.5.4.~~

~~**2109.4.3 Support elements.** Lateral support shall be provided by crosswalls, pilasters, buttresses or structural frame members when the limiting distance is taken horizontally, or by floors, roofs acting as diaphragms or structural frame members when the limiting distance is taken vertically.~~

~~**2109.5 Thickness of masonry.** Minimum thickness requirements shall be based on nominal dimensions of masonry.~~

~~**2109.5.1 Thickness of walls.** The thickness of masonry walls shall conform to the requirements of Section 2109.5.~~

~~**2109.5.2 Minimum thickness.**~~

~~**2109.5.2.1 Bearing walls.** The minimum thickness of masonry bearing walls more than one story high shall be 8 inches (203 mm). Bearing walls of one-story buildings shall not be less than 6 inches (152 mm) thick.~~

~~**2109.5.2.2 Rubble stone walls.** The minimum thickness of rough, random or coursed rubble stone walls shall be 16 inches (406 mm).~~

~~**2109.5.2.3 Shear walls.** The minimum thickness of masonry shear walls shall be 8 inches (203 mm).~~

~~**2109.5.2.4 Foundation walls.** The minimum thickness of foundation walls shall be 8 inches (203 mm) and as required by Section 2109.5.3.1.~~

~~**TABLE 2109.3.2**~~
~~**ALLOWABLE COMPRESSIVE STRESSES FOR EMPIRICAL DESIGN OF MASONRY**~~

~~**2109.5.2.5 Foundation piers.** The minimum thickness of foundation piers shall be 8 inches (203 mm).~~

~~**2109.5.2.6 Parapet walls.** The minimum thickness of parapet walls shall be 8 inches (203 mm) and as required by Section 2109.5.4.1.~~

~~**2109.5.2.7 Change in thickness.** Where walls of masonry of hollow units or masonry bonded hollow walls are decreased in thickness, a course or courses of solid masonry shall be interposed between the wall below and the thinner wall above, or special units or construction shall be used to transmit the loads from face shells or wythes above to those below.~~

~~**2109.5.3 Foundation walls.** Foundation walls shall comply with the requirements of Section 2109.5.3.1 or 2109.5.3.2.~~

~~**2109.5.3.1 Minimum thickness.** Minimum thickness for foundation walls shall comply with the requirements of Table 2109.5.3.1. The provisions of Table 2109.5.3.1 are only applicable where the following conditions are met:~~

1. ~~The foundation wall does not exceed 8 feet (2438 mm) in height between lateral supports;~~
2. ~~The terrain surrounding foundation walls is graded to drain surface water away from foundation walls;~~

3. ~~Backfill is drained to remove ground water away from foundation walls;~~
4. ~~Lateral support is provided at the top of foundation walls prior to backfilling;~~
5. ~~The length of foundation walls between perpendicular masonry walls or pilasters is a maximum of three times the basement wall height;~~
6. ~~The backfill is granular and soil conditions in the area are nonexpansive; and~~
7. ~~Masonry is laid in running bond using Type M or S mortar.~~

~~TABLE 2109.5.3.1~~
~~FOUNDATION WALL CONSTRUCTION~~

~~2109.5.3.2 Design requirements.~~ ~~Where the requirements of Section 2109.5.3.1 are not met, foundation walls shall be designed in accordance with Section 1805.5.~~

~~2109.5.4 Parapet walls.~~

~~2109.5.4.1 Minimum thickness.~~ ~~The minimum thickness of unreinforced masonry parapets shall meet Section 2109.5.2.6 and their height shall not exceed three times their thickness.~~

~~2109.5.4.2 Additional provisions.~~ ~~Additional provisions for parapet walls are contained in Sections 1503.2 and 1503.3.~~

~~2109.6 Bond.~~

~~2109.6.1 General.~~ ~~The facing and backing of multiwythe masonry walls shall be bonded in accordance with Section 2109.6.2, 2109.6.3 or 2109.6.4.~~

~~2109.6.2 Bonding with masonry headers.~~

~~2109.6.2.1 Solid units.~~ ~~Where the facing and backing (adjacent wythes) of solid masonry construction are bonded by means of masonry headers, no less than 4 percent of thewall surface of each face shall be composed of headers extending not less than 3 inches (76 mm) into the backing. The distance between adjacent full-length headers shall not exceed 24 inches (610 mm) either vertically or horizontally. In walls in which a single header does not extend through the wall, headers from the opposite sides shall overlap at least 3 inches (76 mm), or headers from opposite sides shall be covered with another header course overlapping the header below at least 3 inches (76 mm).~~

~~2109.6.2.2 Hollow units.~~ ~~Where two or more hollow units are used to make up the thickness of a wall, the stretcher courses shall be bonded at vertical intervals not exceeding 34 inches (864 mm) by lapping at least 3 inches (76 mm) over the unit below, or by lapping at vertical intervals not exceeding 17 inches (432 mm) with units that are at least 50 percent greater in thickness than the units below.~~

~~2109.6.2.3 Masonry bonded hollow walls.~~ ~~In masonry bonded hollow walls, the facing and backing shall be bonded so that not less than 4 percent of the wall surface of each face is composed of masonry bonded units extending not less than 3 inches (76 mm) into the backing. The distance between adjacent bonders shall not exceed 24 inches (610 mm) either vertically or horizontally.~~

~~2109.6.3 Bonding with wall ties or joint reinforcement.~~

~~2109.6.3.1 Bonding with wall ties.~~ ~~Except as required by Section 2109.6.3.1.1, where the facing and backing (adjacent wythes) of masonry walls are bonded with wire size W2.8 (MW18) wall ties or metal wire of equivalent stiffness embedded in the horizontal mortar joints, there shall be at least one metal tie for each 4 1/2 square feet (0.42 m~~^{2}~~) of wall area. The maximum vertical distance between ties shall not exceed 24 inches (610 mm), and the maximum horizontal distance shall not exceed 36 inches (914 mm). Rods or ties bent to rectangular shape shall be used with hollow masonry units laid with the cells vertical. In other walls, the ends of ties shall be bent to 90-degree (1.57 rad) angles to provide hooks no less than 2 inches (51 mm) long. Wall ties shall be without drips. Additional bonding ties shall be provided at all openings, spaced not more than 36 inches (914 mm) apart around the perimeter and within 12 inches (305 mm) of the opening.~~

~~2109.6.3.1.1 Bonding with adjustable wall ties.~~ ~~Where the facing and backing (adjacent wythes) of masonry are bonded with adjustable wall ties, there shall be at least one tie for each 1.77 square feet (0.164 m~~^{2}~~) of wall area. Neither the vertical nor horizontal spacing of the adjustable wall ties shall exceed 16 inches (406 mm). The maximum vertical offset of bed joints from one wythe to the other shall be 1 1/4 inches (32 mm). The maximum clearance between connecting parts of the ties shall be 1/16 inch (1.6 mm). When pintle legs are used, ties shall have at least two wire size W2.8 (MW18) legs.~~

~~**2109.6.3.2 Bonding with prefabricated joint reinforcement.** Where the facing and backing (adjacent wythes) of masonry are bonded with prefabricated joint reinforcement, there shall be at least one cross wire serving as a tie for each 2 2/3 square feet (0.25m^2) ofwall area. The vertical spacing of the joint reinforcing shall not exceed 24 inches (610 mm). Cross wires on prefabricated joint reinforcement shall not be less than W1.7 (MW11) and shall be without drips. The longitudinal wires shall be embedded in the mortar.~~

~~**2109.6.4 Bonding with natural or cast stone.**~~

~~**2109.6.4.1 Ashlar masonry.** In ashlar masonry, bonder units, uniformly distributed, shall be provided to the extent of not less than 10 percent of the wall area. Such bonder units shall extend not less than 4 inches (102 mm) into the backing wall.~~

~~**2109.6.4.2 Rubble stone masonry.** Rubble stone masonry 24 inches (610 mm) or less in thickness shall have bonder units with a maximum spacing of 36 inches (914 mm) vertically and 36 inches (914 mm) horizontally, and if the masonry is of greater thickness than 24 inches (610 mm), shall have one bonder unit for each 6 square feet (0.56 m^2) of wall surface on both sides.~~

~~**2109.6.5 Masonry bonding pattern.**~~

~~**2109.6.5.1 Masonry laid in running bond.** Each wythe of masonry shall be laid in running bond, head joints in successive courses shall be offset by not less than one-fourth the unit length or the masonry walls shall be reinforced longitudinally as required in Section 2109.6.5.2.\~~

~~**2109.6.5.2 Masonry laid in stack bond.** Where unit masonry is laid with less head joint offset than in Section 2109.6.5.1, the minimum area of horizontal reinforcement placed in mortar bed joints or in bond beams spaced not more than 48 inches (1219 mm) apart, shall be 0.0003 times the vertical cross-sectional area of the wall.~~

~~**2109.7 Anchorage.**~~

~~**2109.7.1 General.** Masonry elements shall be anchored in accordance with Sections 2109.7.2 through 2109.7.4.~~

~~**2109.7.2 Intersecting walls.** Masonry walls depending upon one another for lateral support shall be anchored or bonded at locations where they meet or intersect by one of the methods indicated in Sections 2109.7.2.1 through 2109.7.2.5.~~

~~**2109.7.2.1 Bonding pattern.** Fifty percent of the units at the intersection shall be laid in an overlapping masonry bonding pattern, with alternate units having a bearing of not less than 3 inches (76 mm) on the unit below.~~

~~**2109.7.2.2 Steel connectors.** Walls shall be anchored by steel connectors having a minimum section of 1/4 inch (6.4 mm) by 1 1/2 inches (38 mm), with ends bent up at least 2 inches (51 mm) or with cross pins to form anchorage. Such anchors shall be at least 24 inches (610 mm) long and the maximum spacing shall be 48 inches (1219 mm).~~

~~**2109.7.2.3 Joint reinforcement.** Walls shall be anchored by joint reinforcement spaced at a maximum distance of 8 inches (203 mm). Longitudinal wires of such reinforcement shall be at least wire size W1.7 (MW 11) and shall extend at least 30 inches (762 mm) in each direction at the intersection.~~

~~**2109.7.2.4 Interior nonload-bearing walls.** Interior nonload-bearing walls shall be anchored at their intersection, at vertical intervals of not more than 16 inches (406 mm) with joint reinforcement or 1/4-inch (6.4 mm) mesh galvanized hardware cloth.~~

~~**2109.7.2.5 Ties, joint reinforcement or anchors.** Other metal ties, joint reinforcement or anchors, if used, shall be spaced to provide equivalent area of anchorage to that required by this section.~~

~~**2109.7.3 Floor and roof anchorage.** Floor and roof diaphragms providing lateral support to masonry shall comply with the live loads in Section 1607.3 and shall be connected to the masonry in accordance with Sections 2109.7.3.1 through 2109.7.3.3. Roof loading shall be determined in accordance with Chapter 16 and, when net uplift occurs, uplift shall be resisted entirely by an anchorage system designed in accordance with the provisions of Sections 2.1 and 2.3, Sections 3.1 and 3.3 or Chapter 4 of ACI 530/ASCE 5/TMS 402.~~

~~**2109.7.3.1Wood floor joists.** Wood floor joists bearing on masonry walls shall be anchored to the wall at intervals not to exceed 72 inches (1829 mm) by metal strap anchors. Joists parallel to the wall shall be anchored with metal straps spaced not more than 72 inches (1829 mm) o.c. extending over or under and secured to at least three joists. Blocking shall be provided between joists at each strap anchor.~~

~~**2109.7.3.2 Steel floor joists.** Steel floor joists bearing on masonry walls shall be anchored to the wall with 3/8-inch (9.5 mm) round bars, or their equivalent, spaced not more than 72 inches (1829 mm) o.c. Where joists are parallel to the wall, anchors shall be located at joist bridging.~~

~~**2109.7.3.3 Roof diaphragms.** Roof diaphragms shall be anchored to masonry walls with 1/2-inch-diameter (12.7 mm) bolts, 72 inches (1829 mm) o.c. or their equivalent. Bolts shall extend and be embedded at least 15 inches (381 mm) into the masonry, or be hooked or welded to not less than 0.20 square inch (129 mm2) of bond beam reinforcement placed not less than 6 inches (152 mm) from the top of the wall.~~

~~**2109.7.4 Walls adjoining structural framing.** Where walls are dependent upon the structural frame for lateral support, they shall be anchored to the structural members with metal anchors or otherwise keyed to the structural members. Metal anchors shall consist of 1/2-inch (12.7 mm) bolts spaced at 48 inches (1219 mm) o.c. embedded 4 inches (102 mm) into the masonry, or their equivalent area.~~

5. Revise as follows:

~~**2109.8**~~ **2109.3 Adobe construction.** Adobe construction shall comply with this section and shall be subject to the requirements of this code for Type V construction and Chapter 5 of ACI 530/ASCE 5/TMS 402.

(Renumber subsequent sections)

Reason: This change removes transcribed provisions from the referenced standard ACI 530/ASCE 5/TMS 402, thus simplifying the IBC and avoiding the chance that the provisions in the IBC and the referenced standard vary unnecessarily.

Since the development of the IBC, the sponsoring organizations of the ACI 530/ASCE 5/TMS 402 have been concerned about portions of that standard that were transcribed into the IBC directly because the provisions in the IBC could begin to digress from those in the referenced standard. If this happened, two differing sets of requirements would exist, creating unnecessary confusion in the design, construction and compliance communities. However, at the request of the masonry industry, these provisions were maintained in the IBC.

During the development of the 2006 IBC however, concerns about maintaining accurate transcription were again raised, and considerable confusion was caused in trying to make sure the IBC matched the referenced standard. Because of this, and because designers, builders, and inspectors must have the referenced standard for other portions of masonry construction, the transcribed provisions are proposed for removal from the IBC.

The sections proposed for deletion are included directly in Chapter 5 of ACI 530/SCE 5/ TMS 402. Requirements for Surface Bonded Masonry and Adobe masonry, which are not included in ACI 530/ASCE 5/TMS 402 are maintained in the IBC. No technical changes are being proposed by this Code Change Proposal.

In the future, updates to empirical requirements for masonry will be appropriately made and balloted in the proper consensus forum – that being the Masonry Standards Joint Committee, which is charged with updating the ACI 530/ASCE 5/TMS 402.

As noted, most of the IBC provisions in Section 2109 are taken directly from Chapter 5 of the ACI 530/ASCE 5/TMS 402. The Commentary to ACI 530/ASCE 5/TMS 402 provides substantiation to the provisions that are in Chapter 5 of that standard, and which currently exist in IBC Section 2109.

Cost Impact: The code change proposal will not increase the cost of construction because no design, construction, or compliance requirements have changed.

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

S60–06/07

2101.2.4, 2101.2.5, 2101.2.7, 2101.2.8, 2109, 2110 & 404.5

Proponents: Phillip Samblanet, The Masonry Society/Jason J. Thompson, Masonry Alliance for Codes and Standards.

1. Revise as follows:

2101.2.4 Empirical design. Masonry designed by the empirical design method shall comply with the provisions of ~~Sections 2106 and 2109 or~~ Chapter 5 of ACI 530/ASCE 5/TMS 402. In buildings that exceed one or more of the limitations in Section 5.1.2 of ACI 530/ASCE 5/TMS 402, masonry shall be designed in accordance with the engineered design provisions of Section 2101.2.1, 2101.2.2, 2101.2.3 or the foundation wall provisions of Section 1805.5.

2101.2.5 Glass masonry. Glass masonry shall comply with the provisions of ~~Section 2110 or with the requirements of~~ Chapter 7 of ACI 530/ASCE 5/TMS 402 and this section.

2101.2.5.1 Limitations. Solid or hollow approved glass block shall not be used in fire walls, party walls, fire barriers or fire partitions, or for load-bearing construction. Such blocks shall be erected with mortar and reinforcement in metal channel-type frames, structural frames, masonry or concrete recesses, embedded panel anchors as provided for both exterior and interior walls or other approved joint materials. Wood strip framing shall not be used inwalls required to have a fire-resistance rating by other provisions of this code.

Exceptions:

1. Glass-block assemblies having a fire protection rating of not less than 3/4 hour shall be permitted as opening protectives in accordance with Section 715 in fire barriers and fire partitions that have a required fire-resistance rating of 1 hour or less and do not enclose exit stairways or exit passageways.
2. Glass-block assemblies as permitted in Section 404.5, Exception 2.

2. Add new text as follows:

2101.2.7 Surface-bonded masonry. Dry-stacked, surface-bonded masonry shall comply with the provisions of Section 2109.

2101.2.8 Adobe masonry. Adobe masonry shall comply with the provisions of Section 2110.

3. Revise as follows:

SECTION 2109
~~EMPIRICAL DESIGN OF MASONRY~~ SURFACE-BONDED MASONRY

2109.1 General. ~~Empirically~~ ~~designed~~ Dry-stacked, surface-bonded masonry shall conform to ~~this chapter or~~ Chapter 5 of ACI 530/ASCE 5/TMS 402 except as modified in this section.

2109.1.1 Limitations. The use of ~~empirical design~~ of dry-stacked, surface-bonded masonry ~~design~~ shall be limited in accordance with Section 5.1.2 of ACI 530/ASCE 5/TMS 402. The use of dry-stacked, surface-bonded masonry shall be prohibited in structures assigned to Occupancy Category IV.

1. ~~Empirical design shall not be used for buildings assigned to Seismic Design Category D, E or F as specified in Section 1613, nor for the design of the seismic-force-resisting system for buildings assigned to Seismic Design Category B or C.~~
2. ~~Empirical design shall not be used for masonry elements that are part of the lateral-force-resisting system where the basic wind speed exceeds 110 mph (79 m/s).~~
3. ~~Empirical design shall not be used for interior masonry elements that are not part of the lateral force-resisting system in buildings other than enclosed buildings as defined in Chapter 6 of ASCE 7 in:~~
   - 3.1. ~~Buildings over 180 feet (55 100 mm) in height.~~
   - 3.2. ~~Buildings over 60 feet (18 400 mm) in height where the basic wind speed exceeds 90 mph (40 m/s).~~
   - 3.3. ~~Buildings over 35 feet (10 700 mm) in height where the basic wind speed exceeds 100 mph (45 m/s).~~
   - 3.4. ~~Where the basic wind speed exceeds 110 mph (79 m/s).~~
4. ~~Empirical design shall not be used for exterior masonry elements that are not part of the lateral force- resisting system and that are more than 35 feet (10 700 mm) above ground:~~
   - 4.1. ~~Buildings over 180 feet (55 100 mm) in height.~~
   - 4.2. ~~Buildings over 60 feet (18 400 mm) in height where the basic wind speed exceeds 90 mph (40 m/s).~~
   - 4.3. ~~Buildings over 35 feet (10 700 mm) in height where the basic wind speed exceeds 100 mph (45 m/s).~~
5. ~~Empirical design shall not be used for exterior masonry elements that are less than or equal to 35 feet (10 700 mm) above ground where the basic wind speed exceeds 110 mph (79 m/s).~~
6. ~~Empirical design shall only be used when the resultant of gravity loads is within the center third of the wall thickness and within the central area bounded by lines at one-third of each cross-sectional dimension of foundation piers.~~
7. ~~Empirical design shall not be used for AAC masonry. In buildings that exceed one or more of the above limitations, masonry shall be designed in accordance with the engineered design provisions of Section 2107 or 2108 or the foundation wall provisions of Section 1805.5.~~

In buildings that exceed one or more of the ~~above~~ limitations in Section 5.1.2 of ACI 530/ASCE 5/TMS 402, masonry shall be designed in accordance with the engineered design provisions of Section 2~~107 or 2108~~ 2101.2.1, 2101.2.2, 2101.2.3 or the foundation wall provisions of Section 1805.5.

4. Delete without substitution:

~~**2109.2 Lateral stability.**~~

~~**2109.2.1 Shear walls.** Where the structure depends upon masonry walls for lateral stability, shear walls shall be provided parallel to the direction of the lateral forces resisted.~~

~~**2109.2.1.1 Cumulative length of shear walls.** In each direction in which shearwalls are required for lateral stability, shear walls shall be positioned in two separate planes. The minimum cumulative length of shear walls provided shall be 0.4 times the long dimension of the building. Cumulative length of shear walls shall not include openings or any element with a length that is less than one-half its height.~~

~~**2109.2.1.2 Maximum diaphragm ratio.** Masonry shear walls shall be spaced so that the length-to-width ratio of each diaphragm transferring lateral forces to the shear walls does not exceed the values given in Table 2109.2.1.2.~~

~~**TABLE 2109.2.1.2**~~
~~**DIAPHRAGM LENGTH-TO-WIDTH RATIOS**~~

~~**2109.2.2 Roofs.** The roof construction shall be designed so as not to impart out-of-plane lateral thrust to the walls under roof gravity load.~~

~~**2109.2.3 Surface-bonded walls.** Dry-stacked, surface-bonded concrete masonry walls shall comply with the requirements of this code for masonry wall construction, except where otherwise noted in this section.~~

5. Revise as follows:

~~**2109.2.3.12**~~ **2109.2** **Strength.** Dry-stacked, surface-bonded concrete masonry walls shall be ... …listed in Table ~~2109.2.3.1~~ 2109.2. Allowable stresses not specified in Table ~~2109.2.3.1~~ 2109.2 shall comply with the requirements of Chapter 5 of ACI 530/ASCE 5/TMS 402.

TABLE ~~2109.2.3.1~~2109.2
ALLOWABLE STRESS GROSS CROSS-SECTIONAL
AREA FOR DRY-STACKED, SURFACE-BONDED
CONCRETE MASONRY WALLS

(No change to table entries)

~~**2109.2.3.2**~~ **2109.3** **Construction.** Construction of dry-stacked, surface-bonded masonry walls, including stacking and leveling of units, mixing and application of mortar and curing and protection shall comply with ASTM C 946.

6. Delete without substitution:

~~**2109.3 Compressive stress requirements.**~~

~~**2109.3.1 Calculations.** Compressive stresses in masonry due to vertical dead plus live loads, excluding wind or seismic loads, shall be determined in accordance with Section 2109.3.2.1. Dead and live loads shall be in accordance with Chapter 16, with live load reductions as permitted in Section 1607.9.~~

~~**2109.3.2 Allowable compressive stresses.** The compressive stresses in masonry shall not exceed the values given in Table 2109.3.2. Stress shall be calculated based on specified rather than nominal dimensions.~~

~~**2109.3.2.1 Calculated compressive stresses.** Calculated compressive stresses for single wythe walls and for multiwythe composite masonrywalls shall be determined by dividing the design load by the gross cross-sectional area of the member. The area of openings, chases or recesses in walls shall not be included in the gross cross-sectional area of the wall.~~

~~**2109.3.2.2 Multiwythe walls.** The allowable stress shall be as given in Table 2109.3.2 for theweakest combination of the units used in each wythe.~~

~~**2109.4 Lateral support.**~~

~~**2109.4.1 Intervals.** Masonry walls shall be laterally supported in either the horizontal or vertical direction at intervals not exceeding those given in Table 2109.4.1.~~

~~**TABLE 2109.4.1**~~
~~**WALL LATERAL SUPPORT REQUIREMENTS CONSTRUCTION MAXIMUM WALL LENGTH TO THICKNESS OR WALL HEIGHT TO THICKNESS**~~

~~**2109.4.2 Thickness.** Except for cavity walls and cantilever walls, the thickness of a wall shall be its nominal thickness measured perpendicular to the face of the wall. For cavity walls, the thickness shall be determined as the sum of the nominal thicknesses of the individual wythes. For cantilever walls, except for parapets, the ratio of height-to-nominal thickness shall not exceed 6 for solid masonry or 4 for hollow masonry. For parapets, see Section 2109.5.4.~~

~~**2109.4.3 Support elements.** Lateral support shall be provided by cross walls, pilasters, buttresses or structural frame members when the limiting distance is taken horizontally, or by floors, roofs acting as diaphragms or structural frame members when the limiting distance is taken vertically.~~

~~**2109.5 Thickness of masonry.** Minimum thickness requirements shall be based on nominal dimensions of masonry.~~

~~**2109.5.1 Thickness of walls.** The thickness of masonry walls shall conform to the requirements of Section 2109.5.~~

~~**2109.5.2 Minimum thickness.**~~

~~**2109.5.2.1 Bearing walls.** The minimum thickness of masonry bearing walls more than one story high shall be 8 inches (203 mm). Bearing walls of one-story buildings shall not be less than 6 inches (152 mm) thick.~~

~~**2109.5.2.2 Rubble stone walls.** The minimum thickness of rough, random or coursed rubble stone walls shall be 16 inches (406 mm).~~

~~**2109.5.2.3 Shear walls.** The minimum thickness of masonry shear walls shall be 8 inches (203 mm).~~

~~**2109.5.2.4 Foundation walls.** The minimum thickness of foundation walls shall be 8 inches (203 mm) and as required by Section 2109.5.3.1.~~

~~**TABLE 2109.3.2**~~
~~**ALLOWABLE COMPRESSIVE STRESSES FOR EMPIRICAL DESIGN OF MASONRY**~~

~~**2109.5.2.5 Foundation piers.** The minimum thickness of foundation piers shall be 8 inches (203 mm).~~

~~**2109.5.2.6 Parapet walls.** The minimum thickness of parapet walls shall be 8 inches (203 mm) and as required by Section 2109.5.4.1.~~

~~**2109.5.2.7 Change in thickness.** Where walls of masonry of hollow units or masonry bonded hollow walls are decreased in thickness, a course or courses of solid masonry shall be interposed between the wall below and the thinner wall above, or special units or construction shall be used to transmit the loads from face shells or wythes above to those below.~~

~~**2109.5.3 Foundation walls.** Foundationwalls shall comply with the requirements of Section 2109.5.3.1 or 2109.5.3.2.~~

~~**2109.5.3.1 Minimum thickness.** Minimum thickness for foundation walls shall comply with the requirements of Table 2109.5.3.1. The provisions of Table 2109.5.3.1 are only applicable where the following conditions are met:~~

1. ~~The foundation wall does not exceed 8 feet (2438 mm) in height between lateral supports;~~
2. ~~The terrain surrounding foundation walls is graded to drain surface water away from foundation walls;~~
3. ~~Backfill is drained to remove ground water away from foundation walls;~~
4. ~~Lateral support is provided at the top of foundation walls prior to backfilling;~~
5. ~~The length of foundation walls between perpendicular masonry walls or pilasters is a maximum of three times the basement wall height;~~
6. ~~The backfill is granular and soil conditions in the area are nonexpansive; and~~
7. ~~Masonry is laid in running bond using Type M or S mortar.~~

~~**TABLE 2109.5.3.1**~~
~~**FOUNDATION WALL CONSTRUCTION**~~

~~**2109.5.3.2 Design requirements.** Where the requirements of Section 2109.5.3.1 are not met, foundation walls shall be designed in accordance with Section 1805.5.~~

~~**2109.5.4 Parapet walls.**~~

~~**2109.5.4.1 Minimum thickness.** The minimum thickness of unreinforced masonry parapets shall meet Section 2109.5.2.6 and their height shall not exceed three times their thickness.~~

~~**2109.5.4.2 Additional provisions.** Additional provisions for parapet walls are contained in Sections 1503.2 and 1503.3.~~

~~**2109.6 Bond.**~~

~~**2109.6.1 General.** The facing and backing of multiwythe masonry walls shall be bonded in accordance with Section 2109.6.2, 2109.6.3 or 2109.6.4.~~

~~**2109.6.2 Bonding with masonry headers.**~~

~~**2109.6.2.1 Solid units.** Where the facing and backing (adjacent wythes) of solid masonry construction are bonded by means of masonry headers, no less than 4 percent of the wall surface of each face shall be composed of headers extending not less than 3 inches (76 mm) into the backing. The distance between adjacent full-length headers shall not exceed 24 inches (610 mm) either vertically or horizontally. In walls in which a single header does not extend through the wall, headers from the opposite sides shall overlap at least 3 inches (76 mm), or headers from opposite sides shall be covered with another header course overlapping the header below at least 3 inches (76 mm).~~

~~**2109.6.2.2 Hollow units.** Where two or more hollow units are used to make up the thickness of a wall, the stretcher courses shall be bonded at vertical intervals not exceeding 34 inches (864 mm) by lapping at least 3 inches (76 mm) over the unit below, or by lapping at vertical intervals not exceeding 17 inches (432 mm) with units that are at least 50 percent greater in thickness than the units below.~~

~~**2109.6.2.3 Masonry bonded hollow walls.** In masonry bonded hollow walls, the facing and backing shall be bonded so that not less than 4 percent of the wall surface of each face is composed of masonry bonded units extending not less than 3 inches (76 mm) into the backing. The distance between adjacent bonders shall not exceed 24 inches (610 mm) either vertically or horizontally.~~

~~**2109.6.3 Bonding with wall ties or joint reinforcement.**~~

~~**2109.6.3.1 Bonding with wall ties.** Except as required by Section 2109.6.3.1.1, where the facing and backing adjacent wythes) of masonry walls are bonded with wire size W2.8 (MW18) wall ties or metal wire of equivalent stiffness embedded in the horizontal mortar joints, there shall be at least one metal tie for each 41/2 square feet (0.42 m^2) of wall area. The maximum vertical distance between ties shall not exceed 24 inches (610 mm), and the maximum horizontal distance shall not exceed 36 inches (914 mm). Rods or ties bent to rectangular shape shall be used with hollow masonry units laid with the cells vertical. In other walls, the ends of ties shall be bent to 90-degree (1.57 rad) angles to provide hooks no less than 2 inches (51 mm) long. Wall ties shall be without drips. Additional bonding ties shall be provided at all openings, spaced not more than 36 inches (914 mm) apart around the perimeter and within 12 inches (305 mm) of the opening.~~

~~**2109.6.3.1.1 Bonding with adjustable wall ties.** Where the facing and backing (adjacent wythes) of masonry are bonded with adjustable wall ties, there shall be at least one tie for each 1.77 square feet (0.164 m^2) of wall area. Neither the vertical nor horizontal spacing of the adjustable wall ties shall exceed 16 inches (406 mm). The maximum vertical offset of bed joints from one wythe to the other shall be 11/4 inches (32 mm). The maximum clearance between connecting parts of the ties shall be 1/16 inch (1.6 mm). When pintle legs are used, ties shall have at least two wire size W2.8 (MW18) legs.~~

~~**2109.6.3.2 Bonding with prefabricated joint reinforcement.** Where the facing and backing (adjacent wythes) of masonry are bonded with prefabricated joint reinforcement, there shall be at least one cross wire serving as a tie for each 22/3 square feet (0.25m^2) of wall area. The vertical spacing of the joint reinforcing shall not exceed 24 inches (610 mm). Cross wires on prefabricated joint reinforcement shall not be less than W1.7 (MW11) and shall be without drips. The longitudinal wires shall be embedded in the mortar.~~

~~**2109.6.4 Bonding with natural or cast stone.**~~

~~**2109.6.4.1 Ashlar masonry.** In ashlar masonry, bonder units, uniformly distributed, shall be provided to the extent of not less than 10 percent of the wall area. Such bonder units shall extend not less than 4 inches (102 mm) into the backing wall.~~

~~**2109.6.4.2 Rubble stone masonry.** Rubble stone masonry 24 inches (610 mm) or less in thickness shall have bonder units with a maximum spacing of 36 inches (914 mm) vertically and 36 inches (914 mm) horizontally, and if the masonry is of greater thickness than 24 inches (610 mm), shall have one bonder unit for each 6 square feet (0.56 m^2) of wall surface on both sides.~~

~~**2109.6.5 Masonry bonding pattern.**~~

~~**2109.6.5.1 Masonry laid in running bond.** Each wythe of masonry shall be laid in running bond, head joints in successive courses shall be offset by not less than one-fourth the unit length or the masonry walls shall be reinforced longitudinally as required in Section 2109.6.5.2.~~

~~**2109.6.5.2 Masonry laid in stack bond.** Where unit masonry is laid with less head joint offset than in Section 2109.6.5.1, the minimum area of horizontal reinforcement placed in mortar bed joints or in bond beams spaced not more than 48 inches (1219 mm) apart, shall be 0.0003 times the vertical cross-sectional area of the wall.~~

~~**2109.7 Anchorage.**~~

~~**2109.7.1 General.** Masonry elements shall be anchored in accordance with Sections 2109.7.2 through 2109.7.4.~~

~~**2109.7.2 Intersecting walls.** Masonry walls depending upon one another for lateral support shall be anchored or bonded at locations where they meet or intersect by one of the methods indicated in Sections 2109.7.2.1 through 2109.7.2.5.~~

~~**2109.7.2.1 Bonding pattern.** Fifty percent of the units at the intersection shall be laid in an overlapping masonry bonding pattern, with alternate units having a bearing of not less than 3 inches (76 mm) on the unit below.~~

~~**2109.7.2.2 Steel connectors.** Walls shall be anchored by steel connectors having a minimum section of 1/4 inch (6.4 mm) by 11/2 inches (38 mm), with ends bent up at least 2 inches (51 mm) or with cross pins to form anchorage. Such anchors shall be at least 24 inches (610 mm) long and the maximum spacing shall be 48 inches (1219 mm).~~

~~**2109.7.2.3 Joint reinforcement.** Walls shall be anchored by joint reinforcement spaced at a maximum distance of 8 inches (203 mm). Longitudinal wires of such reinforcement shall be at least wire size W1.7 (MW 11) and shall extend at least 30 inches (762 mm) in each direction at the intersection.~~

~~**2109.7.2.4 Interior nonload-bearing walls.** Interior nonload-bearing walls shall be anchored at their intersection, at vertical intervals of not more than 16 inches (406 mm) with joint reinforcement or 1/4-inch (6.4 mm) mesh galvanized hardware cloth.~~

~~**2109.7.2.5 Ties, joint reinforcement or anchors.** Other metal ties, joint reinforcement or anchors, if used, shall be spaced to provide equivalent area of anchorage to that required by this section.~~

~~**2109.7.3 Floor and roof anchorage.** Floor and roof diaphragms providing lateral support to masonry shall comply with the live loads in Section 1607.3 and shall be connected to the masonry in accordance with Sections 2109.7.3.1 through 2109.7.3.3. Roof loading shall be determined in accordance with Chapter 16 and, when net uplift occurs, uplift shall be resisted entirely by an anchorage system designed in accordance with the provisions of Sections 2.1 and 2.3, Sections 3.1 and 3.3 or Chapter 4 of ACI 530/ASCE 5/TMS 402.~~

~~**2109.7.3.1Wood floor joists.** Wood floor joists bearing on masonry walls shall be anchored to the wall at intervals not to exceed 72 inches (1829 mm) by metal strap anchors. Joists parallel to the wall shall be anchored with metal straps spaced not more than 72 inches (1829 mm) o.c. extending over or under and secured to at least three joists. Blocking shall be provided between joists at each strap anchor.~~

~~**2109.7.3.2 Steel floor joists.** Steel floor joists bearing on masonry walls shall be anchored to the wall with 3/8-inch (9.5 mm) round bars, or their equivalent, spaced not more than 72 inches (1829 mm) o.c. Where joists are parallel to the wall, anchors shall be located at joist bridging.~~

~~**2109.7.3.3 Roof diaphragms.** Roof diaphragms shall be anchored to masonry walls with 1/2-inch-diameter (12.7 mm) bolts, 72 inches (1829 mm) o.c. or their equivalent. Bolts shall extend and be embedded at least 15 inches (381 mm) into the masonry, or be hooked or welded to not less than 0.20 square inch (129 mm2) of bond beam reinforcement placed not less than 6 inches (152 mm) from the top of the wall.~~

~~**2109.7.4 Walls adjoining structural framing.** Where walls are dependent upon the structural frame for lateral support, they shall be anchored to the structural members with metal anchors or otherwise keyed to the structural members. Metal anchors shall consist of 1/2-inch (12.7 mm) bolts spaced at 48 inches (1219 mm) o.c. embedded 4 inches (102 mm) into the masonry, or their equivalent area.~~

7. Revise as follows:

SECTION 2110
ADOBE MASONRY

~~2109.8~~ 2110.1 Adobe construction. Adobe construction shall comply with this section and shall be subject to the requirements of this code for Type V construction and Chapter 5 of ACI 530/ASCE 5/TMS 402.

2110.1.1 Limitations. The use of adobe masonry shall be limited as noted in Section 5.1.2 of ACI 530/ASCE 5/TMS 402. The use of adobe masonry shall be prohibited in structures assigned to Occupancy Category IV. In buildings that exceed one or more of the limitations in Section 5.1.2 of ACI 530/ASCE 5/TMS 402, masonry shall be designed in accordance with the engineered design provisions of Section 2101.2.1, 2101.2.2, 2101.2.3 or the foundation wall provisions of Section 1805.5.

~~2109.8.1~~ 2110.2 Unstablized adobe.

(Renumber Sections 2109.8.1.1 through 2109.8.4.7)

1704.5 Masonry construction. Masonry construction shall be inspected and evaluated in accordance with the requirements of Sections 1704.5.1 through 1704.5.3, depending on the classification of the building or structure or nature of the occupancy, as defined by this code.

Exception: Special inspections shall not be required for:

1. Empirically designed masonry, glass unit masonry, ~~or~~ masonry veneer, surface-bonded masonry or adobe masonry designed by Section ~~2109, 2110, or Chapter 14, respectively, or by Chapter 5, 7 or 6 of the ACI 530/ASCE 5/TMS 402,~~ 2101.2.4, 2101.2.5, 2101.2.6, 2101.2.7, or 2101.2.8 respectively, when they are part of structures classified as Occupancy Category I, II or III in accordance with Section 1604.5.
2. Masonry foundation walls constructed in accordance with Table 1805.5(1), 1805.5(2), 1805.5(3) or 1805.5(4).
3. Masonry fireplaces, masonry heaters or masonry chimneys installed or constructed in accordance with Section 2111, 2112 or 2113, respectively.

1704.5.1 Empirically designed masonry, glass unit masonry and masonry veneer, in Occupancy Category IV. The minimum special inspection program for empirically designed masonry, glass unit masonry or masonry veneer designed by Section 2101.2.4, 2101.2.5, or 2101.2.6~~2109, 2110 or Chapter 14, respectively, or by Chapters 5, 7 or 6 of ACI 530/ASCE 5/TMS 402~~, respectively in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with Table 1704.5.1.

1704.5.2 Engineered masonry in Occupancy Category I, II or III. The minimum special inspection program for masonry designed by Section 2101.2.1, 2101.2.2 or 2101.2.3~~2107 or 2108 or by chapters other than Chapters 5, 6 or 7 of ACI 530/ASCE 5/TMS 402~~ in structures classified as Occupancy Category I, II or III, in accordance with Section 1604.5, shall comply with Table 1704.5.1.

1704.5.3 Engineered masonry in Occupancy Category IV. The minimum special inspection program for masonry designed by Section 2101.2.1, 2101.2.2 or 2101.2.3~~2107 or 2108 or by chapters other than Chapters 5, 6 or 7 of ACI 530/ASCE 5/TMS 402~~ in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with Table 1704.5.3.

1708.1.1 Empirically designed masonry, ~~and~~ glass unit masonry, surface-bonded masonry and adobe masonry in Occupancy Category I, II, or III. For masonry designed by Section ~~2109 or 2110 or by Chapters 5 or 7 ACI 530/ASCE 5/TMS 402~~2101.2.4, 2101.2.5, 2101.2.7, or 2101.2.8 in structures classified as Occupancy Category I, II or III, in accordance with Section 1604.5, certificates of compliance used in masonry construction shall be verified prior to construction.

1708.1.2 Empirically designed masonry and glass unit masonry in Occupancy Category IV. The minimum testing and verification prior to construction for masonry special inspection program for masonry designed by Section ~~2109 or 2110 or by Chapters 5 or 7 ACI 530/ASCE 5/TMS 402~~2101.2.4 or 2101.2.5 in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with Table 1708.1.2

1708.1.3 Engineered masonry in Occupancy Category I, II or III. The minimum testing and verification prior to construction for masonry designed by Section 2101.2.1, 2101.2.2 or 2101.2.3~~2107 or 2108 or by chapters other than Chapters 5, 6 or 7 of ACI 530/ASCE 5/TMS 402~~ in structures classified as Occupancy Category I, II or III, in accordance with Section 1604.5, shall comply with Table 1708.1.2

1708.1.4 Engineered masonry in Occupancy Category IV. The minimum testing and verification prior to construction for masonry designed by Section 2101.2.1, 2101.2.2 or 2101.2.3~~2107 or 2108 or by chapters other than Chapters 5, 6 or 7 of ACI 530/ASCE 5/TMS 402~~ in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with Table 1708.1.4

404.5 Enclosure of atriums. Atrium spaces shall be separated from adjacent spaces by a 1-hour fire barrier constructed in accordance with Section 706 or a horizontal assembly constructed in accordance with Section 711, or both.

Exceptions:

1. A glass wall forming a smoke partition where automatic sprinklers are spaced 6 feet (1829 mm) or less along both sides of the separation wall, or on the room side only if there is not a walkway on the atrium side, and between 4 inches and 12 inches (102mm and 305 mm) away from the glass and designed so that the entire surface of the glass is wet upon activation of the sprinkler system without obstruction. The glass shall be installed in a gasketed frame so that the framing system deflects without breaking (loading) the glass before the sprinkler system operates.

2. A glass-block wall assembly in accordance with Section ~~2110~~ 2101.2.5 and having a 3/4-hour fire protection rating.
3. The adjacent spaces of any three floors of the atrium shall not be required to be separated from the atrium where such spaces are included in the design of the smoke control system.

Reason: This is a comprehensive change to remove transcribed provisions for empirically designed masonry and glass unit masonry that were taken from the referenced standard ACI 530/ASCE 5/TMS 402. By removing these duplicate provisions, the IBC is simplified and the chance that the provisions in the IBC and the referenced standard vary unnecessarily is avoided. In addition, for clarity, the provisions for Surface-bonded masonry and Adobe Masonry are proposed to be moved to separate chapters.

Since the development of the IBC, the sponsoring organizations of the ACI 530/ASCE 5/TMS 402 have been concerned about portions of that standard that were transcribed into the IBC directly because the provisions in the IBC could begin to digress from those in the referenced standard. If this happened, two differing sets of requirements would exist, creating unnecessary confusion in the design, construction and compliance communities. However, at the request of the masonry industry, these provisions were maintained in the IBC.

During the development of the 2006 IBC however, concerns about maintaining accurate transcription were again raised, and considerable confusion was caused in trying to make sure the IBC matched the referenced standard. Because of this, and because designers, builders, and inspectors must have the referenced standard for other portions of masonry construction, the transcribed provisions are proposed for removal from the IBC.

This comprehensive change would remove redundant provisions for empirically designed masonry and glass unit masonry, and move extra IBC limitations on glass unit masonry to Section 2101.2.5 so that the major Sections on Empirical Design (Section 2109) and Glass Unit Masonry (Section 2110) can be used instead for Surface-bonded Masonry and Adobe masonry, which are not addressed in ACI 530/ASCE 5/TMS 402, and are not truly addressed in the Empirical Design provisions of that standard. This makes use of these provisions much more understandable, and makes the IBC easier to follow.

To maintain a clear and logical design and inspection path, section references are updated in Chapter 17 to clearly identify the design methods as described in Section 2101 of the IBC. With the exceptions noted below, this does not change the application of the inspection provisions, but more clearly and easily identifies which requirements apply to which design method.

To the best of knowledge of the proponents, this change has not technical impact with one exception. Currently, it could be interpreted that the IBC permits the use of surface bonded masonry and adobe masonry in essential facilities classified as Occupancy Category IV. The proponents are not sure this was truly the intent of the IBC and thus have proposed prohibiting the use of these methods from Occupancy Category IV.

The proponents have submitted two other code change proposals that address the removal of transcribed empirical design and glass unit masonry separately, and in a less comprehensive manner that would necessitate a few less broad changes than this proposal (in essence they keep empirical design, surface bonded masonry and adobe masonry in an abbreviated form in Section 2109 while maintaining a skeletal Section 2110 for glass unit masonry). The proponents strongly favor this proposal because of the resulting clarity it provides. However, if it is felt that IBC Section 2109 and 2110 should be maintained for Empirical Design and Glass Unit Masonry, another third alternative could be to add new Sections at the end of Chapter 21 specifically for surface-bonded masonry and adobe masonry.

If this change is accepted the other two changes to remove transcribed empirical design and adobe masonry provisions would be withdrawn by the proponents.

In the future, updates to empirical requirements and glass unit masonry provisions will be appropriately made and balloted in the proper consensus forum – that being the Masonry Standards Joint Committee, which is charged with updating the ACI 530/ASCE 5/TMS 402.

As noted, most of the empirical design and glass unit masonry in IBC Section 2109.1 through 2109.7 and in IBC 2110.2 through 2110.7 are taken directly from Chapters 5 and 7, respectively, of the ACI 530/ASCE 5/TMS 402. The Commentary to ACI 530/ASCE 5/TMS 402 provides substantiation to the provisions that are in these chapters, and which currently exist in IBC Section 2109.

Cost Impact: The code change proposal will not increase the cost of construction because no design, construction, or compliance requirements have changed.

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

S61–06/07

2110, 2110.2 through 2110.7

Proponents: Phillip Samblanet, The Masonry Society/Jason J. Thompson, Masonry Alliance for Codes and Standards

1. Revise as follows:

2110.1 ~~Scope~~ General. ~~This section covers the empirical requirements for non-load-bearing glass unit masonry elements in exterior or interior walls.~~ Glass unit masonry shall comply with Chapter 7 of ACI 530/ASCE 5/TMS 402 and this section.

2. Delete without substitution:

~~**2110.2 Units.** Hollow or solid glass-block units shall be standard or thin units.~~

~~**2110.2.1 Standard units.** The specified thickness of standard units shall be at least 3 7/8 inches (98 mm).~~

~~**2110.2.2 Thin units.** The specified thickness of thin units shall be 3 1/8 inches (79 mm) for hollowunits or 3 inches (76 mm) for solid units.~~

~~**2110.3 Panel size.**~~

~~**2110.3.1 Exterior standard-unit panels.** The maximum area of each individual exterior standard-unit panel shall be~~

~~144 square feet (13.4 m_2) when the design wind pressure is 20 psf (958 N/m_2). The maximum panel dimension between structural supports shall be 25 feet (7620 mm) in width or 20 feet (6096 mm) in height. The panel areas are permitted to be adjusted in accordance with Figure 2110.3.1 for other wind pressures.~~

~~**2110.3.2 Exterior thin-unit panels.** The maximum area of each individual exterior thin-unit panel shall be 85 square feet (7.9 m^2). The maximum dimension between structural supports shall be 15 feet (4572 mm) in width or 10 feet (3048 mm) in height. Thin units shall not be used in applications where the design wind pressure exceeds 20 psf (958 N/m^2).~~

~~**2110.3.3 Interior panels.** The maximum area of each individual standard-unit panel shall be 250 square feet (23.2 m^2). The maximum area of each thin-unit panel shall be 150 square feet (13.9 m^2). The maximum dimension between structural supports shall be 25 feet (7620 mm) in width or 20 feet (6096 mm) in height.~~

~~**2110.3.4 Solid units.** The maximum area of solid glass-block wall panels in both exterior and interior walls shall not be more than 100 square feet (9.3 m^2).~~

~~**2110.3.5 Curved panels.** The width of curved panels shall conform to the requirements of Sections 2110.3.1, 2110.3.2 and 2110.3.3, except additional structural supports shall be provided at locations where a curved section joins a straight section, and at inflection points in multicurved walls.~~

~~**2110.4 Support.**~~

~~**2110.4.1 General requirements.** Glass unit masonry panels shall be isolated so that in-plane loads are not imparted to the panel.~~

~~**2110.4.2 Vertical.** Maximum total deflection of structural members supporting glass unitmasonry shall not exceed $l/600$.~~

~~**2110.4.2.1 Support on wood construction.** Glass unit masonry having an installed weight of 40 psf (195 kg/m^2) or less and a maximum height of 12 feet (3658 mm) shall be permitted to be supported on wood construction.~~

~~**2110.4.2.2 Expansion joint.** A vertical expansion joint in glass unit masonry shall be provided to isolate the glass unit masonry supported by wood construction from that supported by other types of construction.~~

~~**2110.4.3 Lateral.** Glass unit masonry panels more than one unit wide or one unit high shall be laterally supported along their tops and sides. Lateral support shall be provided by panel anchors along the top and sides spaced not more than 16 inches (406 mm) o.c. or by channel-type restraints. Glass unit masonry panels shall be recessed at least 1 inch (25 mm) within channels and chases. Channel-type restraints shall be oversized to accommodate expansion material in the opening and packing and sealant between the framing restraints and the glass unit masonry perimeter units. Lateral supports for glass unit masonry panels shall be designed to resist applied loads, or a minimum of 200 pounds per lineal feet (plf) (2919 N/m) of panel, whichever is greater.~~

~~**Exceptions:**~~

1. ~~Lateral support at the top of glass unit masonry panels that are no more than one unit wide shall not be required.~~
2. ~~Lateral support at the sides of glass unit masonry panels that are no more than one unit high shall not be required.~~

~~**2110.4.3.1 Single unit panels.** Single unit glass unit masonry panels shall conform to the requirements of Section 2110.4.3, except lateral support shall not be provided by panel anchors.~~

~~**2110.5 Expansion joints.** Glass unit masonry panels shall be provided with expansion joints along the top and sides at structural supports. Expansion joints shall have sufficient thickness to accommodate displacements of the supporting structure, but shall not be less than 3/8 inch (9.5 mm) in thickness. Expansion joints shall be entirely free of mortar or other debris and shall be filled with resilient material. The sills of glass-block panels shall be coated with approved water-based asphaltic emulsion, or other elastic waterproofing material, prior to laying the first mortar course.~~

~~**2110.6 Mortar.** Mortar for glass unit masonry shall comply with Section 2103.8.~~

~~**2110.7 Reinforcement.** Glass unit masonry panels shall have horizontal joint reinforcement spaced not more than 16 inches (406 mm)on center, located in the mortar bed joint, and extending the entire length of the panel but not across expansion joints. Longitudinal wires shall be lapped a minimum of 6 inches (152 mm) at splices. Joint reinforcement shall be placed in the bed joint immediately below and above openings in the panel. The reinforcement shall have not less than two parallel longitudinal wires of size W1.7 (MW11), and have welded cross wires of size W1.7 (MW11).~~

Reason: This change removes transcribed provisions from the referenced standard ACI 530/ASCE 5/TMS 402, thus simplifying the IBC and avoiding the chance that the provisions in the IBC and the referenced standard vary unnecessarily.

Since the development of the IBC, the sponsoring organizations of the ACI 530/ASCE 5/TMS 402 have been concerned about portions of that standard that were transcribed into the IBC directly because the provisions in the IBC could begin to digress from those in the referenced standard. If this happened, two differing sets of requirements would exist, creating unnecessary confusion in the design, construction and compliance communities. However, at the request of the masonry industry, these provisions were maintained in the IBC.

During the development of the 2006 IBC however, concerns about maintaining accurate transcription were again raised, and considerable confusion was caused in trying to make sure the IBC matched the referenced standard. Because of this, and because designers, builders, and inspectors must have the referenced standard for other portions of masonry construction, the transcribed provisions are proposed for removal from the IBC.

The sections proposed for deletion are included directly in Chapter 7 of ACI 530/ASCE 5/TMS 402. No technical change is being proposed by this Code Change Proposal.

In the future, updates to glass unit masonry provisions will be appropriately made and balloted in the proper consensus forum – that being the Masonry Standards Joint Committee, which is charged with updating the ACI 530/ASCE 5/TMS 402.

A final note for the editorial layout of the IBC. If this change is approved, this will leave only one subsection in Section 2110. If this is not permitted, an alternate method of addressing this issue would be to delete Section 2110 in its entirety, and to add the text in the limitation to Section 2101.2.5. The following describes how this could be done, and the proponents would accept whichever format is more appropriate to the IBC. However, if Section 2110 is deleted, all sections after that would require renumbering, and because it was thought that this would cause more work for staff, the change was drafted as proposed above.

Alternative revision: Revise Section 2101.2.5 as follows:

2101.2.5 Glass masonry. Glass masonry shall comply with the ~~provisions of Section 2110 or with the~~ requirements of Chapter 7 of ACI 530/ASCE 5/TMS 402 <u>and this section</u>.

<u>2101.2.5.1 Limitations. Solid or hollow approved glass block shall ….</u>

<u>Exceptions:</u>

<u>1. Glass-block assemblies having ...</u>
<u>2. Glass-block assemblies as permitted ...</u>

Delete without substitution:
Section 2110, and renumber remaining sections of Chapter 21.

Because of these revisions, delete "~~2110~~" and replace with "<u>2101.2.5</u>" in Sections 404.5, Item 2, 1704.5, Item 1, 1704.5.1, 1708.1.1, and 1708.1.2. Also delete unneeded references to ACI 530/ASCE 5/TMS 402 in Chapter 35 for consistency.

As noted, the IBC provisions in Section 2110 are taken directly from Chapter 7 of the ACI 530/ASCE 5/TMS 402. The Commentary to ACI 530/ASCE 5/TMS 402 provides substantiation to the provisions that are in Chapter 7 of that standard, and which currently exist in IBC Section 2110.

Cost Impact: The code change proposal will not increase the cost of construction because no design, construction, or compliance requirements have changed.

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

S62–06/07

2203.1; IRC R301.2.1.1.1 (New)

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: Grady Halamicek, City of Longmont, CO

PART I – IBC

Revise as follows:

SECTION 2203
IDENTIFICATION AND PROTECTION OF STEEL FOR STRUCTURAL PURPOSES

2203.1 Identification. Steel furnished for structural load-carrying purposes shall be ~~properly identified for conformity to the ordered grade in accordance with the specified ASTM standard or other specification and the provisions of this chapter. Steel that is not readily identifiable as to grade from marking and test records shall be tested to determine conformity to such standards.~~ <u>permanently identified as to its size, weight and grade by stenciling, stamping or other approved methods at intervals not to exceed 6 feet (1829 mm)</u>

PART II – IRC

Add new text as follows:

<u>**R301.2.1.1.1 Identification.** Steel furnished for structural load carrying purposes shall be permanently identified as to its size, weight and grade by stenciling, stamping or other approved methods at intervals not to exceed 6 feet (1829 mm).</u>

Reason: To quickly and accurately determine size and weight of steel. The current code does not go far enough in the requirement for identifying steel.

Cost Impact: Cost impact is negligible.

PART I – IBC

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

PART II – IRC

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

S63–06/07
2206.4

Proponent: George Thomas, P.E., C.B.O., Pleasanton, CA, representing Tri-Chapter Code Committee

Revise as follows:

2206.4 Steel joist drawings. Steel joist placement plans shall be provided to show the steel joist products as specified on the construction documents and are to be utilized for field installation in accordance with specific project requirements as stated in Section 2206.2. Steel placement plans shall include, at a minimum, the following:

1. Listing of all applicable loads as stated in Section 2206.2 and used in the design of the steel joists and joist girders as specified in the construction documents.
2. Profiles for nonstandard joist and joist girder configurations (standard joist and joist girder configurations are as indicated in the SJI catalog).
3. Connection requirements for:
   3.1. Joist supports;
   3.2. Joist girder supports;
   3.3. Field splices; and
   3.4. Bridging attachments.
4. Deflection criteria for live and total loads for non-SJI standard joists.
5. Size, location and connections for all bridging.
6. Joist headers.

~~Steel joist placement plans do not require the seal and signature of the joist manufacturer's registered design professional.~~

Reason: To delete a current provision that is inconsistent with other code provisions.

The last sentence of Section 2206.4 specifically exempts the steel joist placement plan from requiring the seal and signature of the responsible joist manufacturer's registered design professional. We believe that if the joist manufacturer is required by the code to provide (e.g., prepare) steel joist placement plans containing all of the specific information listed in Section 2206.4, then it is reasonable to allow the Building Official to request the preparer of those plans to take formal responsibility for this information by sealing and signing the layout plan, if requested. The proposed deletion of the sentence does not require sealed and signed plans, it simply reinstates the legitimate authority of the Building Official to request sealed and signed steel joist placement plans if desired.

In the entirety of the IBC there are currently no other examples where a specific exemption is granted from providing a seal and signature by a design professional on a specific type of drawing. In fact, in IBC Section 2303.4.1.3, there is a specific requirement for seal and signature on wood truss placement diagrams, when prepared under the direct supervision of a registered design professional. Given the content and nature of the six items that Section 2206.4 requires to be included on the steel joist placement plans, it is very unlikely that the placement plan would be prepared outside of the direct supervision of the steel joist design professional. Section 2206.4 currently results in unequal requirements for steel joists submittal documents compared to wood trusses submittals, and that inequity does not appear to be justified.

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

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

S64–06/07
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:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S65–06/07
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:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S66–06/07
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**~~ **2303.4.1.1 Truss 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:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S67–06/07
2303.4.1.5, Chapter 35

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

1. Revise as follows:

2303.4.1.5 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 resisted internally by the structure (e.g. buckling member T-bracing, L-bracing, etc.) of the individual truss. The truss individual member buckling reinforcement shall be installed as shown on the truss design drawing or on supplemental truss member buckling reinforcement diagrams provided by the truss designer.
2. Permanent bracing shall be installed using standard industry bracing details that conform with generally accepted engineering practice. Individual truss member continuous lateral bracing location(s) shall be shown on the truss design drawing.

In the absence of specific bracing requirements, trusses shall be braced in accordance with the Building Component Safety Information (BCSI 1) Guide to Good Practice for Handling, Installing & Bracing of Metal Plate Connected Wood Trusses.

2. Add standard to Chapter 35 as follows:

BCSI 1 Guide to Good Practice for Handling, Installing & Bracing of Metal Plate Connected Wood Trusses

Reason: The purpose of the proposed revision is to include reference to the Truss Industry's standard bracing details as provided in the Building Component Safety Information (BCSI 1) Guide to Good Practice for the Handling, Installing & Bracing of Metal Plate Connected Wood Trusses.

Item 2 of this section makes reference to using "standard industry bracing details". The proposed addition to this section references a truss industry document that is most often referred to for standard industry bracing guidance. This will also harmonize this code requirement with the existing language in the IRC.

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

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

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

S68–06/07
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:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S69–06/07
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:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S70–06/07
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)		
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
		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.

PART I – IBC

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

PART II – IRC

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

S71–06/07
Table 2304.7(3)

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

Revise as follows:

TABLE 2304.7(3)
ALLOWABLE SPANS AND LOADS FOR WOOD STRUCTURAL PANEL SHEATHING 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)	Allowable Live Load (psf) [h]		Maximum span (inches)		Load [e] (psf)		Maximum span (inches)
		Span=16 inches	Span=24 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	30	---	16	16	40	30	0
20/0	5/16, 3/8	50	---	20	20	40	30	0
24/0	3/8, 7/16, 1/2	100	30	24	20[g]	40	30	0
24/16	7/16, 1/2	100	40	24	24	50	40	16
32/16	15/32, 1/2, 5/8	180	70	32	28	40	30	16 [h]
40/20	19/32, 5/8, 3/4, 7/8	305	130	40	32	40	30	20 [h, i]
48/24	23/32, 3/4, 7/8	---	175	48	36	45	35	24
~~54/32~~	~~7/8, 1~~			~~54~~	~~40~~	~~45~~	~~35~~	~~32~~
60/32	7/8	---	305	60	48	45	35	32
SINGLE FLOOR GRADES		**ROOF [c]**						**FLOOR [d]**
Panel span rating	**Panel thickness (inches)**	**Span=16 inches**	**Span=24 inches**	**With edge support [f]**	**Without edge support**	**Total load**	**Live load**	**Maximum span (inches)**
16 o.c.	1/2, 19/32, 5/8	100	40	24	24	50	40	16 [h]
20 o.c.	19/32, 5/8, 3/4	150	60	32	32	40	30	20 [h, j]
24 o.c.	23/32, 3/4	240	100	48	36	35	25	24
32 o.c.	7/8, 1	---	185	48	40	50	40	32
48 o.c.	1-3/32, 1-1/8	---	290	60	48	50	40	48

For SI: 1 inch = 25.4 mm, 1 pound per square foot = 0.0479 kN/m^2.

a. Applies to panels 24 inches and wider.
b. Floor and roof sheathing conforming with this table shall be deemed to meet the design criteria of Section 2304.7.
c. Uniform load deflection limitations 1/180 of span under live load plus dead load, 1/240 of span under dead load only.
d. Panel edges shall have approved tongue-and-groove joints or shall be supported by blocking unless 1/4-inch minimum thickness underlayment or 1-1/2 inches of approved cellular or lightweight concrete is placed over the subfloor, or finish flooring is 3/4-inch wood strip. Allowable uniform load based on deflection of 1/360 of span is 100 pounds per square foot, except the span rating of 48 inches on center is based on a total load of 65 pounds per square foot.
e. Allowable load at maximum span.
f. Tongue-and-groove edges, panel edge clips (one midway between each support, except two equally spaced between supports 48 inches on center), lumber blocking, or other. Only lumber blocking shall satisfy blocked diaphragm requirements
g. For 15/32- and ½-inch panel, maximum span shall be 24 inches.
h. Span is permitted to be 24 inches on center where 3/4-inch wood strip flooring is installed at right angles to joists.
i. Span is permitted to be 24 inches on center where 1-1/2 inches of cellular or lightweight concrete is applied over the panels.

Reason: To make the IBC more useable to the public by providing user with additional information on the application of roof sheathing over less-than–maximum spans. Note that the new information added is the same that was added to the corresponding table in the IRC (Table R503.2.1.1(1)) during the last code change cycle.

Two columns of information were added to the existing table. These columns give allowable roof live loads for wood structural panels when placed at less than the maximum span. For example, to accommodate a heavy snow load, a designer may want to use a 5/8-inch sheathing panel over roof supports at 16" on center. The addition to the table would give him the necessary information (305 psf live load) without having to go to a referenced standard to calculate the capacity. As stated above, this information was added to the corresponding IRC table this last cycle. This IRC change was proposed by a building official.

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

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

S72–06/07

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)				NAIL (common or galvanized box) or staple size[k]	Fastener spacing at panel edges (inches)			
				6	4	3	2[e]		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.

PART I – IBC

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

PART II – IRC

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

S73–06/07
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:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S74–06/07
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 <ins>at each joist</ins>

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:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S75–06/07
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 <ins>or 1-inch crown</ins> and ~~1 ⅛~~ <ins>1¼</ins>-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;~~ <ins>7/16" crown or 1" crown</ins> staple 16ga., ~~1 ½~~ <ins>1 ¼</ins>" long
25/32" structural cellulosic fiberboard sheathing	1 3/4" galvanized roofing nail, ~~8d common (2 ½" X 0.131") nail;~~ <ins>7/16" crown or 1" crown</ins> staple 16ga., ~~1 ¾~~ <ins>1 ½</ins>" 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.

PART I – IBC

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

PART II – IRC

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

S76–06/07
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.~~

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.

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.

> **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.

2304.9.5.2 Fastenings for wood foundations. Fastenings for wood foundations shall be as required in AF&PA Technical Report No. 7.

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.

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.

PART I – IBC

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

PART II – IRC

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

S77–06/07
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.

Proponent: John Kurtz, International Staple, Nail and Tool Association

PART I – IBC

Revise 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.

PART II – IRC

Revise 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.

Exception~~s~~:

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

Reason: The purpose of this proposal is to remove blanket inclusion of mechanically galvanized fasteners. For galvanized fasteners, the code should only include fasteners galvanized using the hot-dip galvanizing process. Fasteners galvanized by other processes should be approved based on specific evaluation.

The hot-dip galvanizing process enjoys the relative advantage of being a process very insensitive to process variables. Though mechanically galvanized fasteners can perform equally to hot-dip galvanized fasteners, the mechanical galvanizing process, if not performed properly, may produce fasteners whose corrosion resistance does not equal that of hot-dip galvanized fasteners.

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

PART I – IBC

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

PART II – IRC

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

S78–06/07
2304.9.5

Proponent: David Rochester, Plating Systems & Technologies, Inc.

Revise 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 withASTMB695, Class 55 minimum.

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

Reason: The IRC (R319.3) only excludes nails and timber rivets, at a minimum that is all that should be excluded by the IBC. Since many of the powder actuated pins being used in preservative treated lumbers are mechanically galvanized (Example: Remington), and Desa has done a significant amount of work getting approved by the ICC-ES, the restriction should be deleted. After all, one (1) ounce per square foot of zinc coating is a weighted coating and when it is applied by either the mechanical galvanizing process or the hot-dip galvanizing process, yields the same amount of zinc coating. In theory, both should provide equal amounts of corrosion protection, but in actuality, mechanical galvanizing provides significantly more corrosion protection in neutral salt spray testing. A true measure of a coating's viability should be the coating thickness followed by the corrosion protection given from such coating. Since mechanical galvanizing can equal hot-dip galvanizing in coating weight, and can exceed it in corrosion protection, the restriction on nails, timber rivets, wood screws and lag screws should be removed.

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

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

S79–06/07
2304.9.5

Proponent: Randall Shackelford, Simpson Strong-Tie Co.

Revise 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~~,~~ and 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

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

Reason: Clarify code. We proposed this code change last year, but the wording turned out to be different in the IBC and IRC. This change will make the IBC and IRC say the same thing. Mechanical galvanizing is very common for screws, and in fact is possibly the only way to deposit a thick zinc coating on them. Wood screws and lag screws are frequently installed in pre-drilled holes, so abrasion of the finish is not the same problem as for nails.

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

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

S80–06/07
2304.11.2, 2304.11.3

Proponent: Jeffrey B. Stone, Ph.D., C.B.O., American Forest & Paper Association

Revise as follows:

2304.11.2 Wood used above ground. Wood installed above ground in the locations specified in Sections 2304.11.2.1 through 2304.11.2.7, 2304.11.3 and 2304.11.5 shall be naturally durable wood or preservative-treated wood ~~using water-borne preservatives,~~ in accordance with AWPA U1(Commodity Specifications A or F) for above-ground use.

2304.11.3 Laminated timbers. The portions of glued-laminated timbers that form the structural supports of a building or other structure and are exposed to weather and not fully protected from moisture by a roof, eave or similar covering shall be pressure treated with preservative in accordance AWPA standard C28, or be manufactured from naturally durable ~~or preservative-treated~~ wood.

Reason: Part I - There is no technical reason to limit the treatments for wood used above ground to water-borne preservatives.
Part II - Clarifies the applicable AWPA standard that should be used for glued laminated timber. Many of the AWPA standards do not limit pressure preservative treatments to water-borne preservatives. The specific AWPA standard governs the type of treatment to be used.

Cost Impact: This code change proposal will not increase the cost of construction. It merely clarifies the applicable AWPA standard for glued laminated timber.

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

S81–06/07
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.

PART I – IBC

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

PART II – IRC

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

S82–06/07
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 AF&PA SDPWS and the provisions of Section 2305, 2306, and 2307. ~~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.

$$\Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\sum(\Delta_c X)}{2b}$$ **(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 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~~ 2305.3 Shear wall 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:

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

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~~ Staple 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 anels 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:	Committee:	AS	AM	D
	Assembly:	ASF	AMF	DF

S83–06/07
2305.1, 2305.1.1 through 2305.3.11

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

1. 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 AF&PA SDPWS ~~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.~~

2. Delete without substitution:

~~**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 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 seismicforces 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 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 throughout is permitted to be calculated by using the following equation. If not uniformly nailed, the constant 0.188 (For SI: 1/1627) in the third term must be modified accordingly.~~

~~**(Equation 23-1)**~~

~~**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.~~

~~**TABLE 2305.2.2(1)**~~
~~**en VALUES (inches) FOR USE IN CALCULATING DIAPHRAGM**~~
~~**DEFLECTION DUE TO FASTENER SLIP (Structural I)a,d**~~

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

~~**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 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 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:~~

~~**(Equation 23-2)**~~

~~**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, $\sum Li$, 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 a compression 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 shear wall 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 shear wall with openings uniformly fastened throughout shall be taken as the maximum individual deflection of the shear wall segments calculated in accordance with Section 2305.3.2, divided by the appropriate shear 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.~~

Reason: 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	Requirements for force transfer for shear walls with openings are covered in SDPWS 4.3.5. SDPWS includes general criteria by reference to NDS for ASD and LRFD which addresses effect of net section on design; however, a specific requirement to detail on plans the reinforcing of holes in shear panels is not included.
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. 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. 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 all anchor bolts for both wind and seismic loading. 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, 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.

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

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

S84–06/07
Table 2305.3.4

Proponent: Louis Wagner, American Fiberboard Association

Revise table as follows:

TABLE 2305.3.4
MAXIMUM SHEAR WALL DIMENSION RATIOS

TYPE	MAXIMUM HEIGHT-WIDTH RATIO
Wood structural panels or particleboard, nailed edges	For other than seismic: 3 ½ : 1 For seismic: 2:1[a]
Diagonal sheathing, single	2:1
Fiberboard structural sheathing	3 : 1 ~~1 ½ : 1~~
Gypsum board, gypsum lath, Cement plaster	1 1/2 : [b]

(No changes to footnotes)

Reason: This change revises outdated information.
A conservative interpretation of new test data justifies increasing the aspect ratio for fiberboard structural sheathing to 3:1. Fiberboard shear wall segments with aspect ratios of 1:1, 2:1, 3:1 and 4:1 were cyclically tested using ASTM E2126. Cross head displacement was controlled per the CUREE protocol. Holddowns were used at each end of the segments. The fiberboard complied with the requirement of a minimum of 5200 pounds maximum load when tested monotonically using ASTM E72. Roofing nails were spaced at 3" on panel edges and at 6" on any intermediate studs. Unit shear and deflection data is reported in Report Number EG3209-031506 which is available at www.fiberboard.org.

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

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

S85–06/07
2305.3.8 (New)

Proponent: Zeno Martin, APA-The Engineered Wood Association and Erol Karacabeyli, Forintek

Add new text as follows:

2305.3.8 Unblocked wood structural panel shear walls. Wood structural panel shear walls are permitted to be unblocked provided the allowable shear is calculated in accordance with this section. Unblocked shear walls height shall not exceed 16 feet (4880 mm). Design coefficients and factors for blocked shear walls shall be permitted to be used.

The allowable shear for unblocked wood structural panel shear walls shall be permitted to be calculated using the following formula:

$v_{ub} = v_{allowable} C_{ub}$ **(Equation 23-3)**

where:

C_{ub} = Unblocked shear wall adjustment factor from Table 2305.3.8

$v_{allowable}$ = Allowable shear from Table 2306.4.1 for wood structural panel shear wall with 24 inch (610 mm) stud spacing and nails spaced at 6 inches (152 mm)(on center around panel edges and 12 inches (305 mm) on center along intermediate framing members.

v_{ub} = Adjusted allowable shear for unblocked shear wall.

The deflection of an unblocked wood structural panel shear wall shall be permitted to be equal to the calculated deflection from Section 2305.3.2 for a blocked wood structural panel shear wall with 24 inch (610 mm) stud spacing and nails spaced at 6 inches (152 mm) on center around panel edges and 12 inches (305 mm) on center along intermediate framing members where the shear due to design loads at the top of the unblocked wall, v, used in Equation 23-2 shall be divided by C_{ub} before use in Equation 23-2.

TABLE 2305.3.8
UNBLOCKED SHEAR WALL ADJUSTMENT FACTOR, C_{ub}

NAIL SPACING (INCHES)		STUD SPACING (INCHES)			
SUPPORTED EDGES	INTERMEDIATE STUDS	12	16	20	24
6	6	1.0	0.8	0.6	0.5
6	12	0.8	0.6	0.5	0.4

(Renumber subsequent sections)

Reason: Add provisions for calculating the strength and deflection for unblocked wood structural panel shear walls.

In recent years there has been a growing interest among design professionals in having the option to specify shear walls without blocking on all edges. Unblocked shear walls have labor and material saving benefits. In addition, there are cases when having such provisions is beneficial to a designer when evaluating existing structures and for retrofit applications.

Despite the reduced strength of unblocked shear walls, there are occasions where ample strength is provided and users can realize the benefits of unblocked shear walls. This is especially true for the large areas of the country with low seismic and wind loads. Unblocked diaphragms have been recognized in US model codes for more than 35 years for the same practical reasons. In 2001, design provisions for unblocked wood shear walls were introduced into the Canadian standards (CSA, 2001).

Monotonic and cyclic testing has been completed on unblocked wood structural panel shear walls at APA and Forintek (APA, 1999; Ni and Karacabeyli, 2002; Ni et al, 2000; Mi et al, 2006). This testing has led to the proposed and confirmed that the proposed is conservative for a range of different sized walls. Strength, stiffness, ultimate displacement, and energy dissipation characteristics have been considered in the proposed unblocked shear wall adjustment factors. Closer nail spacings are not recommended to avoid stud splitting due to tension perpendicular to grain stresses. The height limit of 16-ft is proposed to be consistent with what has been tested.

References Cited:

APA, 1999. *Wood Structural Panel Shear Walls*, Research Report 154, APA – The Engineered Wood Association, Tacoma, WA.
CSA, 2001. Engineering Design in Wood, CSA 086-01, Canadian Standards Association, Toronto, Ontario.
Mi, H., Ni, C., Chui, Y.H., Karacabeyli, E., 2006. *Racking Performance of Tall Unblocked Shear Walls*. ASCE Journal of Structural Engineering. Volume 132, Number 1, 145-152.
Ni, C., Karacabeyli, E., 2002. *Effect of Blocking in Horizontally Sheathed Shear Walls*, Wood Design Focus, Forest Products Society, Volume 12, Number 2 Summer 2002, Madison, WI.
Ni, C., Karacabeyli, E., and Ceccotti, A., 2000. *Lateral Load Capacities of Horizontally Sheathed Unblocked Shearwalls*, Proceedings from International Council for Building Research Studies and Documentation, Working Commission W18 – Timber Structures, University of Karlsruhe, Germany. Paper 33-15-1.

Bibliography:

Mi, H., Ni, C., Chui, Y.H., Karacabeyli, E., 2006. *Racking Performance of Tall Unblocked Shear Walls*. ASCE Journal of Structural Engineering. Volume 132, Number 1, 145-152.
Ni, C., Karacabeyli, E., 2002. *Effect of Blocking in Horizontally Sheathed Shear Walls*, Wood Design Focus, Forest Products Society, Volume 12, Number 2 Summer 2002, Madison, WI.

Cost Impact: The code change proposal will not increase the cost of construction. The proposal will result in cost savings in buildings with ample amount of walls particularly in low wind and seismic zones.

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

S86–06/07
2305.3.1, 2305.3.8.2.1

Proponent: Randall Shackelford, Simpson Strong-Tie Co.

Revise as follows:

1. 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 therefrom, 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 of ASCE 7. Shear wall sheathing other than wood structural panels shall not be permitted in Seismic Design Category E or F(see Section 1613).

Shear walls designed in accordance with this section shall not be used to transfer wind uplift force from wall framing into wall sill plates.

2. 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).

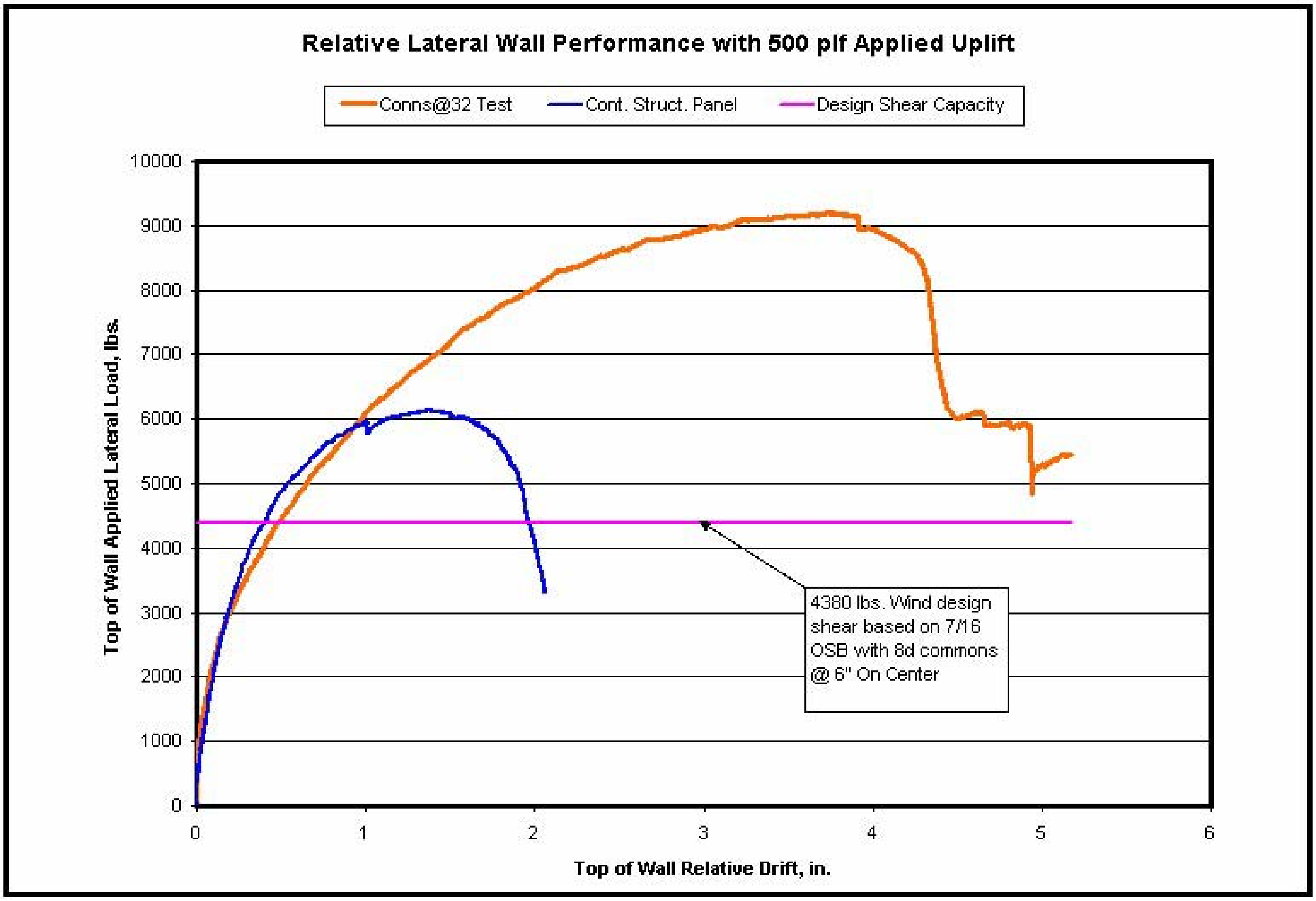
3. Where out-of-plane offsets occur, portions of the wall on each side of the offset shall be considered as separate perforated shear walls.
4. Collectors for shear transfer shall be provided through the full length of the perforated shear wall.
5. A perforated shear wall shall have uniform top of wall 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).
7. <u>Perforated shear wall sheathing shall not be designed to resist wind uplift forces except for uplift due to overturning from lateral forces.</u>

Reason: Interest has increased recently in using wood structural panels to resist wind uplift forces at the same time as they act as a shear wall to resist lateral forces. Simpson Strong-Tie has recently completed testing in our state-of-the-art full-scale test facility that conclusively shows that wood structural panel shear walls cannot be subjected to both wind uplift and shear forces and maintain the required safety factors. The failure is sudden splitting of the sill plate, even if large plate washers are installed.

Based on this testing, item 1 only intends to limit the application that results in sill plate splitting, that is transferring wind uplift to the sill plate. Conceivably, if properly detailed, wood structural panels may be able to transfer uplift forces in other locations in the wall.

The purpose of Item 2 is to add a limitation to the code. Perforated shear wall segments already transfer both shear forces and uplift forces into the sill plate along their entire length (See Section 2305.3.8.2.6), so there is not additional capacity to transfer any wind uplift force without reducing the shear capacity.

The graph below shows the reduction in shear capacity of a shear wall that is designed for and subjected to combined uplift and shear simultaneously, compared to the same wall subjected to the same loads that uses connectors to resist the uplift.



Cost Impact: The code change proposal will only increase the cost of construction if designers are currently using sheathing for combined uplift and shear resistance, which is not common.

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

S87–06/07
2305.3.8.2.1

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

Revise as follows:

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)~~ 870 plf (12690 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).

Reason: Revise current limitation. The existing limit of 490 plf is based on the fact that perforated shear wall testing had not been conducted on higher capacity walls. Recently, cyclic shear wall tests have been conducted of perforated shear walls with an 870 plf base capacity. Test results show that 870 plf based perforated shear walls performed with similar margins as lesser capacity walls and there appears to be no reason to limit perforated shear walls to a 490-plf basis. A test report is attached and is available to interested parties upon request.

Bibliography: APA test report "Perforated Shear Wall Testing" dated March 24, 2006.

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

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

S88–06/07
2305.3.8.2.2

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

Revise as follows:

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 percent 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 clear opening ~~clear~~ height. Where areas above and below an opening remain unsheathed, the height of 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 $3^1/_2$: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, ΣL_i, is greater than 2:1 but does not exceed $3^1/_2$: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.

Reason: Clarify the intent of the code. This change was proposed at the request of the ICC staff. Apparently there has been some confusion in the field as to the exact meaning as currently phrased. The change proposed seems to be easier to understand by those the ICC staff has had to deal with.

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

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

S89–06/07
2305.3.8.2.9

Proponent: Randall Shackelford, Simpson Strong-Tie Co.

Revise as follows:

2305.3.8.2.9 Deflection of shear walls with openings. The controlling deflection of a blocked shear wall with openings uniformly fastened throughout shall be ~~taken as the maximum individual deflection of the shear wall segments~~ calculated in accordance with Section 2305.3.2 with the width of the shear wall, *b*, equal to the sum of the full height shear wall segment widths that meet the aspect ratios of Section 2305.3.4, and the shear demand, *v*, equal to the adjusted shear resistance determined in accordance with Section 2305.3.8.2.2. ~~divided by the appropriate shear resistance adjustment factors of Table 2305.3.8.2.~~

Reason: Clarify the code by substituting new material. The American Forest and Paper Association has developed a methodology for determining the deflection of a perforated wood shear wall. The current language is somewhat confusing and requires more work. This code change clarifies and simplifies the process to calculate the deflection of a shear wall with openings.

Bibliography: Wood Design Focus Magazine, 2002 "Perforated Shearwall Design", written by Phil Line, P.E.

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

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

S90–06/07

2305.3.11, 2308.6, 2308.12.8, 2308.12.9; IRC 403.1.6.1, R602.11.1

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: Randall Shackelford, Simpson Strong-Tie Co.

PART I – IBC

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 Anchor 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.