S71–07/08
Table 1607.1, 1607.9.1.4, 1607.11.1, 1607.11.2.1, 1607.11.2.2

Proponent: Philip Brazil, PE, SE, Reid Middleton, Inc., representing himself

Revise as follows:

TABLE 1607.1 (Supp)
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND MINIMUM CONCENTRATED LIVE LOADS[g]

(Portions of table and footnotes not shown remain unchanged)

1607.9.1.4 Special structural elements. Live loads shall not be reduced for one-way slabs except as permitted in Section 1607.9.1.1. ~~Live loads of 100 psf (4.79 kN/m²) or less shall not be reduced for roof members except as specified in Section 1607.11.2.~~

1607.11.1 Distribution of roof loads. Where uniform roof live loads are reduced to less than 20 psf (0.96 kN/m^2) in accordance with Section 1607.11.2.1 and are ~~involved in~~ applied to the design of structural members arranged so as to create continuity, the ~~minimum applied loads~~ reduced roof live load shall be ~~the full dead loads on all spans in combination with the roof live loads on~~ applied to adjacent spans or ~~on~~ to alternate spans, whichever produces the greatest unfavorable effect. See Section 1607.11.2 for minimum roof live loads and Section 7.5 of ASCE 7 for partial snow loading.

1607.11.2.1 Flat, pitched and curved roofs. Ordinary flat, pitched and curved roofs are permitted to be designed for a reduced roof live load as specified in the following equations or other controlling combinations of loads in Section 1605, whichever produces the greater load. In structures such as greenhouses, where special scaffolding is used as a work surface for workers and materials during maintenance and repair operations, a lower roof load than specified in the following equations shall not be used unless approved by the building official. ~~Greenhouses~~ Such structures shall be designed for a minimum roof live load of 12 psf (0.58 kN/m^2).

$L_r = L_o R_1 R_2$ **(Equation 16-27)**
where: $12 \leq L_r \leq 20$
For SI: $L_r = L_o R_1 R_2$
where: $0.58 \leq L_r \leq 0.96$

L_r = Reduced live load per square foot (m^2) of horizontal projection in pounds per square foot (kN/m^2).

The reduction factors R_1 and R_2 shall be determined as follows:

$R_1 = 1$ for $A_t \leq 200$ square feet (18.58 m^2) **(Equation 16-28)**

$R_1 = 1.2 - 0.001 A_t$ for 200 square feet $< A_t <$ 600 square feet **(Equation 16-29)**
For SI: $1.2 - 0.011 A_t$ for 18.58 square meters $< A_t <$ 55.74 square meters

$R_1 = 0.6$ for $A_t \geq 600$ square feet (55.74 m^2) **(Equation 16-30)**

where:

A_t = Tributary area (span length multiplied by effective width) in square feet (m^2) supported by any structural member, and
$R_2 = 1$ for $F \leq 4$ **(Equation 16-31)**
$R_2 = 1.2 - 0.05 F$ for $4 < F < 12$ **(Equation 16-32)**
$R_2 = 0.6$ for $F \geq 12$ **(Equation 16-33)**

F = For a sloped roof, the number of inches of rise per foot (for SI: F = 0.12 x slope, with slope expressed as a percentage), or for an arch or dome, the rise-to-span ratio multiplied by 32.

1607.11.2.2 Special-purpose roofs. Roofs used for promenade purposes, roof gardens, assembly purposes or other special purposes shall be designed for a minimum live load, L_o, as ~~required~~ specified in Table 1607.1. Such roof live loads are permitted to be reduced in accordance with 1607.9.

Reason: The purpose of this proposal is to align IBC Section 1607.11 on roof live load reductions with similar provisions in Section 4.9 of ASCE 7-05 and to make related editorial revisions. Section 1607.11.1 on distribution of roof loads is revised for consistency with Footnote (h) of Table 4-1 in ASCE 7-05.

In Section 1607.11.2.1, "equation" is changed to "equations" in two places because the related provisions refer to Equations 16-27 through 16-33. A second paragraph is created to distinguish the general provisions from the specific provisions related to structures with special scaffolding. The reference to greenhouses is relocated for consistency with Section 4.9.1 of ASCE 7-05. Without this last revision, the last sentence of the section has no specific relationship to the second sentence, which is judged not to be the intent.

The notation, L_o, is added to Table 1607.1 for consistency with Table 4-1 of ASCE 7-05. Note that Sections 1607.9 and 1607.11 both reference minimum uniformly distributed live loads, L_o, in Table 1607.1, in the same manner as Sections 4.8 and 4.9, respectively, of ASCE 7-05. The notation is also added to Section 1607.11.2.2 on special purpose roofs. In the same section, "required" is changed to "specified" because tables don't require, they specify in conjunction with charging language that, in this case, is found in Section 1607.11.

In Section 1607.9.1.4, the second sentence on the limitations for the reduction of live loads is deleted for consistency with Section 4.9 of ASCE 7-05 and because it is judged to be archaic. The deletion eliminates a potential conflict with the charging language in Section 1607.9, which excludes roof live loads from the scope of Section 1607.9. The reduction of roof live loads is covered in Section 1607.11.2. Section 1607.9.1.4 prohibits reductions in roof live loads of 100 psf or less except as specified in Section 1607.11.2. It is silent, however, on reductions in roof live loads greater than 100 psf. Section 1607.11.2 is limited in scope to the roof live loads specified in Table 1607.1. All of the roof live loads listed in Table 1607.1 are 100 psf or less.

This proposal was prepared in conjunction with related proposals on reduction of floor live loads, live loads at marquees, and reduction of live loads at roofs used for assembly purposes.

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

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

S72–07/08
1607.7.1.2; IRC R312.2

Proponent: Bruce Dodge, Building Official, City of Grand Haven, MI, representing himself

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STRUCTURAL AND THE IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

1607.7.1.2 Components. Intermediate rails (all those except the handrail), balusters and panel fillers shall be designed to withstand a horizontally applied normal load of 50 pounds (0.22 kN) on an area equal to 1 square foot ($0.093m^2$), including openings and space between rails. Reactions due to this loading are not required to be superimposed with those of Section 1607.7.1 or 1607.7.1.1. <u>Where balusters or cables or individual components must comply with opening limitations in accordance with Section 1013.3, the individual components shall not allow the passage of a 4 inch (102 mm) sphere except when a force greater than 50 pounds (0.22 kN) is applied to the sphere in any direction.</u>

PART II – IRC BUILDING/ENERGY

Revise as follows:

R312.2 Guard opening limitations. Required guards on open sides of stairways, raised floor areas, balconies and porches shall have intermediate rails<u>, balusters, cables</u> or ornamental closures which do not allow passage of a sphere 4 inches (102 mm) or more in diameter <u>except when a force greater than 50 pounds (0.22 kN) is applied to the sphere in any direction.</u>

Exceptions:

1. The triangular openings formed by the riser, tread and bottom rail of a guard at the open side of a stairway are permitted to be of such a size that a sphere 6 inches (152 mm) cannot pass through.
2. Openings for required guards on the sides of stair treads shall not allow a sphere 4 3/8 inches (107 mm) to pass through

Reason: With guard rails being made of plastic or cables which can be very strong in one direction and weak in the other I have found some guard rails that can be spread with little effort allowing a four inch sphere to go through with little or no effort. Section 1607.7.1.2 only require the12 inch square horizontal test showing that the components will withstand the side pressure of 50 pounds. I spoke to ICC about this issue and found that when ESS approves a guardrail system the guardrails are tested to an ASTM Standard but it does not include a requirement to test for separation of the components.

What good is a guardrail if children can squeezes through? Therefore, I'm proposing a change to require that the balusters / components of the guardrail be test to show that it will take a minimum of 50 pound pressure to spread them apart to allow a 4 inch sphere to pass through. An inspector, manufacture, or contractor can do a test very easily by getting a 4-inch sphere and a fish scale and pulling the 4 inch sphere through the railing when an inspector thinks that it will not meet the 50 pound test.

I have inspected guardrails where a 4-inch sphere will come through the balusters with less than 10 pounds of pressure. Next time you see a plastic guardrail try putting your knee through the balusters and see how much pressure it takes. Some I have tried are very strong but others will allow it with very little pressure.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

S73–07/08
1607.7.3

Proponent: Donald R. Monahan, Walker Parking Consultants, representing Parking Consultants Council of the National Parking Association

Revise as follows:

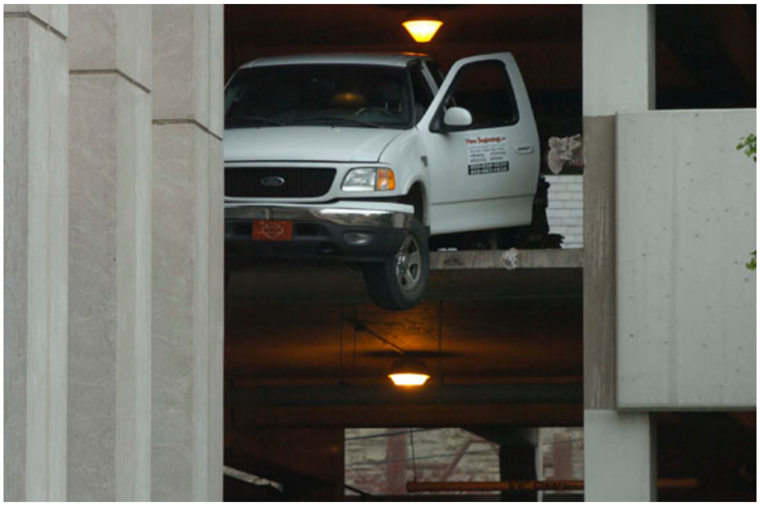
1607.7.3 Vehicle barriers. Vehicle barrier systems for passenger cars shall be designed to resist a single load of 6,000 pounds (26.70 kN) applied horizontally in any direction to the barrier system and shall have anchorage or attachment capable of transmitting this load to the structure. For design of the system, the load shall be assumed to act at a minimum height of 1 foot, 6 inches (457 mm) above the floor or ramp surface on an area not to exceed 1 square foot (305 mm^2), and is not required to be assumed to act concurrently with any handrail or guard loadings specified in the preceding paragraphs of Section 1607.7.1. <u>Connections of the barrier system to the supporting structure shall be designed to be ductile and to extend and deform to absorb impact energy prior to ultimate failure or disconnecting.</u> Garages accommodating trucks and buses shall be designed in accordance with an approved method that contains provision for traffic railings.

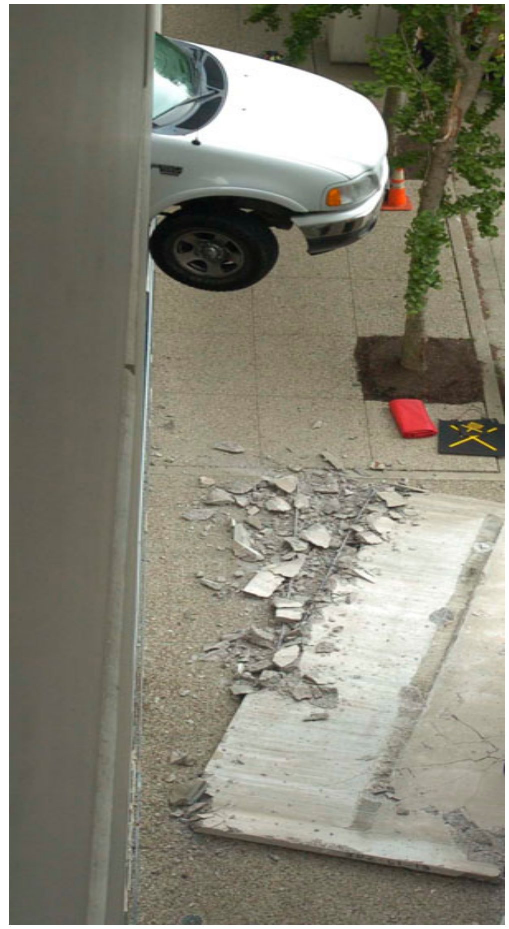
Reason: Purpose: The current code provisions need to be revised to assure that brittle or sudden failures of the barrier systems do not occur when the barrier is impacted by a vehicle.

Recent history of vehicle barrier systems in parking structures has shown that barriers have failed when they have not be designed according to the code, or when impact has caused brittle and sudden failures of the barrier system connections to the primary structure—particularly when drilled-in anchorages have been used to connect precast concrete panel barriers to the primary structure.

Cast-in-place concrete vehicle barriers have performed very well, even when impacted at a high speed, because of the ruggedness and ductility of the design.

In May 2006, in Lexington, Kentucky, a Ford F-150 pick-up truck impacted a precast barrier panel at approximately 4 miles-per-hour (as indicated by the impact recording system in the vehicle). The panel dislodged from the building and killed a pedestrian on the sidewalk below. The drilled-in expansion anchors had a clean non-ductile release from the supporting reinforced concrete structure as illustrated in the following photographs:.





Substantiation: In the 1960's and 1970's, a number of accidents occurred in parking garages and open parking structures where passenger vehicles went through the exterior walls and often over the edge of the parking facility with severe injury and often death to the vehicle occupants. These events coincided with the building boom of self-park parking facilities where the driver parked his/her own vehicle.

At that time, some of the state and city building codes had design requirements for the barrier restraints, sometimes called bumper walls or guard rails. However, the commonly used model building codes such as the Uniform Building Code (UBC) by the International Conference of Building Officials mainly used in the West, the BOCA Code by the Building Officials & Code Administrators International used in the Midwest and East, and the Standard Building Code (SBC) by the Southern Building Code Congress International used mainly in the Southeast had no specific provisions for the design of barrier restraints in multistory parking facilities. Several state codes including the New York, Wisconsin, Kentucky and Ohio codes did have barrier restraint requirements. Ohio requirements were 500 pounds per lineal foot at 18 inches above the floor at the ends of parking spaces and 1000 pounds per lineal foot at 18 inches above the floor at the ends of drive aisles.

To fill this lack of consensus on the proper method to design parking facility barrier restraints, the Parking Consultants Council (PCC) of the National Parking Association (NPA) formed in the mid 1970's a Building Code Committee to develop *Recommended Building Code Provisions for Open Parking Structures.* This document was published in July 1980.

Regarding barrier restraints, the committee made a survey of NPA members, who are mainly parking facility operators, asking for information and experience with barrier restraint failures. This information showed that where rational design methods had been used with as low as a 2000 pound horizontal load applied against a barrier in a parking space, no failures had occurred. However, failures had occurred where unreinforced masonry walls, pipe railings, precast concrete wheel stops, and similar restraints had been used.

The PCC Building Code Committee also obtained proprietary test data of mid-1970's vintage from the Automotive Research Laboratories at the University of Michigan, Ann Arbor, Michigan. This testing was for the energy absorption of passenger vehicle bumper systems. The goal of the testing was to set a standard for the manufacture of passenger vehicle bumper systems such that for a vehicle striking a wall in a perpendicular manner at a maximum speed of 5 miles-per-hour, it would sustain little or no damage. Also, the maximum weight of a passenger vehicle at that time was approximately 5000 pounds. Based on this information and with the assistance of the Structural Engineering Department at the University of Michigan, a static ultimate horizontal design point load of 10,000 pounds located 18 inches above the floor was developed as the criteria for the design of parking structure barrier restraint systems.

It should be noted that the act of a bumper wall resisting a vehicle striking it is truly a dynamic energy problem—not a static load problem. However, building codes at that time used percentages of static loads to allow for the impact effects on structures. Thus, the use of the 10,000 pound ultimate horizontal static load was deemed appropriate for a 5,000-pound vehicle traveling at a speed of 5 mph.

Therefore, in 1980, the PCC Code Committee developed the following for the design of barrier restraints, "*Barrier railings should be placed at the ends of drive lanes and at the ends of parking spaces at the perimeter of the structure and at the end of parking spaces where the difference in floor elevation is greater than one foot. Barrier railings should be not less than two feet in height and should be designed for a minimum horizontal ultimate load of 10,000 pounds applied at a height of one foot six inches above the floor at any point along the structure."* A footnote stated, "*It is the intent that the horizontal load be considered as applied over a one-foot square area with the load distributed through the barrier railing system into the main structural elements in a manner which is logical and appropriate for the barrier railing system under consideration."*

The PCC barrier rail recommendation was first adopted by the ICBO in the 1990 UBC Supplement. Many multistory parking structures designed prior to 1990 did not meet this requirement. Similar language was incorporated into a number of the model building codes with, in some cases, the load being changed from a 10,000 pound ultimate load to a 6,000 pound service load. The 6000 pound service load with the proper load factor is approximately the same as the 10,000 pound ultimate or factored load.

Barrier Restraint Modifications

During the 10 year period from 1996 to 2006, 14 incidents have been documented (see Table 1) where standard automobiles impacted barrier walls, rails or restraints of parking garages with such force that the barrier systems failed resulting in the deaths of 16 people. A number of the accidents appear to have occurred when the driver hit the accelerator rather than the brake pedal. Most of the failures were in parking structures designed and built prior to the 1980 design recommendations or prior to the 1990 code requirement, and had inadequate barrier restraints including faulty installation of barrier cables and unreinforced masonry walls. Wheel stops or curbs used in many of these facilities were ineffective at stopping the vehicle. Those failures have caused the Parking Consultants Council of the National Parking Association to re-evaluate the design requirements for barrier rail systems.

The characteristics of the passenger vehicle have changed dramatically in the last 27 years. Approximately 50% of the passenger vehicles sold in 2006 consist of light trucks (less than 10,000 pounds Gross Vehicle Weight), vans or SUV's. Those vehicles have a bumper height well in excess of the 18 inch height of load application required by the current building code. An analysis of automobile sales data (see Table 2) indicates the current code requirement of 18 inches only covers 52% of the vehicles. A bumper height of 27 inches would cover 96% of passenger vehicles.

In addition to revising the bumper height used in designing barrier restraints for parking structures, the applied load may also need to be revised. Light trucks and SUV's are heavier than the typical automobile. The empty weight of a Lincoln Navigator, a large sport utility vehicle (SUV), is approximately 7,000 pounds. Some large pick-up trucks have gross (loaded) weights of up to 10,000 pounds.

Also, the speed at point of impact may have to be reconsidered. At least one of the failure incidents reported a speed of 10 to 14 mph compared to a speed of 5 mph used to determine the current load requirement.

Finally, the design methodology may have to be revised. The key to vehicle impact restraint design is to absorb and dissipate the kinetic energy created by the moving vehicle impacting the barrier. The kinetic energy is created by a combination of the vehicle speed or velocity and the vehicle weight where $KE=1/2mv^2$.

This energy is absorbed by a combination of:

• The weight of the resisting element such as a concrete bumper wall,
• the instantaneous elastic or plastic deflection of the wall,
• the crushing or movement of the vehicle components such as bumper energy absorption system, crushing of vehicle fenders, etc.

This is a complex dynamics problem—not unlike designing a building structure for an earthquake. Therefore, it is important that connections of the barrier system to the supporting structure are designed to be ductile in accordance with the requirements of Chapter 16 of the International Building Code.

Summary

A review of the history of vehicle barrier restraint systems shows that systems designed for the 10,000 pound horizontal ultimate static impact load are adequate if proper provision is made to provide toughness and ductility in the barrier restraints and the related connection systems. The connections must be able to extend and deform to absorb impact energy prior to ultimate failure or disconnecting.

Strand or cable barrier systems can perform adequately if they are properly designed, installed, and maintained.

Recent vintage passenger vehicles including SUV's and pick-up trucks are heavier than their predecessors with average bumper heights greater than 18 inches. Loading and height adjustments should be made to provide proper barrier restraint for these heavier and taller vehicles.

Table 1. Parking Structure Vehicle Barrier Failure Incidents

	Facility Name & Location	Year of Incident	Year Built	Barrier Type	Description of Incident
1	Second & Union, Seattle, WA	1987	1969	Concrete curb and cables	3 dead from vehicle falling from fifth floor
2	Claridge Casino, Atlantic City, NJ	1996	1996	Cable Rail	2 dead in vehicle fall from 4th floor, faulty cable installation
3	Pittsburgh, PA	1999	1965	Wheel stops and 3' metal panel	Woman survived vehicle fall from 7th floor
4	Sandcastle Resort, Virginia Beach, VA	2000	1985	Concrete block wall	4 dead in vehicle fall from 5th floor
5	Howard Johnson's Hotel, Ocean City, MD	2002	Unknown	Wheel stops and Cable Rail	2 dead in vehicle fall from 4th floor
6	Golden Nugget, Las Vegas, NV	Jan. 2004	Unknown	Concrete curb and wall	2 dead in vehicle fall from 4th floor
7	City Park Mall, Ft. Lauderdale, FL	2004	1982	Concrete block wall	1 dead in vehicle fall from 5th floor
8	Golden Nugget, Las Vegas, NV	Oct. 2004	Unknown	Concrete curb and wall	2 seriously injured in vehicle fall from 2nd floor
9	Miami, FL	2004	Unknown	Concrete wall	Man injured in vehicle fall from 5th floor
10	Riverpark Square, Spokane, WA	2006	1973	Wheel stops and concrete spandrel wall	1 dead in vehicle fall from 5th floor
11	Lexington, KY	2006	1975	Precast concrete spandrels	Pedestrian killed on sidewalk when spandrel fell from garage after vehicle impact
12	Los Angeles, CA	2007	Unknown	Unknown	Woman injured in vehicle fall from 4th floor
13	Houston, TX	2007	Unknown	Masonry Wall	1 dead in vehicle fall from 5th floor
14	Chumash Casino, CA	2007	Unknown	Concrete Wall	Concrete wall damaged severly, but did not fail. No injuries.

Source: Parking Consultants Council of the National Parking Association, August 2007

Table 2. Bumper Height Analysis for 2007 Car, Truck, SUV and Minivan Models							
July 26th, 2007							
2007 Vehicle Models	Curb Weight (lb)	Payload (lb)	Gr. Veh. Wt (lb)	Bumper Middle Point Height (in)	2006 Vehicle Sales	Percentile	Notes
GMC Acadia	5,070	1,320	6,390	10	480	0.00%	
GMC Yukon XL	5,935	1,460	7,395	14	45,413	0.28%	
Dodge Ram 3500	6,588	2,300	8,888	14	182,089	1.37%	
GMC Sierra 1500	5,360	1,570	6,930	15	210,736	2.64%	
GMC Yukon	5,715	1,580	7,295	16	71,476	3.07%	
Lincoln Navigator	6,245	1,525	7,770	17	23,947	3.21%	
Mercedes-Benz R-Class	5,120	1,060	6,180	18	18,168	3.32%	
Car Models (175)	N/A	N/A	N/A	18	8,129,582	52.25%	Car Models (175), the current code requirement
Dodge Grand Caravan	4,515	1,185	5,700	19	211,140	53.53%	
Chrysler Town & Country	4,515	1,185	5,700	19	159,105	54.48%	
Mercedes-Benz M-Class	4,845	1,165	6,010	19	31,632	54.67%	
Honda Odyssey	4,615	1,320	5,935	19	177,919	55.74%	
Toyota Sienna	4,415	1,120	5,535	19	163,269	56.73%	
Chrysler Aspen	5,335	1,260	6,595	20	7,656	56.77%	
Ford Explorer	4,905	1,275	6,180	21	179,229	57.85%	
Chevrolet Express	5,015	3,254	8,269	22	123,195	58.59%	
Chevrolet Equinox	3,880	1,115	4,995	22	113,888	59.28%	
Chevrolet Trailblazer	4,830	1,020	5,850	23	174,797	60.33%	
Ford Econoline	5,505	3,215	8,720	23	180,457	61.42%	
Honda CRV	3,505	850	4,355	23	170,028	62.44%	
Ford Escape	3,575	950	4,525	23	157,395	63.39%	
Toyota RAV 4	3,485	825	4,310	23	152,047	64.30%	
GMC Sierra 2500	6,000	3,795	9,795	23	105,368	64.94%	
Cadillac Escalade	5,810	1,330	7,140	24	62,206	65.31%	
Chevrolet Avalanche	6,010	1,230	7,240	24	57,076	65.66%	
Chevrolet Suburban	5,935	1,460	7,395	24	77,211	66.12%	
Chevrolet Tahoe	5,715	1,580	7,295	24	161,491	67.09%	
Mercedes-Benz GL-Class	5,575	1,210	6,785	24	18,776	67.21%	
Volvo XC90	4,950	1,210	6,160	24	33,200	67.40%	
Toyota Highlander	4,035	1,160	5,195	24	129,794	68.19%	
Lexus RX	4,235	925	5,160	24	108,348	68.84%	
Toyota 4 Runner	4,345	1,035	5,380	24	103,086	69.46%	
Hummer H3	4,700	1,150	5,850	24	54,052	69.78%	
Chevrolet Silverado 1500	5,360	1,570	6,930	25	636,069	73.61%	
Dodge Durango	5,335	1,260	6,595	25	70,606	74.04%	
Dodge Ram 1500	5,300	1,350	6,650	25	182,089	75.13%	
Ford Expedition	6,245	1,570	7,815	25	87,203	75.66%	
Toyota Tundra	5,740	1,395	7,135	25	124,508	76.41%	
Volkswagen Touareg	5,210	1,280	6,490	25	10,163	76.47%	
Jeep Grand Cherokee	4,725	1,100	5,825	25	139,148	77.31%	
Nissan Pathfinder	4,875	1,125	6,000	26	73,124	77.75%	
Nissan Titan	5,380	1,105	6,485	26	72,192	78.18%	
Honda Pilot	4,535	1,320	5,855	26	152,154	79.10%	
Jeep Liberty	4,125	1,150	5,275	26	133,557	79.90%	
Ford F-150	5,620	1,510	7,130	27	398,020	82.30%	
Jeep Commander	5,245	1,100	6,345	27	88,497	82.83%	
Nissan Armada	5,715	1,375	7,090	27	32,864	83.03%	85th Percentile Vehicle
87 Additional Truck/SUV/Minivan Models	N/A	N/A	N/A	27	2,188,867	96.20%	87 Additional Truck/SUV/Minivan Models
Hummer H2	6,400	2,200	8,600	27	17,107	96.30%	
Ford F-250	8,080	1,905	9,985	28	398,020	98.70%	
Toyota Sequoia	5,280	1,320	6,600	28	34,315	98.91%	
Toyota Tacoma	4,115	1,100	5,215	28	178,351	99.98%	
Toyota Land Cruiser	5,435	1,240	6,675	29	3,376	100.00%	
Total 2006 Vehicle Sales					**16,614,484**		

Total Number of Vehicles	16,614,484	
Number of vehicles that would be included when using the 85th percentile bumper height	15,983,316	96%
Number of vehicles covered by the current requirement of 18 inches	8,681,891	52%
Number of vehicles not covered by the current code provisions	7,932,594	48%
Number of vehicles not covered by the proposed code Including data for the additional 87 models for LTVSUV's that are also 27 inches	631,169	4%

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

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

S74–07/08
1607.9

Proponent: Edwin Huston, National Council of Structural Engineers Association (NCSEA), representing National NCSEA Code Advisory Committee – General Engineering Subcommittee

Revise as follows:

1607.9 Reduction in live loads. Except for roof uniform live loads, all other minimum uniformly distributed live loads, *Lo*, in Table 1607.1 are permitted to be reduced in accordance with Section 1607.9.1 or 1607.9.2. Roof uniform live loads, other than special purpose roofs of Section 1607.11.2.2 are permitted to be reduced in accordance with Section 1607.11.2. Roof uniform live loads of special purpose roofs are permitted to be reduced in accordance with Section 1607.9.1 or 1607.9.2.

Reason: There is a conflict between sections 1607.9, which says that roof live loads can not be reduced using the method of 1607.9 and 1607.9.1.4, which says that roof live loads can be reduced. This proposal eliminates that conflict and points the user to the roof live load reduction section.

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

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

S75–07/08
1607.9.1, Table 1607.9.1, 1607.9.1.1, 1607.9.1.2, 1607.9.1.3, 1607.9.1.4, 1607.9.1.5 (New)

Proponent: Philip Brazil, PE, SE, Reid Middleton, Inc., representing himself

Revise as follows:

1607.9.1 General. Subject to the limitations of Sections 1607.9.1.1 through 1607.9.1.4, members for which a value of $K_{LL}A_T$ is 400 square feet (37.16 m^2) or more are permitted to be designed for a reduced live load in accordance with the following equation:

(No changes to equation 16-24)

where:

L = Reduced design live load per square foot (meter) of area supported by the member.
L_o = Unreduced design live load per square foot (meter) of area supported by the member (see Table 1607.1).
K_{LL} = Live load element factor (see Table 1607.9.1).
A_T = Tributary area, in square feet (square meters).

~~*L* shall not be less than 0.50*Lo* for members supporting one floor and *L* shall not be less than 0.40*Lo* for members supporting two or more floors.~~

L shall not be less than 0.50 L_o for members supporting one floor and *L* shall not be less than 0.40 L_o for members supporting two or more floors.

TABLE 1607.9.1
LIVE LOAD ELEMENT FACTOR, K_{LL}

ELEMENT	K_{LL}
Interior columns Exterior columns without cantilever slabs	4 4
Edge columns with cantilever slabs	3
Corner columns with cantilever slabs Edge beams without cantilever slabs Interior beams	2 2 2
All other members not identified above including: Edge beams with cantilever slabs Cantilever beams <u>One-way slabs</u> Two-way slabs Members without provisions for continuous shear transfer normal to their span	1

~~**1607.9.1.4**~~ <u>**1607.9.1.1**</u> ~~**Special structural elements**~~ <u>**One-way slabs**</u>.~~Live loads shall not be reduced for one-way slabs except as permitted in Section 1607.9.1.1.~~ <u>The tributary area, A_T, for one-way slabs shall not exceed an area defined by the slab span times a width normal to the span of 1.5 times the slab span</u>. ~~Live loads of 100 psf (4.79 kN/m^2) or less shall not be reduced for roof members except as specified in Section 1607.11.2~~

1607.9.1.~~1~~.<u>2</u> Heavy live loads. Live loads that exceed 100 psf (4.79 kN/m2) shall not be reduced.

Exceptions:

1. The live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall not be less than L as calculated in Section 1607.9.1.
2. For uses other than storage, where approved, additional live load reductions shall be permitted where shown by the registered design professional that a rational approach has been used and that such reductions are warranted.

1607.9.1.~~2~~.<u>3</u> Passenger vehicle garages. The live loads shall not be reduced in passenger vehicle garages<u>.</u> ~~except the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall not be less than *L* as calculated in Section 1607.9.1.~~

<u>**Exception:** The live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent, but the live load shall not be less than *L* as calculated in Section 1607.9.1.</u>

1607.9.1.~~3~~.<u>4</u> ~~Special~~ <u>Group A</u> occupancies. Live loads of 100 psf (4.79 kN/m^2) ~~or less~~ shall not be reduced in ~~public assembly~~ <u>Group A</u> occupancies.

<u>**1607.9.1.5 Roofs members.** Live loads of 100 psf (4.79 kN/m^2) or less shall not be reduced for roof members except as specified in Section 1607.11.2.</u>

Reason: The purpose of this proposal is to align IBC Section 1607.9.1 on the general method for floor live load reductions with similar provisions in Section 4.8 of ASCE 7-05 and to make related editorial revisions.

"One way slabs" is added to Table 1607.9.1 for consistency with Table 4-2 of ASCE 7.05. The second part of Section 1607.9.1.2 (Section 1607.9.1.3 in proposal) is reformatted into an exception for consistency with Section 4.8.3 of ASCE 7-05.

Section 1607.9.1.3 (Section 1607.9.1.4 in proposal) on public assembly occupancies is changed to Group A occupancies, thus, replacing language that is vague and unenforceable with a classification that is defined by the IBC (refer to Section 303). Public assembly occupancies could be interpreted as other than Group A occupancies but they would typically have an occupant load of less than 50 (i.e., Exception 1 to Section 303.1) and a prohibition on live load reduction is not judged to be warranted in such cases.

Also in Section 1607.9.1.3, "or less" is deleted, which reduces the scope of the section to live loads of 100 psf. Several items in Table 1607.1 list live loads for areas of public assembly that could be classified as a Group A occupancy, including Items #3 (armories and drill rooms), #4 (assembly areas and theaters), #6 (balconies), #8 (dance halls and ballrooms), #10 (dining rooms and restaurants), #19 (gymnasiums), #26 (lobbies of office buildings) and #28 (residential public rooms). A live load of at least 100 psf is specified for all but areas of fixed seats at Item #4. Prohibiting reductions in live loads at areas of fixed seats is not judged to be warranted. Live loads greater than 100 psf are currently covered by Section 1607.9.1.1 on heavy live loads.

Section 1607.9.1.4 is split into two parts. The first part on one-way slabs is relocated to a new Section 1607.9.1.1 and is changed from a general prohibition on live load reduction (except for heavy live loads) to a limit on the determination of tributary area, A_T, in the same manner as specified in Section 4.8.4 of ASCE 7-05. The relocation to Section 1607.9.1.1 is proposed because the subject matter of tributary area logically follows the calculation of reduced live load, which is based on tributary area. The sections that follow Section 1607.9.1.1 are largely prohibitions on live load reduction. The second part of Section 1607.9.1.4 is renumbered as Section 1607.9.1.5.

This proposal was prepared in conjunction with related proposals on reduction of roof live loads, live loads at marquees, and reduction of live loads at roofs used for assembly purposes.

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

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

S76–07/08
1607.9, 1607.11.1, 1607.11.2

Proponent: Philip Brazil, PE, SE, Reid Middleton, Inc., representing himself

Revise as follows:

1607.9 Reduction in live loads. Except for ~~roof~~ uniform live loads at roofs and marquees, all other minimum uniformly distributed live loads, L_o, in Table 1607.1 are permitted to be reduced in accordance with Section 1607.9.1 or 1607.9.2.

1607.11.1 Distribution of roof loads. Where uniform roof live loads are reduced to less than 20 psf (0.96 kN/m^2) in accordance with Section 1607.11.2.1 and are involved in the design of structural members arranged so as to create continuity, the minimum applied loads shall be the full dead loads on all spans in combination with the roof live loads on adjacent spans or on alternate spans, whichever produces the greatest effect. See Section 1607.11.2 for reductions in minimum roof live loads and Section 7.5 of ASCE 7 for partial snow loading.

1607.11.2 (Supp) Reduction in roof live loads. The minimum uniformly distributed ~~roof~~ live loads of roofs and marquees, L_o, in Table 1607.1 are permitted to be reduced in accordance with Section 1607.11.2.1 or 1607.11.2.2.

1607.11.2.1 Flat, pitched and curved roofs. Ordinary flat, pitched and curved roofs, and awnings and canopies other than of fabric construction supported by lightweight rigid skeleton structures, are permitted to be designed for a reduced roof live load as specified in the following equation or other controlling combinations of loads in Section 1605, whichever produces the greater load. In structures where special scaffolding is used as a work surface for workers and materials during maintenance and repair operations, a lower roof load than specified in the following equation shall not be used unless approved by the building official. Greenhouses shall be designed for a minimum roof live load of 12 psf (0.58 kN/m^2).

(No changes to equations or to their notation)

1607.11.2.2 Special-purpose roofs. Roofs used for promenade purposes, roof gardens, assembly purposes or other special purposes, and marquees, shall be designed for a minimum live load as required in Table 1607.1. Such ~~roof~~ live loads are permitted to be reduced in accordance with 1607.9.

Reason: Section 1607.9 on reductions in live loads applies to all uniformly distributed live loads specified in Table 1607.1 except for roof uniform live loads. Item #30 of Table 1607.1 specifies uniformly distributed live loads for roofs but Item #24 specifies the same for marquees. Section 1607.11 on roof loads includes marquees in its charging language but the technical provisions in the remainder of the section are silent on marquees. The purpose of this proposal is to revise the charging language in Sections 1607.9 and 1607.11 to comprehensively account for marquees. Note that Section 1607.11.2.1 is limited in scope to a reduction in roof live load based on the slope of the roof. Section 1607.11.2.2 is limited in scope to specific types of special purpose roofs, each one of which is also specified in Item #30 of Table 1607.1. The proposal adds marquees to Section 1607.11.2.2.

The proposal also corrects an inadvertent omission of awnings and canopies other than of fabric construction supported by a lightweight rigid skeleton structure from qualifying for a reduction in roof live load due to roof slope. Section 1607.11.2.1 is limited in scope to ordinary flat, pitched and curved roofs, which are one of the listings in Item #30 of Table 1607.1 for roofs. The presence of this listing effectively eliminates awnings and canopies from qualifying for a reduction in roof live due to roof slope because their listing in Item #30 is separate and distinct from flat, pitched and curved roofs. The proposal corrects this oversight. This came about when the addition of Item #30 to Table 1607.1 was approved by Proposal S20-04/05-AM. Before that, Item #29 in Table 1607.1 of the 2003 IBC for roofs referenced Section 1607.11, which specified provisions for reductions of roof live loads at ordinary flat, pitched and curved roofs in Section 1607.11.2.1.

This proposal was prepared in conjunction with related proposals on reduction of floor live loads, reduction of roof live loads, and reduction of live loads at roofs used for assembly purposes.

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

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

S77–07/08
1607.9.2

Proponent: Edwin Huston, National Council of Structural Engineers Association (NCSEA), representing National NCSEA Code Advisory Committee – General Engineering Subcommittee

Revise as follows:

1607.9.2 Alternate floor live load reduction. As an alternative to Section 1607.9.1, floor live loads are permitted to be reduced in accordance with the following provisions. Such reductions shall apply to slab systems, beams, girders, columns, piers, walls and foundations.

1. A reduction shall not be permitted in Group A occupancies.
2. A reduction shall not be permitted where the live load exceeds 100 psf (4.79 kN/m^2) except that the design live load for members supporting two or more floors is permitted to be reduced by 20 percent.

   Exception: For uses other than storage, where approved, additional live load reductions shall be permitted where shown by the registered design professional that a rational approach has been used and that such reductions are warranted.

3. A reduction shall not be permitted in passenger vehicle parking garages except that the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent.
4. For live loads not exceeding 100 psf (4.79 kN/m^2), the design live load for any structural member supporting 150 square feet (13.94 m^2) or more is permitted to be reduced in accordance with the following equation:

Reason: This additional text will align the high live load reduction requirements when using Section 1607.9.2 “Alternate Floor Live Load Reduction” with text already in Section 1607.9.1.1 “Heavy Live Loads” when using the Basic Floor Live Load Reduction.

Cost Impact: The code change proposal will not increase the cost of construction (will possibly reduce construction cost in some instances).

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

S78–07/08
1607.11.3

Proponent: Mark S. Graham, National Roofing Contractors Association, representing Technical Operations Committee of the National Roofing Contractors Association

Revise as follows:

1607.11.3 (Supp) Landscaped roofs. Where roofs are to be landscaped, the minimum uniform design live load in the landscaped area shall be ~~20~~ 100 psf ~~(0.958 kN/m^2)~~. The weight of the landscaping materials shall be considered as dead load and shall be computed on the basis of saturation of the soil.

Reason: This proposed code change is intended to clarify the intent of the Code as it relates to minimum live load requirements applicable to roof gardens and landscaped roofs (also, commonly referred to as vegetative roofs or greens roofs). Section 1607.11.2.3—Landscaped Roofs currently indicates a live load 20 psf for rooftop landscaped areas, while Table 1607.1—Minimum Uniformly Distributed Live Loads and Minimum Concentrated Live Loads indicates a minimum live load of 100 psf for roof gardens.

As a solution to this apparent conflict, this proposal revises Section 1607.11.2.3 to require a minimum 100 psf live load for rooftop landscaped areas. This is consistent with Table 1607.1.

We have also submitted a companion proposal to this proposed code change that, as an alternative, revises Table 1607.1 to a 20 psf live load, making it consistent with the current Section 1607.11.2.3. We ask the code development committee and ICC membership to approve one or the other of these proposals to clarify the Code regarding this apparent conflict.

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

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

S79–07/08
1609.1.1, 1609.1.1.1, 2308.2.1, Chapter 35; IRC R301.2.1.1, Chapter 43

Proponent: Med Kopczynski, City of Keene, NH, representing ICC IS-HRC Standards Committee

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STRUCTURAL AND THE IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

1. Revise as follows:

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

Exceptions:

1. Subject to the limitations of Section 1609.1.1.1, the provisions of ~~SBCCI SSTD 10~~ <u>ICC-600</u> shall be permitted for applicable Group R-2 and R-3 buildings.
2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of the AF&PA WFCM.
3. Designs using NAAMM FP 1001.
4. Designs using TIA-222 for antenna-supporting structures and antennas.
5. Wind Tunnel tests in accordance with Section 6.6 of ASCE 7, subject to the limitations in Section 1609.1.1.2.

1609.1.1.1 Applicability. The provisions of ~~SSTD 10~~ <u>ICC-600</u> are applicable only to buildings located within Exposure B or C as defined in Section 1609.4. The provisions of ~~SBCCI SSTD 10~~ <u>ICC-600</u> and the AF&PA WFCM shall not apply to buildings sited on the upper half of an isolated hill, ridge or escarpment meeting the following conditions:

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

2308.2.1 Basic wind speed greater than 100 mph (3-second gust). Where the basic wind speed exceeds 100 mph (3-second gust), the provisions of either AF&PA WFCM, or the ~~SBCCI SSTD 10~~ <u>ICC-600</u> are permitted to be used.

2. Revise standards as follows:

International Code Council (ICC)
~~SBCCI SSTD 10-99 Standard for Hurricane Resistance Residential Construction~~
<u>ICC-600</u> <u>Standard for Residential Construction in High Wind Regions</u>

PART II – IRC BUILDING/ENERGY

1. Revise as follows:

R301.2.1.1 (Supp) Design criteria. In regions where the basic wind speeds from Figure R301.2(4) equal or exceed 100 miles per hour (45 m/s) in hurricane-prone regions, or 110 miles per hour (49 m/s) elsewhere, the design of buildings shall be in accordance with one of the following methods. The elements of design not addressed by those documents in Items 1 through 4 shall be in accordance with this code.

1. American Forest and Paper Association (AF&PA) *Wood Frame Construction Manual for One- and Two-Family Dwellings* (WFCM); or

2. ~~*Southern Building Code Congress International Standard for Hurricane Resistant Residential Construction* (SSTD 10);~~ *International Code Council (ICC) Standard for Residential Construction in High Wind Regions (ICC-600);* or
3. *Minimum Design Loads for Buildings and Other Structures* (ASCE-7); or
4. American Iron and Steel Institute (AISI), *Standard for Cold-Formed Steel Framing—Prescriptive Method For One- and Two-Family Dwellings (COFS/PM) with Supplement to Standard for Cold-Formed Steel Framing—Prescriptive Method For One- and Two-Family Dwellings*.
5. Concrete construction shall be designed in accordance with the provisions of this code.
6. Structural insulated panels shall be designed in accordance with the provisions of this code.

2. Revise standards as follows:

International Code Council (ICC)

~~SBCCI SSTD 10-99 Standard for Hurricane Resistance Residential Construction~~

ICC-600 Standard for Residential Construction in High Wind Regions

Reason: This proposal is to delete the current ICC legacy Standard SSTD 10 – 99 and replace with the new ICC– 600 Standard for Residential Construction in High Wind Regions.

The ICC legacy standard SSTD 10 – 99 and its predecessors were the first US standards for high wind construction of residential structures. The SSTD 10 is based on the Standard Building Code wind loads and which used fastest-mile wind speeds. Although dated, the SSTD 10 is referenced by the IBC and IRC.

The new ICC– 600 standard provides a set of specifications that is consistent with the International Building Code and ASCE 7 wind loads, wind speed maps, and conventions. The primary focus of the update effort has been to provide a contemporary set of prescriptive requirements that supplement the International Residential Code provisions.

The ICC– 600 was developed by the ICC Consensus Committee on Hurricane Resistant Construction (IS-HRC) that operates under ANSI Approved ICC Consensus Procedures. A copy of a draft of the standard has been submitted to the ICC as allowed by ICC Council Policy; CP#28.ANSI certification of the standard is expected to be received prior to the ICC Final Action Hearings in September 2008.

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

Analysis: A review of the standard(s) proposed for inclusion in the code, ICC 600, for compliance with ICC criteria for referenced standards given in Section 3.6 of Council Policy #CP 28 will be posted on the ICC website on or before January 15, 2008.

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

S80–07/08

1609.1.1, 1609.1.1.1

Proponent: Bonnie Manley, American Iron and Steel Institute, representing American Iron and Steel Institute

Revise as follows:

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

Exceptions:

1. Subject to the limitations of Section 1609.1.1.1, the provisions of SBCCI SSTD 10 shall be permitted for applicable Group R-2 and R-3 buildings.
2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of the AF&PA WFCM.
3. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AISI S230.

4~~3~~. Designs using NAAMM FP 1001.
5~~4~~. Designs using TIA-222 for antenna-supporting structures and antennas.
6~~5~~. Wind Tunnel tests in accordance with Section 6.6 of ASCE 7, subject to the limitations in Section 1609.1.1.2.

1609.1.1.1 Applicability. The provisions of SSTD 10 are applicable only to buildings located within Exposure B or C as defined in Section 1609.4. The provisions of SBCCI SSTD 10, ~~and the~~ AF&PA WFCM and AISI S230 shall not apply to buildings sited on the upper half of an isolated hill, ridge or escarpment meeting the following conditions:

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

Reason: The 2006 IBC recognizes the use of the *AISI Standard for Cold-formed Steel Framing- Prescriptive Method for One- and Two-family Dwellings* in Section 2210.6. In fact, these prescriptive requirements form the basis for the cold-formed steel light frame construction provisions in the IRC. Additionally, the document, which addresses wind speeds up to 150 MPH, has been included by reference in the new ICC-600. Therefore, it is appropriate to further integrate this document as an acceptable method to address wind load requirements by recognizing its applicability and limitations in Section 1609.1.1. This code change references the new 2007 edition of AISI S230 standard, which is based on ASCE 7-05 wind provisions.

Throughout the IBC and IRC, code changes are being introduced to update the recognition of the AISI Prescriptive Method to the 2007 edition, which is identified by the AISI S230-07 designation. Details of the substantive changes between the 2004 AISI Supplement and A230-07 are contained in the supporting statement for AISI's code change proposal to update this reference in Section 2210.6.

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

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

S81-07/08
1609.1.1, 1609.1.1.2, Chapter 35 (New)

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

1. Revise as follows:

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

Exceptions:

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

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

2. Add standard to Chapter 35 as follows:

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

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

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

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

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

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

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

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

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

References:

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

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

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

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

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

S82–07/08

Table 1504.8, Table 1507.3.7, 1609.1.2, Table 1609.1.2 (Supp), 1609.4.3, Table 2308.10.1; IRC Table R301.2(2), Table 301.2(3), R301.2.1.2, Table R301.2.1.2, Table R602.3(1), Table R611.3(1), Table R611.7.4, Table R802.11, Table AH107.4(1)

Proponent: Philip Brazil, PE, SE, Reid Middleton, Inc., representing himself

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STRUCTURAL AND IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

TABLE 1504.8 (Supp)
MAXIMUM ALLOWABLE ~~MEAN ROOF~~ BUILDING HEIGHT PERMITTED FOR BUILDINGS WITH AGGREGATE ON THE ROOF IN AREAS OUTSIDE A HURRICANE-PRONE REGION

BASIC WIND SPEED FROM FIGURE 1609 (mph) ~~b~~ a	MAXIMUM ~~MEAN ROOF~~ BUILDING HEIGHT (ft) ~~a, c~~ b		
	Exposure Category		
	B	C	D
85	170	60	30
90	110	35	15
95	75	20	NP
100	55	15	NP
105	40	NP	NP
110	30	NP	NP
115	20	NP	NP
120	15	NP	NP
Greater than 120	NP	NP	NP

For SI: 1 foot = 304.8 mm; 1 mile per hour = 0.447 m/s.
~~a. Mean roof height as defined in ASCE 7.~~
~~b.~~a. For intermediate values of basic wind speed, the height associated with the next higher value of wind speed shall be used, or direct interpolation is permitted.
~~c.~~b. NP = gravel and stone not permitted for any roof height.

TABLE 1507.3.7
CLAY AND CONCRETE TILE ATTACHMENT[a, b, c]

GENERAL — CLAY OR CONCRETE ROOF TILE				
Maximum basic wind speed (mph)	~~Mean roof~~ Building height (feet)	Roof slope up to 3:12	Roof slope 3:12 and over	
Maximum basic wind speed (mph)	~~Mean roof~~ Building height (feet)	Roof slope up to 5:12	Roof slope 5:12 12:12	Roof slope 12:12 and over
Maximum basic wind speed (mph)	~~Mean roof~~ Building height (feet)	All roof slopes		

(Portions of table not shown remain unchanged)

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

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

Exceptions:

1. Wood structural panels with a minimum thickness of 7/16 inch (11.1 mm) and maximum panel span of 8 feet (2438 mm) shall be permitted for opening protection in one- and two-story buildings classified as Group R-3 or R-4 occupancy. Panels shall be precut so that they shall be attached to the framing surrounding the opening containing the product with the glazed opening. Panels shall be predrilled as required for the anchorage method and shall be secured with the attachment hardware provided. Attachments shall be designed to resist the components and cladding loads determined in accordance with the provisions of ASCE 7, with corrosion resistant attachment hardware provided and anchors permanently installed on the building. Attachment in accordance with Table 1609.1.2 with corrosion resistant attachment hardware provided and anchors permanently installed on the building is permitted for buildings with a ~~mean roof~~ building height of 45 feet (13716 mm) or less where wind speeds do not exceed 140 mph (63 m/s).
2. Glazing in Occupancy Category I buildings as defined in Section 1604.5, including greenhouses that are occupied for growing plants on a production or research basis, without public access shall be permitted to be unprotected.
3. Glazing in Occupancy Category II, III or IV buildings located over 60 feet (18 288 mm) above the ground and over 30 feet (9144 mm) above aggregate surface roofs located within 1,500 feet (458 m) of the building shall be permitted to be unprotected.

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

FASTENER TYPE	FASTENER SPACING (in.)		
	Panel span ≤ 4 foot	4 feet < panel span ≤ 6 feet	6 feet < panel span ≤ 8 feet
No. 8 wood-screw-based anchor with 2-inch embedment length	16	10	8
No. 10 wood-screw- based anchor with 2-inch embedment length	16	12	9
1/4 lag –screw-based anchor with 2-inch embedment length	16	16	16

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

a. This table is based on 140 mph wind speeds and a 45-foot ~~mean roof~~ building height.
b. Fasteners shall be installed at opposing ends of the wood structural panel. Fasteners shall be located a minimum of 1 inch from the edge of the panel.
c. Anchors shall penetrate through the exterior wall covering with an embedment length of 2 inches minimum into the building frame. Fasteners shall be located a minimum of $2^1/_2$ inches from the edge of concrete block or concrete.
d. Where panels are attached to masonry or masonry/stucco, they shall be attached using vibration-resistant anchors having a minimum ultimate withdrawal capacity of 1500 pounds.

1609.4.3 Exposure categories. An exposure category shall be determined in accordance with the following:

Exposure B. Exposure B shall apply where the ground surface roughness condition, as defined by Surface Roughness B, prevails in the upwind direction for a distance of at least 2,600 feet (792 m) or 20 times the height of the building, whichever is greater.

Exception: For buildings whose ~~mean roof~~ building height is less than or equal to 30 feet (9144 mm), the upwind distance is permitted to be reduced to 1,500 feet (457 m).

Exposure C. Exposure C shall apply for all cases where Exposures B or D do not apply.

Exposure D. Exposure D shall apply where the ground surface roughness, as defined by Surface Roughness D, prevails in the upwind direction for a distance of at least 5,000 feet (1524 m) or 20 times the height of the building, whichever is greater. Exposure D shall extend inland from the shoreline for a distance of 600 feet (183 m) or 20 times the height of the building, whichever is greater.

TABLE 2308.10.1
REQUIRED RATING OF APPROVED UPLIFT CONNECTORS (pounds)[a,b,c,e,f,g,h]

(Portions of table not shown remain unchanged)

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 mile per hour = 1.61 km/hr, 1 pound = 0.454 Kg, 1 pound/foot = 14.5939 N/m.

a. The uplift connection requirements are based on a 30-foot ~~mean roof~~ building height located in Exposure B. For Exposure C or D and for other ~~mean roof~~ building heights, multiply the above loads by the adjustment coefficients below.

	~~Mean Roof~~ Building Height (feet)									
EXPOSURE	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**	**55**	**60**

(Portions of table and footnotes not shown remain unchanged)

PART II – IRC BUILDING/ENERGY

Revise as follows:

TABLE R301.2(2) (Supp)
COMPONENT AND CLADDING LOADS FOR A BUILDING WITH A ~~MEAN ROOF~~ BUILDING HEIGHT OF 30 FEET LOCATED IN EXPOSURE B (psf)

(Portions of table and footnotes not shown remain unchanged)

TABLE R301.2(3)
HEIGHT AND EXPOSURE ADJUSTMENT COEFFICIENTS FOR TABLE R301.2(2)

~~MEAN ROOF~~ BUILDING HEIGHT	EXPOSURE		
	B	**C**	**D**

(Portions of table and footnotes not shown remain unchanged)

R301.2.1.2 (Supp) Protection of openings. Windows in buildings located in windborne debris regions shall have glazed openings protected from windborne debris. Glazed opening protection for windborne debris shall meet the requirements of the Large Missile Test of an approved impact resisting standard or ASTM E 1996 and ASTM E 1886 referenced therein. Garage door glazed opening protection for windborne debris shall meet the requirements of an approved impact resisting standard or ANSI/DASMA 115.

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

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

(Portions of table not shown remain unchanged)

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

a. This table is based on 130 mph wind speeds and a 33-foot ~~mean roof~~ building height.
b. Fasteners shall be installed at opposing ends of the wood structural panel. Fasteners shall be located a minimum of 1 inch from the edge of the panel.

c. Anchors shall penetrate through the exterior wall covering with an embedment length of 2 inches minimum into the building frame. Fasteners shall be located a minimum of $2^1/_2$ inches from the edge of concrete block or concrete.
d. Where panels are attached to masonry or masonry/stucco, they shall be attached using vibration-resistant anchors having a minimum ultimate withdrawal capacity of 1500 pounds.

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

(Portions of table not shown remain unchanged)

For SI: 1 inch = 25.4 mm, 1 foot = 304.8 mm, 1 mile per hour = 0.447 m/s; 1ksi = 6.895 MPa.

a. All nails are smooth-common, box or deformed shanks except where otherwise stated. Nails used for framing and sheathing connections shall have minimum average bending yield strengths as shown: 80 ksi for shank diameter of 0.192 inch (20d common nail), 90 ksi for shank diameters larger than 0.142 inch but not larger than 0.177 inch, and 100 ksi for shank diameters of 0.142 inch or less.
b. Staples are 16 gage wire and have a minimum 7/16-inch on diameter crown width.
c. Nails shall be spaced at not more than 6 inches on center at all supports where spans are 48 inches or greater.
d. Four-foot-by-8-foot or 4-foot-by-9-foot panels shall be applied vertically.
e. Spacing of fasteners not included in this table shall be based on Table R602.3(2).
f. For regions having basic wind speed of 110 mph or greater, 8d deformed ($2^1/_2$″×0.120) nails shall be used for attaching plywood and wood structural panel roof sheathing to framing within minimum 48-inch distance from gable end walls, if ~~mean roof~~ building height is more than 25 feet, up to 35 feet maximum.
g. For regions having basic wind speed of 100 mph or less, nails for attaching wood structural panel roof sheathing to gable end wall framing shall be spaced 6 inches on center. When basic wind speed is greater than 100 mph, nails for attaching panel roof sheathing to intermediate supports shall be spaced 6 inches on center for minimum 48-inch distance from ridges, eaves and gable end walls; and 4 inches on center to gable end wall framing.
h. Gypsum sheathing shall conform to ASTM C 79 and shall be installed in accordance with GA 253. Fiberboard sheathing shall conform to ASTM C 208.
i. Spacing of fasteners on floor sheathing panel edges applies to panel edges supported by framing members and required blocking and at all floor perimeters only. Spacing of fasteners on roof sheathing panel edges applies to panel edges supported by framing members and required blocking. Blocking of roof or floor sheathing panel edges perpendicular to the framing members need not be provided except as required by other provisions of this code. Floor perimeter shall be supported by framing members or solid blocking.

6. Revise as follows:

TABLE R611.3(1)
DESIGN WIND PRESSURE FOR USE WITH TABLES R611.3(2), R611.4(1), AND R611.5 FOR ABOVE GRADE WALLS[a]

(Portions of table not shown remain unchanged)

For SI: 1 pound per square foot = 0.0479 kPa; 1 mile per hour = 0.447 m/s; 1 foot = 304.8 mm; 1 square foot = 0.0929 m^2.

a. This table is based on ASCE ~~7-98~~ 7 components and cladding wind pressures using a ~~mean roof~~ building height of 35 ft and a tributary area of 10 ft^2.
b. Buildings in wind-borne debris regions as defined in Section R202 shall be considered as “Partially Enclosed” unless glazed openings are protected in accordance with Section R301.2.1.2, in which case the building shall be considered as “Enclosed.” All other buildings shall be classified as “Enclosed.”
c. Exposure Categories shall be determined in accordance with Section R301.2.1.4.
d. For wind pressures greater than 80 psf, design is required in accordance with ACI 318 and approved manufacturer guidelines.
e. Interpolation is permitted between wind speeds.

TABLE R611.7.4
WIND VELOCITY PRESSURE FOR DETERMINATION OF MINIMUM SOLID WALL LENGTH[a]

(Portions of table not shown remain unchanged)

For SI: 1 pound per square foot = 0.0479 kPa; 1 mile per hour = 0.447 m/s.

a. Table values are based on ASCE ~~7-98~~ 7-05 Figure ~~6-4~~ 6-10 using a ~~mean roof~~ building height of 35 ft.
b. Exposure Categories shall be determined in accordance with Section R301.2.1.4.
c. Design is required in accordance with ACI 318 and approved manufacturer guidelines.
d. Interpolation is permitted between wind speeds.

TABLE R802.11
REQUIRED STRENGTH OF TRUSS OR RAFTER CONNECTIONS TO RESIST WIND UPLIFT FORCES[a, b, c, e, f]
(Pounds per connection)

(Portions of table not shown remain unchanged)

For SI: 1 inch = 25.4 mm, 1 foot = 305 mm, 1 mph = 0.447 m/s, 1 pound/foot = 14.5939 N/m, 1 pound = 0.454 kg.

a. The uplift connection requirements are based on a 30 foot ~~mean roof~~ building height located in Exposure B. For Exposures C and D and for other ~~mean roof~~ building heights, multiply the above loads by the Adjustment Coefficients in Table R301.2(3).
b. The uplift connection requirements are based on the framing being spaced 24 inches on center. Multiply by 0.67 for framing spaced 16 inches on center and multiply by 0.5 for framing spaced 12 inches on center.
c. The uplift connection requirements include an allowance for 10 pounds of dead load.
d. The uplift connection requirements do not account for the effects of overhangs. The magnitude of the above loads shall be increased by adding the overhang loads found in the table. The overhang loads are also based on framing spaced 24 inches on center. The overhang loads given shall be multiplied by the overhang projection and added to the roof uplift value in the table.
e. The uplift connection requirements are based on wind loading on end zones as defined in Figure 6-2 of ASCE 7. Connection loads for connections located a distance of 20% of the least horizontal dimension of the building from the corner of the building are permitted to be reduced by multiplying the table connection value by 0.7 and multiplying the overhang load by 0.8.
f. For wall-to-wall and wall-to-foundation connections, the capacity of the uplift connector is permitted to be reduced by 100 pounds for each full wall above. (For example, if a 600-pound rated connector is used on the roof framing, a 500-pound rated connector is permitted at the next floor level down).

TABLE AH107.4(1)
DESIGN WIND PRESSURES FOR ALUMINUM SCREEN ENCLOSURE FRAMING WITH AN IMPORTANCE FACTOR OF 0.77[a, b, c]

(Portions of table not shown remain unchanged)

For SI: 1 mile per hour = 0.44 m/s, 1 pound per square foot = 0.0479kPa, 1 foot = 304.8 mm.

a. Values have been reduced for 0.77 Importance Factor in accordance with Table 1604.5 of the *International Building Code*.
b. Minimum design pressure shall be 10 psf in accordance with Section 1609.1.2 of the *International Building Code*.
c. Loads are applicable to screen enclosures with a ~~mean roof~~ building height of 30 feet or less. For screen enclosures of different heights the pressures given shall be adjusted by multiplying the table pressure by the adjustment factor given in Table AH107.4(2).
d. For Load Case A flow thru condition the pressure given shall be applied simultaneously to both the upwind and downwind screen walls acting in the same direction as the wind. The structure shall also be analyzed for wind coming from the opposite direction. For the non-flow thru condition the screen enclosure wall shall be analyzed for the load applied acting toward the interior of the enclosure.
e. For Load Case B the table pressure multiplied by the projected frontal area of the screen enclosure is the total drag force, including drag on screen surfaces parallel to the wind, which must be transmitted to the ground. Use Load Case A for members directly supporting the screen surface perpendicular to the wind. Load Case B loads shall be applied only to structural members which carry wind loads from more than one surface.
f. The roof structure shall be analyzed for the pressure given occurring both upward and downward.

Reason: This proposal was prepared in conjunction with a related proposal on mean roof height and is intended as an alternative to that proposal. "Mean roof height" in that proposal, and in Section 6.2 of ASCE 7-05 from which it was extracted, is measured "to the average of the roof eave height and the height of the highest point on the roof surface" (exception for roof slopes no greater than 10°). Buildings, however, frequently have roofs of multiple eave and ridge heights. A definition limited to a single eave or ridge height fails to give the code user the information necessary to determine how to measure to multiple eave or ridge heights. This proposal resolves the problem by proposing the current definition of "building height" in the IBC and IRC, which is the "vertical distance from grade plane to the average height of the highest roof surface" (refer to IBC Section 502.1 and IRC Section R202). Note that the charging language for the defined terms in IBC Section 502.1 states that they "shall, for the purposes of this chapter and as used elsewhere in this code, have the meanings shown herein."

This proposal does not propose to add the exception for roof slopes less than 10 degrees in the definition of "mean roof height" in ASCE 7-05 to the definition of "building height" in the IBC or IRC. Mean roof height is utilized in ASCE 7-05 for the purpose of specifying structural design provisions. In the IBC, however, mean roof height is limited to a small number of prescriptive provisions related to roof covering systems and their substrates (i.e., roof sheathing). The term is used more extensively in the IRC but the effect is essentially the same. A separate defined term for these cases is not judged to be warranted. Relying on the currently defined term of "building height" will simply these provisions.

"Grade plane" was chosen over "grade" in the related proposal on mean roof height noted above because of approved Proposal G44-04/05-AM, which successfully established the distinction between "grade plane" as a measurement of the height and number of stories of a building above the finished ground surface and "grade" as a measurement of the height of a component of the building above the finished ground surface. Grade plane is an imaginary horizontal reference plane representing the weighted average of the finished ground surface adjoining the building at its perimeter. The grade plane of each building is located at a single, unique elevation. Grade, however, is not imaginary but is the actual finished ground surface adjoining the building at its perimeter, which varies in elevation with the ground surface.

With respect to this proposal, wherever "mean roof height" is specified in the IBC, the application is to a building or structure, not to a component of a building or structure. The situation in the IRC is murkier but a careful review by the proponent concluded that the intent is application to a building or structure in virtually all cases. In ASCE 7-05, "mean roof height" is used interchangeably as a measurement for buildings and other structures, and their components. In Section 6.5.12.4.1 on components and cladding, for example, please refer to the definition of velocity pressure (q_v) evaluated at mean roof height (h).

Footnote (a) of IBC Table 1504.8 is deleted in coordination with the proposed definition. The reference to ASCE 7-98 in Footnote (a) of IRC Tables R611.3(1) and R611.7.4 is also corrected.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

S83–07/08
1609.1.2.1 (New), Chapter 35 (New)

Proponent: John Woestman, The Kellen Company, representing Door Safety Council

1. Add new text as follows:

1609.1.2.1 Side-hinged doors. Side-hinged door assemblies shall be permitted to meet the impact testing requirements of ANSI/SDI A250.13.

2. Add standard to Chapter 35 as follows:

ANSI

ANSI/SDI A250.13-XX Testing and Rating of Severe Windstorm Resistant Components for Swinging Door Assemblies

Reason: This proposed change allows an alternative method to demonstrate performance to impact-resistant requirements for side-hinged doors by requiring doors to be tested per ANSI/SDI A250.13-XX. A250.13-XX, which is under development to update A250.13-03, will contain language that prescribes how door components are to be selected to create door assemblies expected to perform equivalently to a door assembly tested to ASTM E 1996 / E 1886 for impact resistance.

This proposal helps resolve performance and code compliance issues when doors are assembled from components from multiple sources and include interchangeable elements.

Through the ASTM standards development process, members of the Steel Door Institute (SDI) and members of the Builders Hardware Manufacturers' Association (BHMA) developed a national standard for a component-based approach to testing for windstorm resistance of swinging door assemblies. The test procedures used in this standard represent the most severe requirements found in the windstorm resistance standards in use in building codes. However, the procedures are designed to isolate, as much as possible, the loads and conditions that a particular component is subjected to in the full assembly test and duplicate these specific conditions. Using a combination of worst-case assembly design and safety factors, this testing was designed to provide a component rating that related directly to the component's ability to withstand the conditions that occur in a full assembly test.

Prior to releasing the current ANSI/SDI A250.13 standard, the BHMA/SDI task group conducted validation testing where components were expected to be rated at three design-load target values. Those components were tested to establish their ratings by the proposed procedure. Following this process, complete assemblies were tested in accordance with the ASTM E1886 test method. The results of this process confirmed that assemblies made up of rated components would perform as expected. In addition, the validation test showed that where a component was identified as the weakest element of an assembly, based on the component tests, the same component would fail in a similar manner when tested as part of an assembly to levels exceeding the component's rated capacity.

Building designers will use performance criteria of door components, per ANSI/SDI A250.13, to select appropriate components to create door assemblies by conducting an opening-by-opening design analysis, specify components, verify code compliance, and submit the results through the normal plans review process. Code Authorities will thus need only to verify that the design load and compliance analysis has been correctly carried out and that the specified components are actually installed during construction in accordance with the manufacturer's instructions and project specifications.

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

Analysis: A review of the standard(s) proposed for inclusion in the code, ANSI/SDI A250.13, for compliance with ICC criteria for referenced standards given in Section 3.6 of Council Policy #CP 28 will be posted on the ICC website on or before January 15, 2008.

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

S84–07/08
1609.1.1, 1609.6 (New)

Proponent: Edwin Huston, National Council of Structural Engineers Associations (NCSEA), representing NCSEA Code Advisory Committee – General Engineering Subcommittee

1. Revise as follows:

1609.1.1 (Supp) Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapter 6 of ASCE 7 or provisions of the Alternate All-heights Method in Section 1609.6. The type of opening protection required, the basic wind speed and the exposure category for a site is permitted to be determined in accordance with Section 1609 or ASCE 7.Wind shall be assumed to come from any horizontal direction and wind pressures shall be assumed to act normal to the surface considered.

Exceptions:

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

2. Add new text as follows:

1609.6 Alternate All-Heights method. The alternate wind design provisions in this section are simplifications of the ASCE 7 Method 2-Analytical Procedure.

1609.6.1 Scope. As an alternate to ASCE 7 Section 6.5, the following provisions are permitted to be used to determine the wind effects on regularly shaped buildings, or other structures which meet all of the following conditions:

1. The building or other structure is less than 100 feet (30480 mm) in height, with a height to least width ratio of 4 or less.
2. The building or other structure is not sensitive to dynamic effects.
3. The building or other structure is not located on a site for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.

1609.6.1.1 Modifications. The following modifications shall be made to certain subsections in ASCE 7: Section 1609.6.3 Symbols and Notations that are specific to this section are used in conjunction with the Symbols and Notations in ASCE 7 Section.6.3.

1609.6.2 Symbols and notations. Coefficients and variables used in the Alternate All-Heights Method equations are as follows:

C_{net} = net-pressure coefficient based on K_d [(G) (C_p) – (GC_{pi})], Ref Table 1609.6.2(2)

G = Gust effect factor equal to 0.85 for rigid structures per ASCE 7 Section 6.5.8.1.
K_d = Wind directionality factor per ASCE 7 Table 6-4.
P_{net} = Design wind pressure to be used in determination of wind loads on buildings or other structures or their components and cladding, in lb/ft^2 (N/m^2).
q_s = Wind velocity pressure in lb/ft^2 (N/m^2). (Per Table 1609.6.2(1))

1609.6.3 Design equations. When using the Alternate All-Heights Method, the Main-Wind-Force-Resisting System, (MWFRS) and Components and Cladding of every structure shall be designed to resist the effects of wind pressures on the building envelope in accordance with Equation (16-36).

$$P_{net} = q_s K_z C_{net} [I K_{zt}] \qquad \text{(Equation 16-36)}$$

Design wind forces for the MWFRS shall not be less than 10 lb/ft^2 (0.48 KN/$_m{}^2$) multiplied by the area of the structure projected on a plane normal to the assumed wind direction. See ASCE Section 6.1.4 for criteria. Design net wind pressure for components and cladding shall not be less than 10 lb/ft^2 (0.48 KN/$_m{}^2$) acting in either direction normal to the surface.

1609.6.4 Design procedure. The MWFRS and the components and cladding of every building or other structure shall be designed for the pressures calculated using Equation (16-36).

1609.6.4.1 Main Wind-Force-Resisting Systems. The MWFRS shall be investigated for the torsional effects identified in ASCE 7 Figure 6-9.

1609.6.4.2 Determination of K_z and K_{zt}. Velocity Pressure Exposure Coefficient, K_z, shall be determined in accordance with ASCE 7 Section 6.5.6.6 and the Topographic Factor, K_{zt} shall be determined in accordance with ASCE 7 Section 6.5.7.

1. For the windward side of a structure, K_{zt} and K_z shall be based on height z.
2. For leeward and side walls, and for windward and leeward roofs, K_{zt} and K_z shall be based on mean roof height *h*.

1609.6.4.3 Determination of net pressure coefficients, C_{net}. For the design of the Main Wind-Force-Resisting-System and for Components and Cladding, the sum of the internal and external net pressure shall be based on the net pressure coefficient C_{net}.

1. The pressure coefficient, C_{net}, for walls and roofs shall be determined from Table 1609.6.2(2).
2. Where C_{net} may have more than one value, the more severe wind load combination shall be used for design.

1609.6.4.4 Application of wind pressures. When using the Alternate All-Heights Method, wind pressures shall be applied simultaneously on, and in a direction normal to, all building envelope wall and roof surfaces.

1609.6.4.4.1 Components and cladding. Wind pressure for each component or cladding element is applied as follows using C_{net} values based on the effective wind area, *A* contained within the zones in areas-of-discontinuity of width and/or length "a", "2a" or "4a" at: corners of roofs and walls; edge strips for ridges, rakes and eaves; or field areas on walls or roofs as indicated in Figures in Table 1609.6.2(2) in accordance with the following:

1. Calculated pressures at local discontinuities acting over specific edge strips or corner boundary areas.
2. Include "field" (zone 1, 2 or 4, as applicable) pressures applied to areas beyond the boundaries of the areas-of-discontinuity.
3. Where applicable, the calculated pressures at discontinuities (zones 2 or 3) shall be combined with design pressures that apply specifically on rakes or eave overhangs.

TABLE 1609.6.2(1)
WIND VELOCITY PRESSURE (q_s) AT STANDARD HEIGHT OF 33 FEET [a, b, c]

BASIC WIND SPEED, V (mph)	85	90	100	105	110	120	125	130	140	150	160	170
PRESSURE, q_s (psf)	18.5	20.7	25.6	28.2	31.0	36.9	40.0	43.3	50.2	57.6	65.5	74.0

a. For Wind Speeds not shown, use $q_s = 0.00256\ V^2$
b. Multiply by 1.61 to convert to km/h
c. Multiply by 0.048 to convert to kN/m^2

TABLE 1609.6.2(2)
NET PRESSURE COEFFICIENTS, C_{net}[a, b, c]

STRUCTURE OR PART THEREOF	DESCRIPTION		C_{net} FACTOR		
1. Main Wind Force Resisting Frames and Systems	WALLS:		Enclosed	Part Enclosed	
	Windward Wall		0.43	0.11	
	Leeward Wall		-0.53	-0.83	
	Side Wall		-0.66	-0.97	
	Parapet Wall	Windward	1.28	1.28	
		Leeward	-0.85	-0.85	
	ROOFS:		Enclosed	Part Enclosed	
	Wind perpendicular to ridge				
	Leeward roof or flat roof		-0.66	-0.97	
	Windward roof slopes:				
	Slope < 2:12 (10°)		-1.09	-1.41	
	Slope = 4:12 (18°)		-0.73	-1.04	
	Slope = 5:12 (23°)		-0.58	-0.90	
	Slope = 6:12 (27°)	Case 1	-0.47	-0.78	
		Case 2	0.20	0.51	
	Slope = 7:12 (30°)	Case 1	-0.37	-0.68	
		Case 2	0.30	0.61	
	Slope 9:12 (37°)	Case 1	-0.27	-0.58	
		Case 2	0.31	0.63	
	Slope 12:12 (45°)		0.37	0.68	
	Wind parallel to ridge and flat roofs		-1.09	-1.41	
	Non Building Structures: Chimneys, Tanks and Similar Structures:				
			h/D		
			1	7	25
	Square (Wind normal to face)		0.99	1.07	1.53
	Square (Wind on diagonal)		0.77	0.84	1.15
	Hexagonal or Octagonal		0.81	0.97	1.13
	Round		0.65	0.81	0.97
	Open Signs and Lattice Frameworks		Ratio of solid to gross area		
			< 0.1	0.1 to 0.29	0.3 to 0.7
	Flat		1.45	1.30	1.16
	Round		0.87	0.94	1.08
2. Components and Cladding not in areas of discontinuity – Roofs and overhangs	Roof Elements and slopes		Enclosed	Partially Enc.	
	Gable or Hipped Configurations (Zone 1)				
	Flat < Slope < 6:12 (27°)				
	Positive	10 SF or less	0.58	0.89	
		100 SF or more	0.41	0.72	
	Negative	10 SF or less	-1.00	-1.32	

		100 SF or more	-0.92	-1.23
	Overhang: Flat < Slope < 6:12 (27°)			
	Negative	10 SF or less	-1.45	
		100 SF or more	-1.36	
		500 SF or more	-0.94	
	6:12 (27°) < Slope < 12:12 (45°)			
	Positive	10 SF or less	0.92	1.23
		100 SF or more	0.83	1.15
	Negative	10 SF or less	-1.00	-1.32
		100 SF or more	-0.83	-1.15
	Monosloped Configurations (Zone 1)		Enclosed	Partially Enc.
	Flat < Slope < 7:12 (30°)			
	Positive	10 SF or less	0.49	0.81
		100 SF or more	0.41	0.72
	Negative	10 SF or less	-1.26	-1.57
		100 SF or more	-1.09	-1.40
	Tall flat topped roofs h> 60’		Enclosed	Partially Enc.
	Flat < slope < 2:12 (10°) (Zone 1)			
	Negative	10 SF or less	-1.34	-1.66
		500 SF or more	-1.00	-1.32
3. Components and Cladding in areas of discontinuities – Roofs and overhangs	Roof Elements and slopes		Enclosed	Partially Enc.
	Gable or Hipped Configurations at Ridges, Eaves and Rakes (Zone 2)			
	Flat < Slope < 6:12 (27°)			
	Positive	10 SF or less	0.58	0.89
		100 SF or more	0.41	0.72
	Negative	10 SF or less	-1.68	-2.00
		100 SF or more	-1.17	-1.49

Overhang for Slope Flat < Slope < 6:12 (27°)			
Negative	10 SF or less	-1.87	
	100 SF or more	-1.87	
6:12 (27°) < Slope < 12:12 (45°)		Enclosed	Partially Enc.
Positive	10 SF or less	0.92	1.23
	100 SF or more	0.83	1.15
Negative	10 SF or less	-1.17	-1.49
	100 SF or more	-1.00	-1.32
Overhang for 6:12 (27°) < Slope < 12:12 (45°)			
Negative	10 SF or less	-1.70	
	100 SF or more	-1.53	
Monosloped Configurations at Ridges, Eaves and Rakes (Zone 2)			
Flat < Slope < 7:12 (30°)			
Positive	10 SF or less	0.49	0.81
	100 SF or more	0.41	0.72
Negative	10 SF or less	-1.51	-1.83
	100 SF or more	-1.43	-1.74
Tall flat topped roofs h> 60'		Enclosed	Partially Enc.
Flat <slope < 2:12 (10°) (Zone 2)			
Negative	10 SF or less	-2.11	-2.42
	500 SF or more	-1.51	-1.83
Gable or Hipped Configurations at Corners (Zone 3)			
Flat < Slope < 6:12 (27°)		Enclosed	Partially Enc.
Positive	10 SF or less	0.58	0.89
	100 SF or more	0.41	0.72
Negative	10 SF or less	-2.53	-2.85
	100 SF or more	-1.85	-2.17
Overhang for Slope Flat < Slope < 6:12 (27°)			
Negative	10 SF or less	-3.15	

		100 SF or more	-2.13	
	6:12 (27°) < Slope < 12:12 (45°)			
	Positive	10 SF or less	0.92	1.23
		100 SF or more	0.83	1.15
	Negative	10 SF or less	-1.17	-1.49
		100 SF or more	-1.00	-1.32
	Overhang for 6:12 (27°) < Slope <		Enclosed	Partially Enc.
	Negative	10 SF or less	-1.70	
		100 SF or more	-1.53	
	Monosloped Configurations at corners (Zone 3)			
	Flat < Slope < 7:12 (30°)			
	Positive	10 SF or less	0.49	0.81
		100 SF or more	0.41	0.72
	Negative	10 SF or less	-2.62	-2.93
		100 SF or more	-1.85	-2.17
	Tall flat topped roofs h> 60’		Enclosed	Partially Enc.
	Flat <slope < 2:12 (10°) (Zone 3)			
	Negative	10 SF or less	-2.87	-3.19
		500 SF or more	-2.11	-2.42
4. Components and Cladding not in areas of discontinuity - Walls and parapets	Wall Elements: h ≤ 60' (Zone 4)		Enclosed	Partially Enc.
	Positive	10 SF or less	1.00	1.32
		500 SF or more	0.75	1.06
	Negative	10 SF or less	-1.09	-1.40
		500 SF or more	-0.83	-1.15
	Wall Elements: h > 60' (Zone 4)			
	Positive	20 SF or less	0.92	1.23
		500 SF or more	0.66	0.98
	Negative	20 SF or less	-0.92	-1.23
		500 SF or more	-0.75	-1.06

	Parapet Walls			
	Positive		2.87	3.19
	Negative		-1.68	-2.00
5. Components and Cladding in areas of discontinuity - Walls and parapets	Wall Elements: h ≤ 60' (Zone 5)		Enclosed	Partially Enc.
	Positive	10 SF or less	1.00	1.32
		500 SF or more	0.75	1.06
	Negative	10 SF or less	-1.34	-1.66
		500 SF or more	-0.83	-1.05
	Wall Elements: h > 60' (Zone 5)			
	Positive	20 SF or less	0.92	1.23
		500 SF or more	0.66	0.98
	Negative	20 SF or less	-1.68	-2.00
		500 SF or more	-1.00	-1.32
	Parapet Walls			
	Positive		3.64	3.95
	Negative		-2.45	-2.76

a. Linear interpolation between values in the table is acceptable.
b. For open buildings, multispan gable roofs, stepped roofs, sawtooth roofs, domed roofs, solid free standing walls and solid signs apply ASCE 7.
c. Some C_{net} values have been grouped together. Less conservative results may be obtained by applying ASCE 7.

Reason: The all heights wind provisions of ASCE 7 are time consuming and confusing. Many engineers make significant errors in their use of this method. There is a simplified method in ASCE 7, but it is limited in use. Member Organizations of NCSEA have brought forward an alternate method which is in full compliance with ASCE 7. This method is being considered by the ASCE 7 Wind Committee, but it won't be able to be placed in the standard until the 2012 IBC is adopted. To speed this transition, this method is proposed for the IBC first.

The derivation of this method from ASCE 7 Chapter 6 is as follows:
C_{net} values
$q_z = 0.00256\, K_z\, K_{zt}\, K_d\, V^2\, I$ Eqn 6-15
$p = q\, G\, C_p - q_i\, (GC_{pi})$ Eqn 6-17

$p = 0.00256\, K_h\, K_{zt}\, K_d\, V^2\, I\, G\, C_p - 0.00256\, K_z\, K_{zt}\, K_d\, V^2\, I\, (GC_{pi})$

Rearranging terms:
$p = (\, 0.00256\, V^2\, K_h\, K_d\, G\, C_p - 0.00256\, V^2\, K_z\, K_d\, (GC_{pi}))\, K_{zt}\, I$

Define: $q_z = 0.00256\, V^2$
so: $p = (q_s\, K_h\, K_d\, G\, C_p - q_s\, K_z\, K_d\, (GC_{pi}))\, K_{zt}\, I$
and: $p = q_s\, K_d\, (\, K_h\, G\, C_p - K_z\, (GC_{pi}))\, K_{zt}\, I$

For leeward wall and roof elements
$K_h = K_z$
so: $p = q_s\, K_z\, (\, K_d\, (G\, C_p - (GC_{pi})))\, K_{zt}\, I$
Substitute $C_{net} = K_d\, (G\, C_p - (GC_{pi}))$
and we get: $p = q_s\, K_z\, C_{net}\, K_{zt}\, I$

which is Eqn. 16-xx in the draft.
For windward roof elements
$K_h \approx K_z$ and the same relationship holds.

For buildings: K_d = 0.85
For rigid structures: G = 0.85
so: C_{net} = 0.85 (0.85 C_p − (GC_{pi}))

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

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

S85–07/08

1609.1.1, 1609.6 (New)

Proponent: James S. Lai, S.E., representing Structural Engineers Association of California

1. Revise as follows:

1609.1.1 (Supp) Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapter 6 of ASCE 7 or provisions under the Alternate Wind Design Procedure in Section 1609.6. The type of opening protection required, the basic wind speed and the exposure category for a site is permitted to be determined in accordance with Section 1609 or ASCE 7.Wind shall be assumed to come from any horizontal direction and wind pressures shall be assumed to act normal to the surface considered.

Exceptions:

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

2. Add new text as follows:

1609.6 Alternate wind load procedure. The following wind load provisions are permitted as an alternative to Section 6.5 Method 2 – Analytical Procedure of ASCE 7.

1609.6.1 Scope: Buildings or other structures whose design wind forces are determined in accordance with Section 1609.6 shall meet the following requirements:

1. The building or other structure shall have no unusual geometric irregularity or spatial form.
2. The building or other structure does not have response characteristics making it subject to across wind loading, vortex shedding, instability due to galloping or flutter; and does not have a site location for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.
3. A building or other structure greater than 100 feet (30480mm) in height shall be limited to a height-to-least-width ratio of 4 or less, and with a fundamental natural frequency greater than or equal to one Hz.

1609.6.2 Modifications. The text of ASCE 7 shall be modified as indicated in Sections 1609.6.3 through 1609.6.6.

1609.6.3 Symbols and notations. Symbols and notations are specific to this section in conjunction with Symbols and notations in ASCE 7 Section 6.3.

B_{MWFRS} = Maximum horizontal distance between vertical elements of MWFRS resisting wind forces in any given direction.

C_{net} = Net-pressure coefficient based on K_d $[GC_p – (GC_{pi})]$, see Table 1609.6.3(2)

G = Gust effect factor equal to 0.85 for rigid buildings as defined in ASCE 7 Section 6.5.8.1

K_d = Wind directionality factor

P_{net} = Design wind pressure used to determine wind loads on buildings or other structures, or their components and cladding, in lb/ft^2 (N/m^2)

q_s = Wind velocity pressure in lb/ft^2 (N/m^2), Table 1609.6.3(1)

1609.6.4 Design wind pressures. When using the Alternate Wind Design Procedure, the Main-Force-Resisting System, (MWFRS) and Components and Cladding of every building or structure shall be designed to resist the effects of wind pressures on the building envelope. The net pressure on exterior building surfaces shall be determined as follows:

$P_{net} = q_s K_z C_{net} [I K_{zt}]$ **Equation (16-36)**

Design wind forces for the MWFRS shall not be less than 10 lb/ft^2 (0.48 kN/m^2) multiplied by the area of the building or structure projected on a plane normal to the wind direction under consideration. See ASCE 7 Section 6.1.4 for criteria. Design wind pressure for components and cladding shall not be less than 10 lb/ft^2 (0.48 kN/m^2) acting in either direction normal to the surface.

1609.6.5 Design procedure. The MWFRS of every building or other structure shall be designed for the combination of the windward and leeward net pressure, P_{net}, using equation (16-36). Components and claddings of every building or structure shall be designed for the critical net pressure, P_{net}, using Equation (16-36).

1609.6.5.1 Main wind force resisting systems. Where the ratio B_{MWFRS}/B is less than 0.7, the MWFRS shall be investigated for the torsional effects identified in ASCE 7 Figure 6-9.

1609.6.5.2 Determination of K_z and K_{zt}. Velocity pressure exposure coefficient, K_z, shall be determined in accordance with ASCE 7 Section 6.5.6.6; and Topography Factor, K_{zt}, shall be determined in accordance with ASCE7 Section 6.5.7.

1. For windward side of a structure, K_z and K_{zt} shall be based on height z.
2. For leeward side and side walls, and for windward and leeward roofs, K_z and K_{zt} shall be based on mean roof height *h*.

1609.6.5.3 Determination of net pressure coefficient C_{net}. For the design of the main wind force resisting system and for components and cladding, the net pressure shall be as follows:

1. The net pressure coefficient, C_{net} for walls and roofs shall be determined from Table 1609.6.3(2).
2. Where C_{net} may have more than one value, the more severe wind load combination shall be used for design.

1609.6.6 Application of wind pressures. When using Alternate Wind Design Procedure, wind pressure shall be applied simultaneously on, and in a direction normal to, all building envelope wall and roof surfaces.

1609.6.6.1 Components and cladding. Wind pressure for each component or cladding element is applied using C_{net} values based on the effective wind area, *A*, contained within the zones in areas of discontinuity of width and/ or length "a", "2a" or 4a" at: corners of roofs and walls; edge strips for ridges, rakes and eaves; or field areas on walls or roofs as indicated in Table 1609.6.3(2), and shall meet the following:

1. Calculated pressure at local discontinuities acting over specific edge strips or corner boundary areas.
2. Include "field" (zones 1, 2 or 4 as applicable) pressures applied to areas beyond the boundaries of the areas of discontinuity.
3. Where applicable, calculated pressures at discontinuities (zones 2 or 3) shall be combined with design pressures on rake or eave overhangs.

TABLE 1609.6.3(1)
WIND VELOCITY PRESSURE (q_s) AT STANDARD HEIGHT OF 33 FEET[a]

Basic Wind Speed, V (mph)[b]	85	90	95	100	110	120	130	140	150
Pressure, q_s (lb/ft^2) [c]	18.5	20.7	23.1	25.6	31.0	36.9	43.3	50.2	57.6

[a] For wind speeds not shown, use $q_s = 0.00256 V^2$
[b] Multiply by 1.61 to convert to km/h
[c] Multiply by 0.0478 to convert to kN/m^2

TABLE 1609.6.3(2)
NET PRESSURE COEFFICIENT, C_{net}

STRUCTURE OR PART THEREOF	DESCRIPTION	C_{NET} FACTOR [a]	
		Enclosed	**Partially enclosed**
1. Main Wind Force Resisting System	Walls:		
	Windward wall	0.43	0.11
	Leeward wall	-0.53	-0.83
	Side wall	-0.66	-0.97
	Parapet wall Windward	1.28	1.28
	Leeward	-0.85	-0.85
	Roofs:		
	Wind perpendicular to ridge		
	Leeward roof or flat roof	-0.66	-0.97
	Windward roof slopes:		
	Slope ≤ 2:12 (or 10º)	-1.09	-1.41
	Slope 4:12 (or 18º)	-0.73	-1.05
	Slope 5:12 (or 22º)	-0.59	-0.90
	Slope 6:12 (or 27º) Case 1	-0.47	-0,79
	Case 2	0.20	0.51
	Slope 7:12 (or 30º) Case 1	-0.37	-0.68
	Case 2	0.30	0.61
	Slope 9:12 (or 37º) Case 1	-0.27	-0.58
	Case 2	0.33	0.64
	Slope 12:12 (or 45º)	0.37	0.68
	Slope > 12:12 (or 45º)	c	c
	Wind parallel to ridge or flat roofs	-1.09	-1.41

STRUCTURE OR PART THEREOF	DESCRIPTION		4	4	5	5
2. Components and cladding - Walls [b]	**Affected zone [d]**		**4**	**4**	**5**	**5**
	Wall elements h ≤ 60 ft.	≤ 10 sf	1.00	-1.09	1.00	-1.34
		≥ 500 sf	0.75	-0.83	0.75	-0.83
	Wall elements h > 60 ft.	≤ 20 sf	0.92	-0.92	0.92	-1.68
		≥ 500 sf	0.66	-0.75	0.66	-1.00
	Parapet walls h ≤ 60 ft		2.53	-1.94	3.38	-2.19
	Parapet walls h > 60 ft		2.87	-1.68	3.64	-2.45

STRUCTURE OR PART THEREOF	DESCRIPTION		1	1	2	2	3	3
3. Components and cladding - Roofs [b]	**Affected zone [d]**		**1**	**1**	**2**	**2**	**3**	**3**
	Roof for h > 60 ft [e]							
	Slope ≤ 2: 12 (or 10º)	≤ 10 sf	-	-1.34	-	-2.11	-	-2.87
		≥ 500 sf	-	-1.00	-	-1.51	-	-2.11
	Gable and Hipped Roof h ≤ 60 ft							
	Slope ≤ 6:12 (or 27º)	≤ 10 sf	0.58	-1.00	0.58	-1.68	0.58	-2.53
		≥ 100 sf	0.41	-0.92	0.41	-1.17	0.41	-1.85
	Overhang	≤ 10 sf	-	-1.45	-	-1.87	-	-3.15
		≥ 100 sf	-	-1.36	-	-1.87	-	-2.13
	Slope 6:12 to 12:12	≤ 10 sf	0.92	-1.00	0.92	-1.17	0.92	-1.17
	(or 27º to 45º)	≥ 100 sf	0.83	-0.83	0.83	-1.00	0.83	-1.00
	Overhang	≤ 10 sf	-	-	-	-1.70	-	-1.70
		≥ 100 sf	-	-	-	-1.53	-	-1.53
	Monoslope Roof h ≤ 60 ft							
	Slope ≤ 7:12 (or 30º)	≤ 10 sf	0.49	-1.26	0.49	-1.51	0.49	-2.62
		≥ 100 sf	0.41	-1.09	0.41	-1.43	0.41	-1.85

STRUCTURE OR PART THEREOF	DESCRIPTION	1	7	25
4. Chimneys, tanks and solid towers [b]	**Height / depth or diameter (h/D)**	**1**	**7**	**25**
	Square (wind normal to face)	0.99	1.07	1.53
	Square (wind along diagonal)	0.77	0.84	1.15
	Hexagonal or Octagonal	0.81	0.97	1.13

	Round	0.65	0.81	0.97
5. Open sign and lattice frameworks	**Ratio of solid to gross area**	**< 0.1**	**0.1 to 0.29**	**0.3 to 0.7**
	Flat Round	1.45 0.87	1.30 0.94	1.16 1.08
6. Freestanding Wall	**Horizontal to vertical dimension (B/s)**	**< 1**	**≥ 1**	
	On ground At roof top	1.30 1.41	1.05 1.30	

a. Linear interpolation between tabulated C_{net} values and between tabulated slope or effective wind areas is acceptable.
b. For components and claddings other than overhangs in partially enclosed buildings, algebraically add or subtract 0.32 to increase values on table.
c. Use wall element values for slopes greater than 12:12 (45º),
d. Refer to ASCE 7 Figure 11-A through Figure 17 for affected zone designations.
e. For roof slope > 2:12 (or 10º), use coefficients tabulated for gable and hipped roof h ≤ 60 ft.

Reason: In response to concerns from design engineers on the complexity of wind design procedures, this proposal provides for an alternate design procedure to Method 2 of ASCE 7.

In using 2006 IBC and ASCE 7, engineers have found that except for low rise light framed buildings, lateral force design of most structures tend to be controlled by seismic forces in the western states. While ASCE 7 includes a simplified procedure under Method 1 for buildings not greater than sixty feet in height, the procedure includes various limitations such as simple diaphragm, low rise building with no unusual geometrical irregularity, and requires an engineer to refer to numerous relatively complicated charts. The complexity of the detailed Analytical Procedure has daunted even the most experience engineers. The need for wind design procedure in the IBC similar to that which was in the 1997 UBC has been echoed throughout most of the United States. [Reference 3 and 4.]

The Structural Engineers Association of California established a Wind *Ad Hoc* Committee in late 2006. The group was charged to develop alternate wind design procedures for all height buildings in conjunction with the Tri-state (California, Oregon and Washington) Wind Committee. The Tri-state Wind Committee with representatives appointed by each of the three states' structural engineers association, all of whom are experience structural engineers, was active in code development for the 1991 UBC using ASCE 7-88 standard as the source document, and also took a primary roll in developing the basic format of the wind design provisions in the 1997 Uniform Building Code, which is still being used in several states.

This proposed alternate design procedure is developed for the most common type of buildings that are not subjected to dynamic response with further limitation for building or other structure over 100 feet. The alternate method follows closely with design requirements Chapter 6 of ASCE 7. Simplification is accomplished by generating a table of net pressure coefficients (C_{net}), combining a number of parameters in a simple and yet conservative manner. Application of the net pressure coefficients meets the intent to reduce the number of steps required for performing a wind loading analysis on buildings that satisfy the criteria prescribed under the scope statements resulting in net forces which meet or exceed those calculated based on Method 2. The reduction of design effort should be helpful in the determination of wind forces for the main wind force resisting system; and should be substantial for components and cladding. The procedure has been designed to give results equal to or more conservative than the present provisions in ASCE 7.

While the proposed code change by SEAOC Wind *Ad Hoc* Committee has been developed in concert with the Tri-state Wind Committee proposed document, this proposal has some uniqueness in addressing buildings of all height and the table developed for C_{net} coefficient has been arranged in a similar format as the 1997 Uniform Building Code, which most engineers preferred in the past. Given the substantial time savings using this proposed alternate design procedure, and given that the next edition of ASCE wind standard will not be published until after 2010, we respectfully request that this proposed change be adopted into the IBC as an alternative procedure until such time as the next edition of ASCE wind standard can incorporate this alternate design method.

Bibliography:

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4. Ghosh. S.K., January 2007, "The evolution of wind provisions in standards and codes in the United States – Part 2," *Structural Engineer*, Zweig White Information Services, Skokie, IL.
5. Lai, J. and Luttrell, K., 2007, "*Who Cares About Wind,*" Structural Engineers Convention Proceedings, Sacramento, CA.
6. ICBO, "*1997 Uniform Building Code, Volume 2, Structural Engineering Provisions*, Division III, Wind Design," International Conference of Building Officials, 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

S86–07/08
1612.2

Proponent: William Easterling, Grand Haven, MI, representing himself

Add new definition as follows:

1612.2 Definitions. The following words and terms shall, for the purposes of this section, have the meanings shown herein.

NEW CONSTRUCTION. Permanent construction to any structure, including any addition, alteration, repair or subsequent improvement to an existing structure.

Reason: The code change proposal clarifies the "portions of building and structures" requirement of Section 1612.1 and makes it clearer as in the *International Residential Code,* Section R102.7.1, by requiring all new construction installed below the design flood elevation be the same as what is required for a new building. Like with other hazards (snow, wind, etc…) that become known of or better understood after a structure is built, any subsequent repairs and alterations to an existing structure should be afforded the same minimum protections as established for new structures.

Such enforcement will incrementally provide, at least to the repair or alteration undertaken, the already established minimum protection for new structures from the known hazard of floods. Additionally consistent enforcement of flood-resistant material requirements on repairs and alterations will help reduce repetitive losses and assist in keeping future repair costs from reaching the substantial damage threshold.

The proposed code clarification is also inline with 44CFR60.3.3 – Floodplain Management Criteria for Flood-Prone Areas; which requires a local jurisdiction participating in FEMA's National Flood Insurance Program to "Review all permit applications to determine whether proposed building sites will be reasonably safe from flooding. If a proposed building site is in a flood-prone area, all new construction and substantial improvements shall (i) be designed (or modified) and adequately anchored to prevent flotation, collapse, or lateral movement of the structure resulting from hydrodynamic and hydrostatic loads, including the effects of buoyancy, (ii) be constructed with materials resistant to flood damage, (iii) be constructed by methods and practices that minimize flood damages, and (iv) be constructed with electrical, heating, ventilation, plumbing, and air conditioning equipment and other service facilities that are designed and/or located so as to prevent water from entering or accumulating within the components during conditions of flooding."

Likewise according to federal law, being 44CFR60.1.d, FEMA encourages jurisdictions to adopt more comprehensive floodplain management regulations such as what *International Residential Code* has already done with plain meaning of Section R102.7.1, Section R301.2.4, and Section R324. Federal law states in part at 44CFR60.1.d that: "Any community may exceed the minimum criteria under this part by adopting more comprehensive flood plain management regulations … Therefore, any flood plain management regulations adopted by a State or a community which are more restrictive than the criteria set forth in this part are encouraged and shall take precedence".

Cost Impact: The code change proposal will not increase cost of construction because it is already required. Even if it is not in the code already the incremental increase in the first cost of construction will be quickly recognized as a savings given the known fact of a flood hazard area and the design flood elevation.

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

S87–07/08
1612.2

Proponent: Cheryl Kent, U. S. Department of Housing and Urban Development, representing U. S. Department of Housing and Urban Development

Revise as follows:

1612.2 Definitions. The following words and terms shall, for the purposes of this section, have the meanings shown herein.

SUBSTANTIAL IMPROVEMENT. Any repair, reconstruction, rehabilitation, addition or improvement of a building or structure, the cost of which equals or exceeds 50 percent of the market value of the structure before the improvement or repair is started. If the structure has sustained substantial damage, any repairs are considered substantial improvement regardless of the actual repair work performed. The term does not, however, include either.

1. Any project for improvement of a building required to correct existing health, sanitary or safety code violations identified by the building official and that are the minimum necessary to assure safe living conditions.
2. Any alteration of a historic structure provided that the alteration will not preclude the structure's continued designation as a historic structure.
3. Any project for improvement of a building to provide accessibility for persons with disabilities.

Reason: While this is not a federal Fair Housing Act accessible design and construction issue, HUD is also a standard setting agency for Section 504 of the Rehabilitation Act and the Architectural Barriers Act.

Accessibility improvements are required under the Americans with Disabilities Act (ADA) even when alterations are not planned. Please see the Department of Justice regulations implementing title III of the ADA covering public accommodations and commercial facilities (28 CFR 36.304 http://www.usdoj.gov/crt/ada/reg3a.html) and title II of the ADA covering state and local governments (28 CFR 35.149 http://www.usdoj.gov/crt/ada/reg2.html). Also see the Department of Housing and Urban Development's regulations implementing Section 504 of the Rehabilitation Act of 1973, as amended which requires program access in existing residential facilities which receive federal financial assistance (24 CFR 8.24 http://www.access.gpo.gov/nara/cfr/waisidx_98/24cfr8_98.html). We believe that it is appropriate for the building code to encourage compliance with these federal laws and other similar state and local requirements. Additionally, the code should advance public welfare by encouraging building owners to improve the accessibility of their properties without triggering additional code requirements unrelated to accessibility.

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

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

S88–07/08

1612.3.2 (New)

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

Revise as follows:

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

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

<u>**1612.3.2 Determination of impacts.** In riverine flood hazard areas where design flood elevations are specified but floodways have not been designated, the applicant shall demonstrate that the cumulative effect of the proposed buildings and structures, when combined with all other existing and anticipated flood hazard area encroachments, will not increase the design flood elevation more than 1 foot (305 mm) at any point within the jurisdiction of the applicable governing authority.</u>

Reason: The purpose of this code change is to improve consistency with the requirements of the National Flood Insurance Program (NFIP) regarding development in flood hazard areas where base (or design) flood elevations are shown on the Flood Insurance Rate Map, but analyses to delineate the floodway were not performed. Development in riverine floodplains can increase flood levels and loads on other properties, especially if it occurs in areas known as floodways that must be reserved to convey flood flows. The floodway, as defined in 1612.2, is the area along riverine waterways that "must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation more than a designated height." For the situation addressed by this code change, the designed height is one foot.

Similar language appears in four locations in the I-Codes: (1) IBC 1803.4(4) to address proposed grading and filling; (2) IBC Appendix G103.4; (3) IBC Appendix J101.2; and IRC R324.1.3.2. The requirement to determine cumulative impacts has been part of the NFIP for more than 20 years and has been administered by more than 20,000 local jurisdictions that participate in the NFIP.

References:

Title 44 Code of Federal Regulations Parts 59 and 60, Regulations for Floodplain Management and Flood Hazard Identification." Online at http://www.fema.gov/business/nfip/laws1.shtm.

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

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

S89–07/08
1612.4, 1801.1

Proponent: Philip Brazil, PE, SE, Reid Middleton, Inc., representing himself

Revise as follows:

1612.4 Design and construction. The design and construction of buildings and structures located in flood hazard areas, including flood hazard areas subject to high velocity wave action, shall be in accordance with Chapter 5 of ASCE 7 and with ASCE 24.

1801.1 Scope. The provisions of this chapter shall apply to building and foundation systems ~~in those areas not subject to scour or water pressure by wind and wave action. Buildings and foundations subject to such scour or water pressure loads shall be designed in accordance with Chapter 16~~.

Reason: The purpose for this proposal is to align the provisions of the IBC, ASCE 7 and ASCE 24 with respect to the structural design of buildings and other structures in flood hazard areas. The current language implies that compliance with IBC Chapter 18 is required except for buildings and foundations subject to scour or water pressure by wind and wave action and the applicable requirements for them are specified in Chapter 16. The load combinations of IBC Section 1605 would not apply because there are none for flood loads. Sections 1605.2.2 and 1605.3.1.2 reference ASCE 7. Sections 2.3.3 and 2.4.2 of ASCE 7-05, in turn, specify load combinations that include flood loads. IBC Sections 1603.1.6 and 1612 specify requirements for determining flood hazard areas and documenting them on the construction documents. There are no other provisions in Chapter 16 specific to the structural design of buildings and other structures in flood hazard areas except for a reference to ASCE 24 in Section 1612.4. Chapter 5 of ASCE 7, however, contains comprehensive provisions for the determination of loads subjected to buildings and other structures located in "areas prone to flooding as defined on a flood hazard map."

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

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

S90–07/08
1613.6.6 (New)

Proponent: Jason J. Krohn, PE, Precast/Prestressed Concrete Institute representing himself

Add new text as follows:

1613.6.6 Additional seismic-force-resisting systems for reinforced concrete braced frames. Add the following lines to Table 12.2-1 of ASCE 7:

Seismic Force-Resisting System	Detailing Reference	R	Ω_0	C_d	B	C	D	E	F
A. BEARING WALL SYSTEMS									
16. Ordinary Reinforced Concrete Braced Frames	14.2	2.5	2	2.5	NL	NL	NP	NP	NP
B. BUILDING FRAME SYSTEMS									
28. Ordinary Reinforced Concrete Braced Frames	14.2	5	2	2.5	NL	NL	NP	NP	NP

Reason: The 1997 Edition of the Uniform Building Code (UBC 97) includes Table 16-N which defines the structural systems that may be used as lateral-force-resisting systems for earthquake design. That Table includes ordinary braced frames of reinforced concrete and assigns this system R-values and overstrength factors. Since the merger of the three model codes into the International Building Code, the definition of structural systems has been removed from the model code (IBC 2006) and is included only by reference to ASCE 7. ASCE 7-05 does not include the ordinary reinforced concrete braced frame system. Now that the IBC is being adopted practically everywhere in the United States, and the use of UBC 97 is being discontinued, it has been realized that this system, which has been in use, is no longer listed. The change is proposed as an additional exception to ASCE 7 so that the use of a previously defined system may continue.

Background to Change: a) UBC 97 includes ordinary concrete braced frames as a building frame system and as bearing wall system. b) As a bearing wall system, UBC 97 assigns the ordinary concrete braced frame system an R factor of 2.8 and an overstrength factor, Ω_o, of 2.2. As a building frame system, UBC 97 assigns the ordinary concrete braced frame system an R factor of 5.6 and an overstrength factor, Ω_o, of 2.2. This system is permitted in seismic zones 1 and 2. c) ACI 318 anticipates the use of concrete braced frames in its definition of "structural trusses" in Chapter 21, Special Provisions for Seismic Design. d) ACI 318 anticipates the use of concrete braced frames with specific provisions in Section

21.9.1: "This section also applies to … trusses serving as parts of the earthquake force-resisting systems." e) Reinforced concrete braced frames have been used in the past under the provisions of the UBC and ACI 318. These systems continue to be employed in many parts of the United States. f) Proposed system values are derived from UBC values, adjusted to the context of comparable systems in ASCE 7.

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

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

S91–07/08
1613.6.6 (New)

Proponent: Randall Shackelford, P.E., Simpson Strong-Tie Co. representing himself

Add new text as shown:

1613.6.6 Seismic design coefficients for horizontal combinations. The following section shall be permitted to be used instead of ASCE 7, Section 12.2.3.2.

Where a combination of different structural systems is utilized to resist lateral forces in the same direction, the values for R, C_d and Ω_0 determined by Table 1613.6.6 shall be used. The value of *R* used for the design of diaphragms in a particular direction shall not be greater than the least value for any of the systems utilized in that same direction.

TABLE 1613.6.6 R, C_d AND Ω_o VALUES FOR COMBINATION OF DIFFERENT STRUCTURAL SYSTEMS USED IN SAME DIRECTION

R value	The least value of *R* for any of the systems used. **Exception:** Resisting elements are permitted to be designed using the least value of *R* for the different structural systems found in each independent line of resistance if the following three conditions are met: 1) Occupancy Category I or II building, 2) two stories or less in height, and 3) use of light frame construction or flexible diaphragms.
C_d value	The C_d value corresponding to the system with the least value of R for any of the systems used. In the case where two or more systems have the same least value of R, the largest of the corresponding values of C_d shall be used.
Ω_o value	The Ω_o value corresponding to the system with the least value of R for any of the systems used. In the case where two or more systems have the same least value of R, the largest of the corresponding values of Ω_o shall be used.

Reason: The second paragraph of ASCE 7-05 Section 12.2.3.2 is far from clear. One possible interpretation is that when different structural systems are combined in the same direction of a building or other structure, the largest C_d- and Ω_0-values of all the individual structural systems shall be used. The other possible interpretation is that the C_d- and Ω_0-values shall correspond to the least R-value of all the individual structural systems. The second interpretation appears to be the more logical in view of the following example. (Because C_d is far more important than Ω_0, which has admissible values of only 2, 2.5 or 3, the following discussion is only in terms of C_d).

Consider an example where a special reinforced masonry shear wall (R = 5, C_d = 3.5) is combined with a special steel moment-resisting frame (R = 8, C_d = 5.5). There is no question that the R-value to be used in design is 5. The question is whether the C_d-value is 3.5 or 5.5. 5.5 does not seem logical – for two reasons. First, the combined system is much more rigid than the special steel moment frame itself. Second, large values of δ_{xe} would automatically result from the low value of R used in design. These, multiplied by the C_d-value of 5.5 would yield unrealistically large total displacements. C_d of 3.5 appears to be much more logical.

The above interpretation was implicit in the 1997 *Uniform Building Code*, where 0.7R was used in place of C_d.

The code change also offers clarification concerning another complication that may arise, which is that different structural systems having the same R-value sometimes have different C_d- and Ω_0-values.

This change should, in the normal course, have been (and will be) submitted to the Seismic Subcommittee of ASCE 7. However, the next edition of ASCE 7 will not be adopted by the IBC until its 2012 edition. It would be preferable from the point of view of the code user to incorporate the change in the 2009 IBC and then remove it from the 2012 IBC, once ASCE 7 has included the change.

The following is included in legislative format to show the modifications to ASCE 7

***12.2.3.2 R, C_d* and *Ω_0* Values for Horizontal Combinations.** Where a combination of different structural systems is utilized to resist lateral forces in the same direction, the values for R, C_d and Ω_0 determined by Table 1616.6.3 shall be used. ~~value of *R* used for design in that direction shall not be greater than the least value of *R* for any of the systems utilized in that direction. Resisting elements are permitted to be designed using the least value of *R* for the different structural systems found in each independent line of resistance if the following three conditions are met: 1) Occupancy Category I or II building, 2) two stories or less in height, and 3) use of light frame construction or flexible diaphragms.~~ The value of *R* used for the design of diaphragms in ~~such structures~~ a particular direction shall not be greater than the least value for any of the systems utilized in that same direction.

~~The deflection amplification factor, C_d, and the system overstrength factor, Ω_0, in the direction under consideration at any story shall not be less than the largest value of this factor for the R factor used in the same direction being considered~~

Table 1616.6.3 R, C_d and Ω_o Values for Combination of Different Structural Systems Used in Same Direction

R value	The least value of *R* for any of the systems used. **Exception:** Resisting elements are permitted to be designed using the least value of *R* for the different structural systems found in each independent line of resistance if the following three conditions are met: 1) Occupancy Category I or II building, 2) two stories or less in height, and 3) use of light frame construction or flexible diaphragms.
C_d value	The C_d value corresponding to the system with the least value of R for any of the systems used. In the case where two or more systems have the same least value of R, the largest of the corresponding values of C_d shall be used.
Ω_o value	The Ω_o value corresponding to the system with the least value of R for any of the systems used. In the case where two or more systems have the same least value of R, the largest of the corresponding values of Ω_o shall be used.

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

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

S92–07/08
1613.6.6 (New)

Proponent: James S. Lai, SE, representing Structural Engineers Association of California

Add new text as follows:

1613.6.6 Minimum distance for building separation. All buildings and structures shall be separated from adjoining structures. Separations shall allow for the maximum inelastic response displacement (δ_M). δ_M shall be determined at critical locations with consideration for both translational and torsional displacements of the structure using Equation 16-45.

$$\delta_M = C_d \delta_{max} \qquad \textbf{(Equation 16-45)}$$

Where

C_d = Deflection amplification factor in Table 12.2-1 of ASCE 7.

δ_{Max} = Maximum displacement defined in Section 12.8.4.3 of ASCE 7.

Adjacent buildings on the same property shall be separated by a distance not less than δ_{MT}, determined by Equation 16-46.

$$\delta_{MT} = \sqrt{(\delta_{M1})^2 + (\delta_{M2})^2} \qquad \textbf{(Equation 16-46)}$$

Where

δ_{M1}, δ_{m2} = The maximum inelastic response displacements of the adjacent buildings in accordance with Equation 16-45.

Where a structure adjoins a property line not common to a public way, the structure shall also be set back from the property line by not less than the maximum inelastic response displacement, δ_M, of that structure.

Exceptions:

1. Smaller separations or property line setbacks shall be permitted when justified by rational analyses.
2. Buildings and structures assigned to the Seismic Design Category A, B or C.

Reason: Purpose: To clarify requirements for separation distance between adjacent buildings.

Section 12.12.3 of ASCE 7-05 including Supplement No. 1 does not provide requirements for separation distances between adjacent buildings. Requirements for separation distances between adjacent buildings, not structurally connected, were included in IBC 2000 and 2003. However, when ASCE 7-05 was adopted by reference for IBC 2006, these requirements were omitted. In addition, ASCE 7-05 defines (δ_x) in Section 12.8.6 as the deflection of Level x at the center of mass. The actual displacement that needs to be used for building separation is the displacement at critical locations with consideration of both the translational and torsional displacements. These values can be significantly different.

This code change restores requirements for building separation in prior editions of IBC, and establishes minimum separation distance between adjoining buildings which are not structurally connected. The purpose of seismic separation is to permit adjoining buildings, or parts thereof, to respond to earthquake ground motion independently and thus preclude possible structural and non-structural damage caused by pounding between buildings or other structures.

References:

1. ICC, *2003 International Building Code*, “Section 1620.3.6, Building Separations; IBC 2003 Section 1620.4.5, Building Separations;” International Code Council, Country Club Hills, IL.
2. ICBO, *1997 Uniform Building Code, Volume 2, Structural Engineering Provisions*, “Section 1630.9.2, Determination of $\square_M$; Section 1630.10.1, General; and Section 1633.2.11, Building Separations,” International Conference of Building Officials, Whittier, CA.
3. SEAOC, *1999 Recommended Lateral Force Requirements and Commentary*, “Section C108.2.11, *Building Separations*,” Structural Engineers Association of California, Sacramento, CA.

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

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

S93–07/08
1613.6.6 (New)

Proponent: Jim W. Sealy, FAIA; Robert E. Bachman, SE; and John D. Gillengerten, Building Seismic Safety Council of the National Institute of Building Sciences, representing FEMA/BSSC Code Resource Support Committee

Add new text as follows:

1613.6.6 HVAC Ductwork with I_p = 1.5 Seismic supports are not required for HVAC ductwork with I_p = 1.5 if either of the following conditions are met for the full length of each duct run:

1. HVAC ducts are suspended from hangers 12 in. (305 mm) or less in length with hangers detailed to avoid significant bending of the hangers and their attachments or
2. HVAC ducts have a cross-sectional area of less than 6 ft^2 (0.557 m^2).

Reason: This proposal extends the exemptions from seismic bracing requirements to include small ducts where I_p=1.5. All ducts are generally braced or guyed to prevent lateral motion or swing. Given the low inertial loads associated with small ducts, this prescriptive bracing is sufficient for seismic loads as well.

The proposed change should result in reduced cost in the installation of HVAC ducting for buildings assigned to Seismic Design Category D, E, and F where HVAC system is assigned a component importance factor of 4.5 (such as hospitals). There will be no change in cost in California amendments to the 2006 IBC.

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

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

S94–07/08
1613.6.6 (New)

Proponent: Bonnie Manley, American Iron and Steel Institute, representing American Institute of Steel Construction

Add new text as follows:

1613.6.6 Steel plate shear wall height limits. Modify Section 12.2.5.4 of ASCE 7 as follows:

12.2.5.4 Increased Building Height Limit for Steel Braced Frames, Special Steel Plate Shear Walls and Special Reinforced Concrete Shear Walls. The height limits in Table 12.2-1 are permitted to be increased from 160 ft (50 m) to 240 ft (75 m) for structures assigned to Seismic Design Categories D or E and from 100 ft (30 m) to 160 ft (50 m) for structures assigned to Seismic Design Category F that have steel braced frames, special steel plate shear walls or special reinforced concrete cast-in-place shear walls and that meet both of the following requirements:

1. The structure shall not have an extreme torsional irregularity as defined in Table 12.2-1 (horizontal structural irregularity Type 1b).
2. The braced frames or shear walls in any one plane shall resist no more than 60 percent of the total seismic forces in each direction, neglecting accidental torsional effects.

Reason: Special Steel Plate Shear Wall (SPSW) systems were first introduced in the 2005 editions of ASCE 7 and AISC 341. During the incorporation of the system's seismic design parameters and height limitations into ASCE 7, Table 12.2-1, the inclusion of this system in the permitted height increase of ASCE 7, Section 12.2.5.4 was inadvertently overlooked. This minor modification to ASCE 7, Section 12.2.5.4 corrects that oversight.

Please note, we will be pursuing a correction to Section 12.2.5.4 for the 2010 edition of ASCE 7, so it is anticipated that this amendment to ASCE 7-05 will be necessary only for the 2009 edition of the IBC.

Additional Background: FEMA 450 (2003), *NEHRP Recommended Provisions For Seismic Regulations For New Buildings And Other Structures*, which is the source document for ASCE 7-05, states the following in Section 4.3.1.4 and its associated commentary:

Provisions:

4.3.1.4 Seismic Design Category D. The structural framing system for structures assigned to Seismic Design Category D shall comply with Sec. 4.3.1.3 and the additional requirements of this section.

4.3.1.4.1 Building height limits. The height limits in Table 4.3-1 are permitted to be increased to 240 ft (70 m) in buildings that have steel braced frames or concrete cast-in-place shear walls if such buildings are configured such that the braced frames or shear walls arranged in any one plane conform to the following:

1. The braced frames or cast-in-place special reinforced concrete shear walls in any one plane shall resist no more than 60 percent of the total seismic forces in each direction, neglecting torsional effects, and

2. The seismic force in any braced frame or shear wall resulting from torsional effects shall not exceed 20 percent of the total seismic force in that braced frame or shear wall.

Commentary:

4.3.1.4 Seismic Design Category D**.** Sec. 4.3.1.4 covers Seismic Design Category D, which compares roughly to California design practice for normal buildings away from major faults. In keeping with the philosophy of present codes for zones of high seismic risk, these requirements continue limitations on the use of certain types of structures over 160 ft (49 m) in height but with some changes. Although it is agreed that the lack of reliable data on the behavior of high-rise buildings whose structural systems involve shear walls and/or braced frames makes it convenient at present to establish some limits, the values of 160 ft (49 m) and 240 ft (73 m) introduced in these requirements are arbitrary. Considerable disagreement exists regarding the adequacy of these values, and it is intended that these limitations be the subject of further study.

According to these requirements require that buildings in Category D over 160 ft (49 m) in height must have one of the following seismic-force-resisting systems:

1. A moment resisting frame system with special moment frames capable of resisting the total prescribed seismic force...
2. A dual system as defined in this chapter, wherein the prescribed forces are resisted by the entire system and the special moment frame is designed to resist at least 25 percent of the prescribed seismic force...
3. <u>The use of a shear wall (or braced frame) system of cast-in-place concrete or structural steel up to a height of 240 ft (73 m) is permitted only if braced frames or shear walls in any plane do not resist more than 60 percent of the seismic design force including torsional effects and the configuration of the lateral-force-resisting system is such that torsional effects result in less than a 20 percent contribution to the strength demand on the walls or frames.</u> The intent is that each of these shear walls or braced frames be in a different plane and that the four or more planes required be spaced adequately throughout the plan or on the perimeter of the building in such a way that the premature failure of one of the single walls or frames will not lead to excessive inelastic torsion.

Although a structural system with lateral force resistance concentrated in the interior core (Figure C4.3-1 is acceptable according to the *Provisions*, it is highly recommended that use of such a system be avoided, particularly for taller buildings. The intent is to replace it by the system with lateral force resistance distributed across the entire building (Figure C4.3-2). The latter system is believed to be more suitable in view of the lack of reliable data regarding the behavior of tall buildings having structural systems based on central cores formed by coupled shear walls or slender braced frames.

Based upon the provision language and accompanying commentary, there seems to be no reason to not include special SPSW in the increased height limitations of ASCE 7, Section 12.2.5.4.

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

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

S95–07/08

1613.7 (New), 1613.7.1 (New), 1613.7.2 (New)

Proponent: Jim Messersmith, Jr., PE, Portland Cement Association

Add new text as follows:

<u>**1613.7 General**. The text of ASCE 7 shall be modified as indicated in Sections 1613.7.1 through 1613.7.2.</u>

<u>**1613.7.1 ASCE 7, Section 12.11.2**. Modify ASCE 7, Section 12.11.2 to read as follows:</u>

<u>**12.11.2 Anchorage of Concrete and Masonry Structural Walls.** The anchorage of concrete or masonry structural walls to supporting construction shall provide a direct connection capable of resisting the force set forth in Section 12.11.1.</u>

<u>Structural walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 ft (1,219 mm).</u>

<u>**1613.7.2 ASCE 7, Section 12.14.7.5.** Modify ASCE 7, Section 12.14.7.5 to read as follows:</u>

<u>**12.14.7.5 Anchorage of Concrete and Masonry Structural Walls**. Concrete or masonry structural walls … with diaphragms that are not flexible.</u>

Anchorage of structural walls to flexible diaphragms shall have the strength to develop the out-of-plane force given by Eq. 12.14.10:

$F_p = 0.8S_{DS}W_p$ (12.14-10)

Where
F_p = the design force in the individual anchor
S_{DS} = the design spectral response acceleration at short periods per Section 12.14.8.1
W_p = the weight of the wall tributary to the anchor

Exception: For Seismic Design Category B, the coefficient shall be 0.4, with a minimum force of 10 percent of the tributary weight of the wall.

Reason: Since ASCE 7-05 will be the loading standard referenced in the 2009 IBC, the only recourse to getting changes made to ASCE 7-05 is through modifications to the standard within the IBC.

The requirement that anchors attaching concrete and masonry walls to supporting construction be designed for a lower bound value of 280 plf or $400S_{DS}I$ plf, whichever is greater, is excessive and discriminatory considering that anchorage of walls of other materials, regardless of their mass, is required to be designed for a horizontal force of $0.40S_{DS}I$ or 10%, whichever is greater, times the weight of the wall tributary to the anchor.

To illustrate the punitive nature of the provision, consider two 10-foot high walls that are representative of walls used in single family dwellings and small commercial buildings; one a 5.5-inch thick concrete wall, the other a light-framed wall with 4-inch nominal masonry veneer anchored to the wall framing. For this example we'll assume that S_{DS} equals 0.32 (SDC B). The weight of the light-framed wall, including veneer is estimated to be 45 psf; therefore, the weight tributary to an anchor at the top of the wall is 225 plf (45 * (10/2) = 225). Therefore, the required design anchorage force for the light-framed wall is 29 plf (0.40 * 0.32 * 1 * 225 = 29). For a 5.5-inch concrete wall, which weighs approximately 69 psf, the weight tributary to an anchor at the top of the wall is 345 plf (69 * (10/2) = 345). Based on the requirement that applies to walls of other than concrete or masonry, the required anchorage design force for the concrete wall should be 44 plf (0.40 * 0.32 * 1 * 345 = 44); however, 280 plf must be used to design the anchorage.

Let's examine how the 280 plf requirement compares to wind design. ASCE 7, Section 6.1.4.2 requires that components and cladding be designed for a minimum service level design wind pressure of 10 psf. For our example walls using this minimum design wind pressure, the factored force at the top of the wall due to wind is 80 plf (1.6 * 10 * (10/2) = 80). Now let's determine what basic wind speed is required to produce a design force at the top of the wall of 280 plf (strength level). The service level (unfactored) force equal to 280 plf is 175 plf (280/1.6 = 175). Since 175 plf is based on a tributary wall height of 5 feet, the unfactored design wind pressure is 35 psf (175/5 = 35). From ASCE 7, Figure 6-3, for a building in exposure B, height of 30 feet, K_{zt} of 1.0, and effective wind area of less than or equal to 10 square feet, for wall area 4 the negative design pressure for a basic wind speed of 140 mph is 38.2 psf. Therefore, the requirement that the 5.5-inch, 10-foot high concrete wall be anchored against a force of 280 plf is the same as requiring that the connection be designed for a basic wind speed of approximately 135 mph in exposure B.

Next we'll examine the second criterion that the design anchorage force be not less than $400S_{DS}I$ plf. By setting the two criteria equal to each other, it can be determined that the 280 plf criterion controls at values of S_{DS} less than 0.70 (S_{DS} = 280/400 = 0.70) for I equal to 1.0. For values of S_{DS} greater than 0.70, $400S_{DS}I$ governs. If we're designing a connection between the concrete wall and a rigid diaphragm in a building where S_{DS} equals 1.0, the design force will be 400 plf; whereas, without the special criteria for concrete and masonry walls, the design force would be 138 plf (0.40 * 1 * 1 * 345 = 138). In other words, the design force for a concrete or masonry wall is 2.9 times greater than it is for another type of wall with the same mass.

If the concrete wall is being connected to a flexible diaphragm in a building where S_{DS} equals 1.0, the special provisions of Section 12.11.2.1 apply. This requires that the design force for the anchor be $0.80S_{DS}I$ times the weight of the wall tributary to the anchor. In the case of the concrete wall, this will require the anchor design force to be 276 plf (0.80 * 1 * 1 * 345 = 276); however, this too is less than 280, which is less than 400. Therefore, the design force for connections of the concrete wall to a flexible or rigid diaphragm where S_{DS} equals 1.0 is the same (i.e., 400 plf).

Another aspect of these high connection design forces that should be considered is the force for which the wall itself must be designed. ASCE 7, Section 12.11.1 requires that all structural walls in buildings assigned to SDC B or higher, regardless of materials of construction, be designed for a force normal to surface equal to $0.40S_{DS}I$ or 10%, whichever is greater, times the weight of the wall. Again considering the concrete wall cited above in a building where S_{DS} equals 0.32, the wall will be designed for a lateral force of 9 psf (0.40 * 0.32 * 1 * 69 = 9). On the other hand, the connection between the top of the wall and the diaphragm or other laterally supporting element must be designed for 280 plf. Since the height of the wall tributary to the connection is 5 feet, this suggests that the design force on the wall is 56 psf (280/5 = 56), or over 6 (56/9 = 6.22) times the force for which the wall is actually designed. One has to question the logic of requiring the connection to be designed for a force that is so much greater than the design force for the wall, given that the load factor on E is 1.0.

There are three other factors that need to be considered. First, many anchors that resist wall out-of-plane forces also must be designed to resist other forces at the same time. In fact, an anchor may be resisting vertical and horizontal shear forces in addition to the out-of-plane tensile force. Second, Appendix D of ACI 318 (Anchoring to Concrete) requires the anchor design strength be reduced 25% in structures assigned to Seismic Design Category C and higher. This is the same as requiring that the anchor design force be increased by 33-1/3%. This increase is in addition to the higher forces imposed by the requirements of ASCE 7 that apply only to concrete and masonry walls. Third, Appendix D of ACI 318 requires that in structures assigned to Seismic Design Category C and higher, the anchor design strength must be based on the failure of a ductile steel element. This means that the design strength based on all the concrete failure modes must be greater than the design strength based on the steel anchor. The only way to get around this is to increase the required anchor design force by 2.5 times per the IBC modification to ACI 318 Section D.3.3.5 in IBC Section 1908.1.16. All of these requirements are compounded by these minimum forces of ASCE 7 that apply only to concrete and masonry walls.

It is obvious that in a world that is rapidly embracing performance-based design, the requirement that anchorages for concrete and masonry walls be designed for a force of 280 or $400S_{DS}I$ pounds per linear foot of wall, whichever is greater, without considering the mass of the wall tributary to the anchor is not necessary and discriminates against these materials. In addition, by singling our walls of concrete and masonry, walls of other materials that could have and equal or greater mass per unit area are exempt from the requirement. Based on the foregoing, the anchorage force requirement for concrete and masonry walls in items "b" and "c" of Section 12.11.2 should be deleted, as should the provision in the exception to Section 12.14.7.5.

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

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

S96–07/08
1613.7 (New), 1613.7.1 (New)

Proponent: Jim Messersmith, Jr. PE, Portland Cement Association

Add new text as follows:

1613.7 General. The text of ASCE 7 shall be modified as indicated in Section 1613.7.1.

1613.7.1 ASCE 7, Section 11.7.5. Modify ASCE 7, Section 11.7.5 to read as follows:

11.7.5 Anchorage of walls. Walls shall be anchored to the roof and all floors and members that provide lateral support for the wall or that are supported by the wall. The anchorage shall provide a direct connection between the walls and the roof or floor construction. The connections shall be capable of resisting the forces specified in Section 11.7.3 applied horizontally, substituted for *E* in load combinations of Section 2.3 or 2.4.

Reason: Since ASCE 7-05 will be the loading standard referenced in the 2009 IBC, the only recourse to getting changes made to ASCE 7-05 is through modifications to the standard within the IBC.

The requirement that anchors attaching concrete and masonry walls to supporting construction be designed for a lower bound value of 280 pounds per linear foot is excessive and discriminatory considering that anchorage of walls of other materials, regardless of their mass, is required to be designed for a horizontal force of 5% of the weight of the wall tributary to the anchor.

To illustrate the punitive nature of the provision, consider two 10-foot high walls that are representative of walls used in single family dwellings and small commercial buildings; one a 5.5-inch thick concrete wall, the other a light-framed wall with 4-inch nominal masonry veneer anchored to the wall framing. The weight of the light-framed wall, including veneer, is estimated to be 45 psf; therefore, the weight tributary to an anchor at the top of the wall is 225 plf (45 * (10/2) = 225). The required design anchorage force for this wall is 11 plf (225 * 0.05 = 11). On the other hand, for the 5.5-inch concrete wall, which weighs 69 psf, the weight tributary to an anchor at the top of the wall is 345 plf (69 * (10/2) = 345). Based on the requirement that applies to walls of other than concrete or masonry, the required anchorage design force for the concrete wall should be 17 plf (345 * 0.05 = 17); however, 280 plf must be used to design the anchorage.

Let's examine how the 280 plf requirement compares to wind design. ASCE 7, Section 6.1.4.2 requires that components and cladding be designed for a minimum service level design wind pressure of 10 psf. For our example walls using this minimum design wind pressure, the strength level (factored) force at the top of the wall due to wind is 80 plf (10 * (10/2) * 1.6 = 80). Now let's determine what basic wind speed is required to produce a factored design force at the top of the wall of 280 plf (strength level). The service level (unfactored) force comparable to 280 plf is 175 plf (280/1.6 = 175). Since 175 plf is based on a tributary wall height of 5 feet, the design wind pressure is 35 psf (175/5 = 35). From ASCE 7, Figure 6-3, for a building in exposure B, height of 30 feet, K_{zt} of 1.0, and effective wind area of less than or equal to 10 square feet for wall area 4, the negative design pressure for a basic wind speed of 140 mph is 38.2 psf. Therefore, the requirement that the 5.5-inch, 10-foot high concrete wall be anchored against a force of 280 plf is the same as requiring that the connection be designed for a basic wind speed of approximately 135 mph.

It is obvious that in a world that is rapidly embracing performance-based design, the requirement that anchorages for concrete and masonry walls be designed for a force of 280 plf is not necessary and discriminates against these products. In addition, by singling out walls of concrete and masonry, walls of other materials that could have comparable mass per unit area are exempt from the requirement. Based on the foregoing, the requirement that anchorages for concrete and masonry walls be designed for 280 plf should be deleted.

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

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

S97–07/08
1613.7 (New), 1613.7.1 (New), 1613.7.2 (New),

Proponent: Jim Messersmith, Jr., PE, Portland Cement Association

Add new text as follows:

1613.7 General. The text of ASCE 7 shall be modified as indicated in Sections 1613.7.1 through 1613.7.2.

1613.7.1 ASCE 7, Section 12.11.2.2.3. Modify ASCE 7, Section 12.11.2.2.3 to read as follows:

12.11.2.2.3 Wood Diaphragms. In wood diaphragms, the continuous ties shall be in addition to the diaphragm sheathing. The diaphragm sheathing shall not be considered effective as providing the ties or struts required by this section.

> **Exception:** The diaphragm sheathing is permitted to serve as continuous ties provided all of the following are satisfied:

1. Panel edges in the direction orthogonal to the force being resisted are offset from panel to panel not less than 2 feet (610 mm),
2. Main framing members are spaced less than or equal to 2 feet (610 mm) on center,
3. The span of the diaphragm in the direction parallel to the main framing members is less than or equal to 40 feet (12,192 mm),
4. The aspect ratio of the diaphragm does not exceed 2.5 to 1, and
5. Connections shall extend into the diaphragm a sufficient distance to develop the force to be transferred into the diaphragm.

Anchorage shall not be accomplished by use of toenails or nails subject to withdrawal nor shall wood ledgers or framing be used in cross-gain bending or cross-grain tension.

1613.7.2 ASCE 7, Section 12.14.7.5.2. Modify ASCE 7, Section 12.14.7.5.2 to read as follows:

12.14.7.5.2 Wood Diaphragms. In wood diaphragms, the continuous ties shall be in addition to the diaphragm sheathing. The diaphragm sheathing shall not be considered effective as providing the ties or struts required by this section.

Exception: The diaphragm sheathing is permitted to serve as continuous ties provided all of the following are satisfied:

1. Panel edges in the direction orthogonal to the force being resisted are offset from panel to panel not less than 2 feet (610 mm),
2. Main framing members are spaced less than or equal to 2 feet (610 mm) on center,
3. The span of the diaphragm in the direction parallel to the main framing members is less than or equal to 40 feet (12,192 mm),
4. the aspect ratio of the diaphragm does not exceed 2.5 to 1, and
5. Connections shall extend into the diaphragm a sufficient distance to develop the force to be transferred into the diaphragm.

Anchorage shall not be accomplished by use of toenails or nails subject to withdrawal nor shall wood ledgers or framing be used in cross-gain bending or cross-grain tension.

Reason: Since ASCE 7-05 will be the loading standard referenced in the 2009 IBC, the only recourse to getting changes made to ASCE 7-05 is through modifications to the standard within the IBC.

The requirement in Sections 12.11.2.2.1 and 12.14.7.5.1 for continuous ties or struts between chords first appeared in the 1973 UBC following the 1971 San Fernando earthquake because of failures observed between connections of tilt-up concrete walls and their laterally supporting wood panelized roof systems. These provisions eventually found their way into the NEHRP Recommended Provisions and then into ASCE 7. Despite the fact that the adverse experience that prompted these stringent provisions occurred almost exclusively in large buildings with walls generally much taller than normally used in single family dwellings and small commercial buildings, the provisions apply to all structures regardless of size and height of wall. The requirement for continuous ties is compounded by the prohibition in Sections 12.11.2.2.3 and 12.14.7.5.2 on the use of wood sheathing as the continuous tie.

In the direction parallel to framing members, such as joists and trusses, the framing members can serve as the continuous tie, provided they are adequately connected at splices, or from member to member in a line. However, in the direction perpendicular to the framing members, a steel strap or other tie must be added at each wall-to-diaphragm connection, unless subdiaphragms are created. Therefore, a wall parallel to the main floor or roof framing members that is anchored to the floor or roof diaphragm at 24 inches on center must have continuous cross-ties at 24 inches on center. Incorporating these ties into a building is very labor-intensive.

Since the requirement for continuous cross-ties only applies to anchorage of concrete and masonry walls, it discriminates against these materials in favor of the use of walls constructed of other materials, even though there is nothing that assures that the other materials will have less mass per unit area. Also problematic is the lack of a similar requirement for wind design, even though the required wall anchorage forces for wind may be higher than for seismic for a given building. For example, consider a 5.5-inch thick concrete wall, which weighs 69 psf, in a building assigned to SDC D, with S_{DS} equal to 0.70. The height of the wall is 10 feet, and it will be attached to a structural wood diaphragm at the top for lateral support. The required strength design anchorage force is 193 plf (0.80*0.70*1*(5.5/12)*150*5 = 193); however, the lower-bound anchorage design force of 280 plf applies. The unfactored design wind pressure required to produce this force is 35 psf (280/(1.6*5) = 35 psf). This design wind pressure is produced by a basic wind speed of 112 mph in exposure C ((35/(0.00256*1*1*0.85*(1.1+0.18))^0.5 = 112). Therefore, if the same building was not required to be designed for seismic, other than provisions for SDC A, a concrete or masonry wall, or wall of any other material could be anchored to a flexible wood structural diaphragm without continuous ties at each anchor. Buildings are being designed and constructed regularly for this and higher wind speeds, which have walls more than 10 feet tall, without continuous ties.

In a world that is rapidly embracing performance-based design, the requirement that anchorages for concrete and masonry walls be accompanied by continuous ties, without considering whether the anchorage force can be resisted by the wood sheathing, is not necessary and discriminates against the two materials. By singling out walls of concrete and masonry, walls of other materials that could have higher mass and consequently higher required anchorage forces are exempt from the continuous tie requirement. Based on the foregoing, the exceptions being proposed to Sections 12.11.2.2.3 and 12.14.7.5.2 will provide some relief from the requirement for continuous ties without sacrificing life safety and property protection.

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

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

S98–07/08
1613.7 (New), 1613.7.1 (New)

Proponent: Jim Messersmith, Jr., PE, Portland Cement Association

Add new text as follows:

1613.7 General. The text of ASCE 7 shall be modified as indicated in Section 1613.7.1.

1613.7.1 ASCE 7, Section 12.14.7.5. Modify ASCE 7, Sections 12.14.7.5.1 through 12.14.7.5.4 to read as follows:

12.14.7.5.1 Transfer of anchorage forces into diaphragm. In buildings assigned to Seismic Design Category C, D or E, diaphragms shall be provided with continuous ties or struts between diaphragm chords to distribute these anchorage forces into the diaphragm. Added chords are permitted to be used to form subdiaphragms to transmit the anchorage forces to the main continuous cross-ties. The maximum length-to-width ratio of the structural subdiaphragm shall be 2.5 to 1. Connections and anchorages capable of resisting the prescribed forces shall be provided between the diaphragm and the attached components. Connections shall extend into the diaphragm a sufficient distance to develop the force transferred into the diaphragm.

12.14.7.5.2 Wood diaphragms. In buildings assigned to Seismic Design Category C, D or E, in wood diaphragms, the continuous ties shall be in addition to the diaphragm sheathing. Anchorage shall not be accomplished by use of toenails or nails subject to withdrawal nor shall wood ledgers or framing be used in cross-gain bending or cross-grain tension. The diaphragm sheathing shall not be considered effective as providing the ties or struts required by this section.

12.14.7.5.3 Metal deck diaphragms. In buildings assigned to Seismic Design Category C, D or E, in metal deck diaphragms, the metal deck shall not be used as the continuous ties required by this section in the direction perpendicular to the deck span.

12.14.7.5.4 Embedded straps. In buildings assigned to Seismic Design Category C, D or E, diaphragm to wall anchorage using embedded straps shall be attached to or hooked around the reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

Reason: Since ASCE 7-05 will be the loading standard referenced in the 2009 IBC, the only recourse to getting changes made to ASCE 7-05 is through modifications to the standard within the IBC.

The provisions in Sections 12.14.7.5.1, 12.14.7.5.2, 12.14.7.5.3 and 12.14.7.5.4 (Simplified Alternate Structural Design Criteria …) were extracted from Sections 12.11.2.2.1, 12.11.2.2.3, 12.11.2.2.4 and 12.11.2.2.5, respectively. In the latter sections the provisions apply to buildings assigned to Seismic Design Category C and higher; whereas, the identical provisions in Section 12.14.7.5 apply to buildings of SDC B and higher. While conservatism is warranted in some cases where simplified provisions are used, applying the provisions in question to buildings assigned to SDC B is not justified. Use of simplified provisions should be encouraged by eliminating unnecessary requirements, rather than discouraging their use by adding requirements that do not apply where regular procedures are used.

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

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

S99–07/08
1613.7 (New), 1613.7.1 (New), 1613.7.2 (New), 1613.7.3 (New), 1613.7.4 (New)

Proponent: Robert E. Bachman, S.E., Consulting Structural Engineers, representing The Consortium of Organizations for Strong-Motion Observation Systems

Add new text as follows:

1613.7 Earthquake recording instrumentation. Buildings that are assigned to Seismic Design Category D, E or F shall be provided with earthquake recording instrumentation as specified in Sections 1613.7.1 through 1613.7.4.

1613.7.1 General. Buildings exceeding six stories above grade plane with an aggregate floor area of 60,000 square feet (5574 m^2) or more, and every building exceeding 10 stories above grade plane regardless of floor area, shall be provided with than three approved recording accelerographs.

The accelerographs shall be interconnected for common start and common timing.

1613.7.2 Location. The instruments shall be located in the basement, midportion, and near the top of the building. Each instrument shall be located so that access is maintained at all times and is unobstructed by room contents. A sign stating MAINTAIN CLEAR ACCESS TO THIS INSTRUMENT shall be posted in a conspicuous location.

1613.7.3 Maintenance. Maintenance and service of the instrumentation shall be provided by the owner of the building, subject to the approval of the building official. Data produced by the instrument shall be made available to the building official on request.

1613.7.4 Instrumentation of existing buildings. All owners of existing structures selected by the jurisdiction authorities shall provide accessible space for the installation of appropriate earthquake-recording instruments. Location of said instruments shall be determined by the jurisdiction authorities. The jurisdiction shall make arrangements to provide, maintain and service the instruments. Data shall be the property of the jurisdiction, but copies of individual records shall be made available to the public on request and the payment of an appropriate fee.

Reason: Earthquake recording instrumentation measurements provide fundamental information needed to cost effectively improve the seismic performance of buildings. The wording of the added Section is taken directly form Appendix Chapter 16, Division II of the 1997 UBC with the exception of the triggering language which is now associated with Seismic Design Category instead of Seismic Zone. In the 1997 UBC, the provisions applied to Seismic Zones 3 & 4. In this proposal, the provisions apply to Seismic Design Categories D, E and F. When the IBC was created, this section was apparently inadvertently not included. The code change proposal is intended to correct this oversight.

This change will minimally increase the cost of new construction (in the range of 0.01% to 0.1%) for buildings over 160 feet in height in high seismic areas. In areas where the UBC requirements were enforced there would be no increase in cost. No increase for other buildings.

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

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

S100–07/08

1613.7 (New), 3403.2.3 (New)

Proponent: James S. Lai, SE, representing Structural Engineers Association of California

Add new text as follows:

1613.7 Earthquake recording instrumentation. Where the mapped spectral accelerations for a 1-second period S_1 is equal to or greater than 0.6 g and the building is assigned to Seismic Design Category is D, E or F, every building exceeding six stories above grade plane with an aggregate floor area of 60,000 square feet (5574 m^2) or more, and every building exceeding 10 stories above grade plane regardless of floor area, shall be equipped with not less than three approved recording accelerographs.The accelerographs shall be interconnected for common start and common timing. The location and installation of the earthquake recording instrumentation shall be approved by building official.

1613.7.1 Location. The instruments shall be located in the basement, mid-portion, and near the top of the building. Each instrument shall be located so that access is maintained at all times and is unobstructed by room contents. A sign stating MAINTAIN CLEAR ACCESS TO THIS INSTRUMENT shall be posted in a conspicuous location.

1613.7.2 Maintenance. Maintenance and service of the instruments shall be provided by the owner of the building, subject to the approval of the building official. Data produced by the instruments shall be made available to the building official on request.

1613.7.3 Instrumentation of existing buildings. When existing buildings or structures are selected by building official for installation of earthquake-recording instruments, installation shall be in accordance with Section 3403.2.3.

3403.2.3 Earthquake instrumentation of existing buildings. All owners of existing buildings or structures selected by building officials shall provide accessible space for the installation of appropriate earthquake-recording instruments. See Section 1613.7 for criteria on earthquake recording instrumentation. Location of said instruments shall be determined by the building official. The building official shall make arrangements to provide, maintain and service the instruments. Data shall be the property of the jurisdiction, but copies of individual records shall be made available to the public on request and the payment of an appropriate fee.

(Renumber subsequent sections)

Reason: Purpose: Provide instrumentation for mid-rise and high-rise buildings in areas with higher seismic activity.

Justification: Much of the current understanding of the behavior of the buildings has been through earthquake recorded data. The instruments will provide the most basic data for the performance of the lateral load resisting systems in the building and will also help in determining the damage state of a building after an earthquake. This information is essential for further understanding of seismic behavior of buildings. The proposed sections were present in previous editions of the Uniform Building Code and the California Building Codes.

The proposed section is limited to sites where S_1 is greater than 0.6g and Seismic Design Category is D, E or F. The limit of S_1 being equal to or greater than 0.6g characterizes sites as being in high seismic zones. The same limit is used for computing the minimum base shear in high seismic zones per ASCE 7-05, Equation 12.8-6.

Instrumentation in certain significant existing buildings or structures at selected locations have in the past helped to the understanding and monitoring of different building types subjected to possible ground motions and the influence of ground motion propagation. Such selection is usually supported by public funding or grants. Valuable data collected from instrumentation will help to better protect the public in future seismic events.

The code change proposal will increase the cost of construction in high seismic areas. The cost will depend on the sophistication of the instruments. One-time costs of the most basic recording instruments are currently estimated at $15,000 to $20,000 including installation for the building. However, the costs are negligible when compared to construction costs of mid-rise and high-rise buildings.

Cost Impact: The code change proposal will increase the cost of construction in high seismic areas. The cost will depend on the sophistication of the instruments. One-time costs of the most basic recording instruments are currently estimated at $15,000 to $20,000 including installation for the building. However, the costs are negligible when compared to construction costs of mid-rise and high-rise buildings.

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

S101 –07/08

1614 (New)

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

Add new section as follows:

SECTION 1614
STRUCTURAL INTEGRITY

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

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

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

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

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

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

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

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

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

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

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

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

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

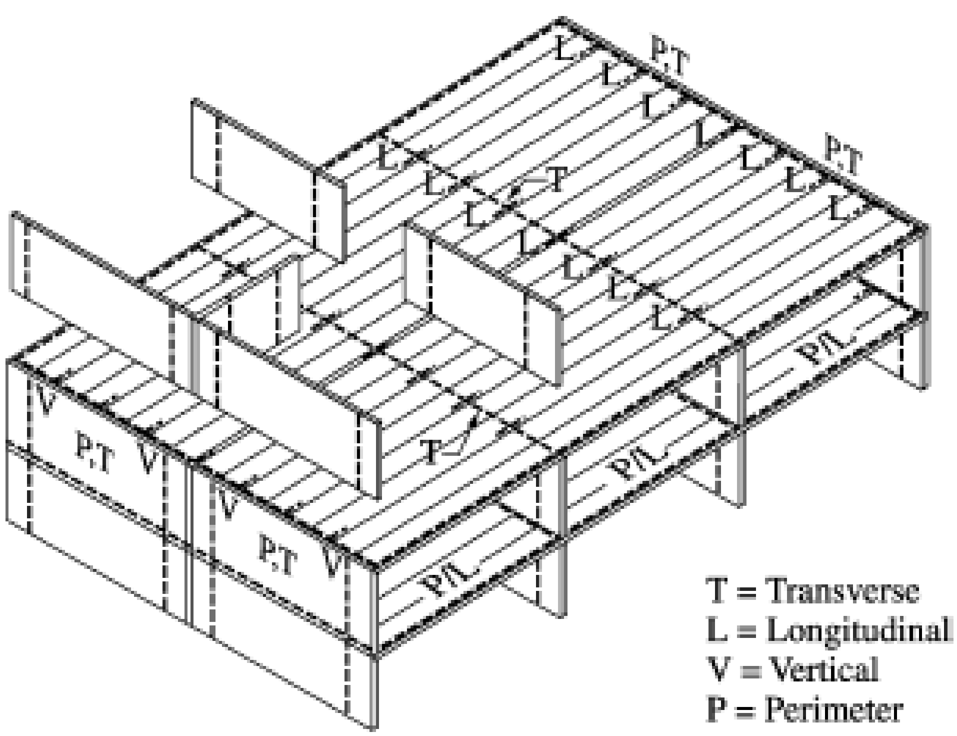

FIGURE 1614.4
LONGITUDINAL, PERIMETER, TRANSVERSE AND VERTICAL TIES

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

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

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

$$T_T = wLs \le \alpha_T s \qquad \textbf{(Equation 16-45)}$$

where:

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

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

S = the spacing between ties, ft (m)

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

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

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

$$T_p = 200w \le \beta_T \qquad \textbf{(Equation 16-46)}$$

For SI:

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

where

w = as defined in Section 1614.4.2.1

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

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

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

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

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

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

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

S102–07/08
1702

Proponent: Tony Crimi, A.C. Consulting Solutions Inc., representing North American Insulation Manufacturers' Association (NAIMA)

THIS CODE CHANGE PROPOSAL IS ON THE AGENDA OF THE IBC FIRE SAFETY CODE DEVELOPMENT COMMITTEE. SEE THE HEARING ORDER FOR THE IBC FIRE SAFETY CODE DEVELOPMENT COMMITTEE.

Add new definitions as follows:

SECTION 1702
DEFINITIONS

INTUMESCENT FIRE RESISTANT COATINGS. Thin film liquid mixture applied to substrates by brush, roller, spray or trowel which expands into a protective foamed layer to provide fire-resistant protection of the substrates when exposed to flame or intense heat.

MASTIC FIRE RESISTANT COATINGS. Liquid mixture applied to a substrate by brush, roller, spray or trowel that provides fire resistant protection of a substrate when exposed to flame or intense heat.

Reason: Purpose: To introduce new definition for Mastic and Intumescent Fire Resistant Coatings which are described in Section 1704.11.

Section 1704.11 provides requirements for Special Inspections for mastic and intumescent fire-resistant coatings applied to structural elements and decks, but the IBC does not contain any definition describing these materials.

Section 1704.11 requires Special inspections for mastic and intumescent fire-resistant coatings applied to structural elements and decks be in accordance with AWCI 12-B which is entitled "Field Applied Thin-Film Intumescent Fire-Resistive Materials". Special inspections are also required to be based on the fire-resistance design as designated in the approved construction documents. However, neither the IBC nor the AWCI 12-B Standard provides any description or definition for these materials. In order to ensure that the Special Inspection procedures are appropriate, some definitions of these materials should be incorporated into the IBC. This would bring these in line with Sprayed Fire Resistive Materials (SFRM).

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

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

S103–07/08
1703.1.1

Proponent: D. Kirk Harman, P.E., S.E., The Harman Group, Inc., representing The NCSEA Code Advisory Committee, Quality Assurance and Special Inspection Subcommittee

Revise as follows:

1703.1 Approved agency. An approved agency shall provide all information as necessary for the building official to determine that the agency meets the applicable requirements.

1703.1.1 Independen~~t~~ce. An approved agency shall be objective, ~~and~~ competent and independent from the contractor who's work is being inspected. The agency shall also disclose possible conflicts of interest so that objectivity can be confirmed.

Reason: This change is to clarify the term "Independent". NCSEA believes that for the Special Inspections process to have adequate integrity, it is important that special inspections not be performed by employees or consultants of the contractor. However, the Code should not be interpreted to require independence from the various design professionals who have undertaken design on the project. The Special Inspections are for assurance of quality of construction and conformance to design standards. They are in no way a check of the design. The proposed edits to the existing language in the code clarify these issues.

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

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

S104–07/08
1704.1

Proponent: Gary J. Ehrlich, P.E., National Association of Home Builders, representing National Association of Home Builders

Revise as follows:

1704.1 (Supp) General. Where application is made for construction as described in this section, the owner or the registered design professional in responsible charge acting as the owner=s agent shall employ one or more special inspectors to provide inspections during construction on the types of work listed under Section 1704. These inspections are in addition to the inspections identified in Section 109.

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

Exceptions:

1. Special inspections are not required for work of a minor nature or as warranted by conditions in the jurisdiction as approved by the building official.
2. Special inspections are not required for building components unless the design involves the practice of professional engineering or architecture as defined by applicable state statutes and regulations governing the professional registration and certification of engineers or architects.
3. Unless otherwise required by the building official, special inspections are not required for occupancies in Group U that are accessory to a residential occupancy including, but not limited to, those listed in Section 312.1.
4. Special inspections are not required for structures designed and constructed in accordance with the conventional construction provisions of Section 2308.

Reason: A proposal in the previous cycle (RB31-06/07) struck the exemption for R-3 structures, subjecting those one- and two-family dwellings and townhouses which happen to fall under the IBC rather than the IRC to special inspections on top of the standard building department inspections. In addition, other structures falling under an R-3 occupancy (group homes, day care) will be subject to special inspections for all elements of their construction. Yet, contrary to the proponent's contentions, many of these structures will be simple structures and conform fully to the conventional construction provisions of Section 2308. They will not necessarily have the complex roof systems, steel framing, reinforced masonry and other elements the proponent offered as justification for removing the exemption.

Section 1704.1.1 states that the registered design professional is not required to prepare, and the permit applicant not required to submit, a statement of special inspections for structures designed and constructed per Section 2308. The implication is that these simple structures do not therefore require special inspections, which are really intended for structures that due to their complexity or use of unusual construction materials and methods require observation by an individual with the qualifications and experience of a special inspector. Thus it should be clarified that conventionally-framed structures are also exempt from the special inspections themselves, and not just from the documentation requirement.

NAHB asks for your support of this proposal.

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

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

S105–07/08
1704.1

Proponent: Gary J. Ehrlich, P.E., National Association of Home Builders, representing National Association of Home Builders

Revise as follows:

1704.1 (Supp) General. Where application is made for construction as described in this section, the owner or the registered design professional in responsible charge acting as the owner=s agent shall employ one or more special inspectors to provide inspections during construction on the types of work listed under Section 1704. These inspections are in addition to the inspections identified in Section 109.

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

Exceptions:

1. Special inspections are not required for work of a minor nature or as warranted by conditions in the jurisdiction as approved by the building official.
2. Special inspections are not required for building components unless the design involves the practice of professional engineering or architecture as defined by applicable state statutes and regulations governing the professional registration and certification of engineers or architects.
3. Unless otherwise required by the building official, special inspections are not required for occupancies in Group U that are accessory to a residential occupancy including, but not limited to, those listed in Section 312.1.
4. <u>Special inspections are not required for detached one- and two-family structures and townhouses governed by this code and designed and constructed in accordance with the conventional construction provisions of Section 2308.</u>

Reason: A proposal in the previous cycle (RB31-06/07) struck the exemption for R-3 structures, subjecting those detached one- and two-family dwellings and townhouses which happen to fall under the IBC rather than the IRC to special inspections on top of the standard building department inspections. Yet, contrary to the proponent's contentions, many of these dwellings and townhouses will be simple structures and conform fully to the conventional construction provisions of Section 2308. They will fall under the IBC only because a local jurisdiction has not yet adopted the IRC, or because the structure does not qualify for the IRC due to wind or seismic limits. They will also not necessarily have the complex roof systems, steel framing, reinforced masonry and other elements the proponent offered as justification for removing the exemption.

Section 1704.1.1 states that the registered design professional is not required to prepare, and the permit applicant not required to submit, a statement of special inspections for structures designed and constructed per Section 2308. The implication is that these simple structures do not therefore require special inspections, which are really intended for structures that due to their complexity or use of unusual construction materials and methods require observation by an individual with the qualifications and experience of a special inspector. Thus it should be clarified that one- and two-family dwellings and townhouses built under the IBC and using conventional construction provisions are also exempt from the special inspections themselves, and not just from the documentation requirement.

NAHB asks for your support of this proposal.

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

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

S106–07/08
1704.1

Proponent: Bill Sliwinski PE, Campbell Construction Inc. representing Structural Engineer Association of Ohio (SEAoO)

Revise as follows:

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

Exceptions:

1. Special inspections are not required for work of a minor nature ~~or as warranted by conditions in the jurisdiction as approved by the building official.~~ if approved by the building official and the design professional responsible for the design of the structure. Structures exceeding 2 stories above grade plane or 25 feet (7620 mm) to the highest point of the structure or with a gross area exceeding 25,000 square feet (2323 m^2) shall not be considered work of a minor nature. Structures of any size assigned to Categories III or IV in accordance with Table 1604.5 shall not be considered work of a minor nature. Unless otherwise required by the building official, special inspections are not required for buildings assigned to Category I in accordance with Table 1604.5.
2. Special inspections are not required for building components unless the design involves the practice of professional engineering or architecture as defined by applicable state statutes and regulations governing the professional registration and certification of engineers or architects.
3. Unless otherwise required by the building official, special inspections are not required for occupancies in Group U that are accessory to a residential occupancy including, but not limited to, those listed in Section 312.1.

Reason: The purpose of the code change is to clarify the definition of "minor structure" to allow the uniform interpretation of the code. The proposal is superior to the current provision because it provides a tool for structural engineers and building officials to correctly interpret the code intent. The current code provision allows wide interpretation of "minor structure". Almost identical projects are treated differently in different jurisdictions. Substantiation is based on the Kentucky Building Code. The Kentucky Building Code has implemented the code change and it has worked very well.

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

Analysis: Is deletion of the phrase "as warranted by conditions in the jurisdiction" intended?

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

S107–07/08

1704.1, 1704.1.2

Proponent: D. Kirk Harman, PE, SE, The Harman Group, Inc., representing The National Council of Structural Engineers (NCSEA) Code Advisory Committee Quality Assurance and Special Inspection Subcommittee

Revise as follows:

1704.1 (Supp) General. Where application is made for construction as described in this section, the owner or the registered design professional in responsible charge acting as the owner's agent shall employ one or more ~~special inspectors~~ approved agencies or provide special inspection services to ~~provide~~ perform inspections during construction on the types of work listed under Section 1704. These inspections are in addition to the inspections identified in Section 109.

The special inspector shall be a qualified person who shall demonstrate competence, to the satisfaction of the building official, for the inspection of the particular type of construction or operation requiring special inspection. The registered design professional in responsible charge and engineers of record involved in the design of the project are permitted to act as the approved agency and their personnel are permitted to inspect the work designed by them. The special inspector shall provide written documentation to the building official demonstrating their competence and relevant experience or training. Experience or training shall be considered relevant when the documented experience or training is related in complexity to the same type of special inspection activities for projects of similar complexity and material qualities. These qualifications are in addition to qualifications specified in other sections of this code.

Exceptions:

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

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

1704.1.2 Report requirement. Special inspectors shall keep records of inspections. The special inspector shall furnish inspection reports to the building official, and to the registered design professional in responsible charge. Reports shall indicate that work inspected was or was not completed ~~done~~ in conformance to approved construction documents. Discrepancies shall be brought to the immediate attention of the registered design professional in responsible charge and the contractor for correction. If the discrepancies are not corrected, the discrepancies shall be brought to the attention of the building official and to the registered design professional in responsible charge prior to the completion of that phase of the work. A final report documenting required special inspections and correction of any discrepancies noted in the inspections shall be submitted at a point in time agreed upon prior to the start of work by the permit applicant and the building official ~~prior to the start of work~~.

Reason: NCSEA has received input from structural engineers in various locations throughout the United States stating that building officials sometimes take the position that the registered design professional in responsible charge and/or engineers of record responsible for design on a project are prohibited by the Code from performing Special Inspections. NCSEA believes that this is an incorrect interpretation of the Code. Special Inspections are a process to help assure conformance with design requirements and are not in any way intended to be a check of the design. This change is to clarify that the registered design professional and/or engineers of record involved in the design of the project may perform the special inspections.

NCSEA believes that Section 1704.1.2 does not clearly require reporting of incomplete work at all and does not require notification to the Registered Design Professional in Responsible Charge as to the outcome of Special Inspections. This change clarifies this in Section 1704.1.2.

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

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

S108–07/08

1704.2, 1704.2.1

Proponent: D. Kirk Harman, PS., S.E., The Harman Group, Inc., representing The National Council of Structural Engineers Associations (NCSEA) Code Advisory Committee, Quality Assurance and Special Inspection Subcommittee

1. Revise as follows:

1704.2 Inspection of fabricators. ~~Where fabrication of structural load-bearing members and assemblies is being performed on the premises of a fabricator's shop, special inspection of the fabricated items shall be required by this section and as required elsewhere in this code~~Where structural elements subject to special inspections are fabricated off-site in a fabricator's shop, inspections and tests stipulated in Sections 1704.3, 1704.4, 1704.5 and 1704.6 shall be performed in the fabricator's shop.

Exceptions:

1. Inspections and tests of fabricated items are permitted to be performed at the site rather than in the shop when fabricated items are accessible for inspection and testing at the site and the contractor has sufficient equipment and resources to correct identified deficiencies at the site as approved by the Building Official.
2. Special Inspection of fabricated items are not required when the work is performed on the premises of a fabricator of structural steel that has been certified by the AISC Fabricator Certification Program or a fabricator of precast concrete that has been certified by the PCI Plant Certification Program or equivalent program approved by the building official. Such fabricators shall maintain detailed reports documenting inspections and tests performed by the fabricator's quality control personnel.

2. Delete without substitution:

~~**1704.2.1 Fabrication and implementation procedures.** The special inspector shall verify that the fabricator maintains detailed fabrication and quality control procedures that provide a basis for inspection control of the workmanship and the fabricator's ability to conform to approved construction documents and referenced standards. The special inspector shall review the procedures for completeness and adequacy relative to the code requirements for the fabricator's scope of work.~~

~~**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.~~

~~**1704.2.2 Fabricator approval.** Special inspections required by this code are not required where the work is done on the premises of a fabricator registered and approved to perform such work without special inspection. Approval shall be based upon review of the fabricator's written procedural and quality control manuals and periodic auditing of fabrication practices by an approved special inspection agency. At completion of fabrication, the approved fabricator shall submit a certificate of compliance to the building official stating that the work was performed in accordance with the approved construction documents.~~

Reason: NCSEA believes that Special Inspections should be performed of the fabricated items not of the fabricator's quality control procedures which are means and methods of construction. In certain instances it is more practical to perform inspections in the field rather than in the shop. The criteria for exemption of fabricators from shop inspection have been clearly defined based on programs currently in place that would satisfy these requirements.

The code change proposal may increase the cost of construction a moderate amount due to some fabrication shop inspections being done that previously were not.

Cost Impact: The code change proposal will increase the cost of construction a moderate amount due to some fabrications shop inspections being done that previously were not.

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

S109–07/08
1704.2.2

Proponent: Bonnie Manley, American Iron and Steel Institute, representing American Institute of Steel Construction

Revise as follows:

1704.2.2 Fabricator approval. Special inspections required by ~~this code~~ Section 1704 are not required where the work is done on the premises of a fabricator registered and approved to perform such work without special inspection. Approval shall be based upon review of the fabricator's written procedural and quality control manuals and periodic auditing of fabrication practices by an approved special inspection agency. At completion of fabrication, the approved fabricator shall submit a certificate of compliance to the building official stating that the work was performed in accordance with the approved construction documents.

Reason: This modification attempts to clarify exactly which inspections are permitted to be waived when work is done by a registered and approved fabricator. As written now, it could be interpreted to mean that the special inspections for seismic resistance required by Section 1707.2 could be waived. This is not appropriate and needs to be corrected.

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

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

S110–07/08
1704.3, 1704.3.1.1 (New)

Proponent: Bonnie Manley, American Iron and Steel Institute

1. Revise as follows:

1704.3 Steel construction. The special inspections for steel elements of buildings and structures shall be as required by Section 1704.3 and Table 1704.3.

Exceptions:

1. Special inspection of the steel fabrication process shall not be required where the fabricator does not perform any welding, thermal cutting or heating operation of any kind as part of the fabrication process. In such cases, the fabricator shall be required to submit a detailed procedure for material control that

demonstrates the fabricator's ability to maintain suitable records and procedures such that, at any time during the fabrication process, the material specification, grade and mill test reports for the main stress-carrying elements are capable of being determined.

2. The special inspector need not be continuously present during welding of the following items, provided the materials, welding procedures and qualifications of welders are verified prior to the start of the work; periodic inspections are made of the work in progress; and a visual inspection of all welds is made prior to completion or prior to shipment of shop welding.
    2.1. Single-pass fillet welds not exceeding 5/16 inch (7.9 mm) in size.
    2.2. Floor and roof deck welding.
    2.3. Welded studs when used for structural diaphragm.
    2.4. Welded sheet steel for cold-formed steel light frame construction ~~framing~~ members ~~such as studs and joists~~.
    2.5. Welding of stairs and railing systems.

2. Add new text as follows:

1704.3.1.1 Cold-formed steel. Welding inspection and welding inspector qualification for cold-formed steel, including metal floor and roof decks, shall be in accordance with AWS D1.3.

Reason: IBC Section 1704.3, Exception 2.4 was corrected to match the terminology used in IBC Section 2210. IBC Section 1704.3.1 on welding currently only references AWS D1.1. Since "Steel Construction" covers steel other than just structural steel, referencing AWS D1.1 alone is not sufficient. AWS D1.3 covers cold-formed steel and is added as a new subsection. Please see companion change which moves the reference to AWS D1.1 into a new subsection for "Structural Steel'."

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

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

S111–07/08

1704.3.1, 1704.3.1.1 (New), 1704.3.3

Proponent: Bonnie Manley, American Iron and Steel Institute, representing American Institute of Steel Construction

Revise as follows:

1704.3.1 Welding. Welding inspection ~~shall be in compliance with AWS D1.1. The basis for~~ and welding inspector qualification shall be in accordance with this section ~~AWS D1.1~~.

1704.3.1.1 Structural steel. Welding inspection and welding inspector qualification for structural steel shall be in accordance with AWS D1.1.

1704.3.3 High-strength bolts. Installation of high-strength bolts shall be ~~periodically~~ inspected in accordance with AISC ~~specifications~~ 360.

Reason: IBC Section 1704.3.1 on welding currently only references AWS D1.1, which specifically applies to structural steel members. This change clarifies the code.

The modification to Section 1704.3.3 is intended to correct and clarify the code. The term 'periodically' is recommended for deletion because 1704.3.3.3 requires continuous inspection for the two methods of installation. Also, the appropriate reference document for this section is AISC 360, Specification for Structural Steel Buildings.

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

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

S112 –07/08
Table 1704.3

Proponent: Bonnie Manley, American Iron and Steel Institute, representing American Institute of Steel Construction

Revise table as follows:

TABLE 1704.3
REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]	IBC REFERENCE
1. Material verification of high-strength bolts, nuts and washers:				
a. Identification markings to conform to ASTM standards specified in the approved construction documents.	—	X	~~Applicable ASTM material specifications:~~ AISC 360, Section A3.3 and applicable ASTM material standards	—
b. Manufacturer's certificate of compliance required	—	X	—	—
2. Inspection of high-strength bolting:				
a. ~~Bearing type connections~~. Snug-tight joints.	—	X	AISC 360, Section M2.5	1704.3.3
b. Pretensioned and slip- ~~Slip~~-critical joints using turn-of-nut with matchmarking, twist-off bolt, or direct tension indicator methods of installation. ~~connections.~~	~~X~~ —	X		
c. Pretensioned and slip-critical joints using turn-of-nut without matchmarking or calibrated wrench methods of installation.	X	—		
3. Material verification of ~~structural~~ steel:				
a. For structural steel, identification markings to conform to AISC 360	—	X	AISC 360, Section M5.5	
~~a.~~ b. For other steel, ~~Identification~~ identification markings to conform to ASTM standards specified in the approved construction documents.	—	— X	~~ASTM A 6 or ASTM A 568~~ Applicable ASTM material standards	~~1708.4~~
~~b.~~ c. Manufacturer's certified mill test reports.	—	— X	~~ASTM A6 or ASTM A568~~	~~1708.4~~
4. Material verification of weld filler materials:				
a. Identification markings to conform to AWS specification in the approved construction documents.	—	— X	AISC 360, Section A3.5 and Applicable AWS A5 documents	—
b. Manufacturer's certificate of compliance required.	—	— X	—	—
5. Inspection of welding: a. ~~Structural~~ Steel:				
1) Complete and partial joint penetration groove welds.	X	—	AWS D1.1	1704.3.1
2) Multipass fillet welds	X	—		
3) Single-pass fillet welds > 5/16"	X	—		
4) Plug and slot welds	X	—		
~~4)~~ 5) Single-pass fillet welds ≤ 5/16"	—	X		
~~5)~~ 6) Floor and roof deck welds.	—	X	AWS D1.3	
b. Reinforcing steel:			~~AWS D1.4 ACI 318; 3.5.2~~	
1) Verification of weldability of reinforcing steel other than ASTM A706	—	X	AWS D1.4 or ACI 318: Section 3.5.2	—
2) Reinforcing steel-resisting flexural and axial forces in intermediate and special moment frames, and boundary elements of special reinforced concrete shear walls and shear reinforcement.	X	—		
3) Shear reinforcement.	X	—		

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]	IBC REFERENCE
4) Other reinforcing steel	—	X		
6. Inspection of steel frame joint details for compliance with approved construction documents:		~~X~~		~~1704.3.2~~
a. Details such as bracing and stiffening.	—	— X	—	1704.3.2
b. Member locations.	—	— X		
c. Application of joint details at each connection.	—	X		

For SI: 1 inch = 25.4 mm.
a. Where applicable, see also Section 1707.1, Special inspection for seismic resistance.

Reason: Modifications to Item #1: The modification in Item #1a clarifies the relationship between the referenced standards and rearranges them for better flow.

Modifications to Item #2a: This modification corrects the terminology from 'bearing-type connections' to 'snug-tight joints.'
Modifications to Item #2b and the addition of Item #2c: These modifications update the terminology used and distinguish between snug tight joints, and pretensioned and slip-critical joints using matchmarked turn-of-nut, twist-off bolt, or direct tension indicator methods of installation, which require periodic inspection in the text of 1704.3.3.2, and pretensioned and slip-critical joints using non-matchmarked turn-of-nut or calibrated wrench methods of installation, which require continuous inspection in 1704.3.3.3

Modifications to Item #3: The modification to Item #3 clarifies the application of this item and eliminates potential confusion with the use of the existing term "structural steel member," which includes only rolled steel structural shapes. Also, reference to the specific ASTM standards has been changed to the more generalized "applicable ASTM material standards" to match the verbiage in Table 1704, Item #1. Additionally, the modifications to Items #3a, 3b and 3c clarify that periodic inspection is required. AISC 360 and the applicable ASTM standards do not necessarily address the frequency of inspection of material identification using the terms "periodic" or "continuous". Instead, the proposal to do material identification on a periodic basis brings it into agreement with bolt material inspection (Items #1a and 1b) and rebar weldability (Item #5b1), also in Table 1704.4 Items #1 and 4, Table 1704.5.1 Item #1a, and Table 1704.5.3 Item #1a. Finally, the reference to Section 1708.4 is not appropriate and has been deleted.

Modifications to Item #4: The modifications to Items #4a and 4b clarify that periodic inspection is required. AISC 360 and the AWS A5 documents do not necessarily address the frequency of inspection of material identification using the terms "periodic" or "continuous". Instead, the proposal to do material identification on a periodic basis brings it into agreement with bolt material inspection (Items #1a and 1b) and rebar weldability (Item #5b1), also in Table 1704.4 Items #1 and 4, Table 1704.5.1 Item #1a, and Table 1704.5.3 Item #1a.

Modification to Item #5a and #5a(1): These modifications correct the terminology.
Addition of new Item #5a(4): The current table is missing an entry for plug and slot welds. The plug and slot weld provision is proposed to be included as continuous because it is not specifically listed as qualifying for periodic inspection in 1704.3.2.

Modification to Item #5b: This change is editorial. In keeping with the style of the table, the applicable referenced standards for Items #5b(1) and 5b (2) have been relocated to the cell below, which has been merged between #5b(1) and 5b (2). In addition, the relationship between the two referenced standards has been clarified.

Modification to Item #6: This change is editorial. In keeping with the style of the table, the IBC reference for Items #6 has been relocated to the cell below, which has been merged between #6a, 6b, and 6c.

Modifications to Item #6a, b, and c: The modifications to Items #6a, 6b and 6c clarify that periodic inspection is required by the IBC reference, Section 1704.3.2.

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

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

S113–07/08

1704.3, Table 1704.3, 1707.2

Proponent: Philip Brazil, PE, SE, Reid Middleton, Inc., representing himself

Revise as follows:

1704.3 Steel construction. The special inspections for steel elements of buildings and structures shall be as required by Section 1704.3 and Table 1704.3.

Exceptions:

1. Special inspection of the steel fabrication process shall not be required where the fabricator does not perform any welding, thermal cutting or heating operation of any kind as part of the fabrication process. In such cases, the fabricator shall be required to submit a detailed procedure for material control that demonstrates the fabricator's ability to maintain suitable records and procedures such that, at any time during the fabrication process, the material specification, grade and mill test reports for the main stress-carrying elements are capable of being determined.

2. The special inspector need not be continuously present during welding of the following items, provided the materials, welding procedures and qualifications of welders are verified prior to the start of the work; periodic inspections are made of the work in progress; and a visual inspection of all welds is made prior to completion or prior to shipment of shop welding.
   2.1. Single-pass fillet welds not exceeding 5/16 inch (7.9 mm) in size.
   2.2. Floor and roof deck welding.
   2.3. Welded studs ~~when~~ where used for structural diaphragms or composite systems.
   2.4. Welded sheet steel for cold-formed steel framing members such as studs and joists.
   2.5. Welding of stairs and railing systems.

TABLE 1704.3
REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]	IBC REFERENCE
5. Inspection of welding: a. Structural steel and steel joists:	X	-		
1) Complete and partial penetration groove welds.	X	-	AWS D1.1	1704.3.1
2) Multipass fillet welds.	X	-		
3) Single-pass fillet welds > 5/16”	X	-		
4) Single-pass fillet welds ≤ 5/16”	-	X		
5) Welds at stairs and railing systems	-	X		
~~5~~ 6) Floor and roof deck welds	-	X	AWS D1.3	—
7) Welded studs at structural diaphragms and composite systems	-	X	—	—

Portions of table not shown do not change

1707.2 Structural steel. Continuous special inspection is required for structural welding in accordance with AISC 341.

Exceptions:

1. Single-pass fillet welds not exceeding 5/16 inch (7.9 mm) in size.
2. Floor and roof deck welding.
3. Welded studs where used for structural diaphragms or composite systems.
4. Welding of stairs and railing systems.

Reason: Section 1704.3 specifies special inspection for steel elements as required by Section 1704.3 and Table 1704.3. Exception #2 to Section 1704.3 permits welding of five types of steel elements to be inspected periodically. Item #5a of Table 1704.3 specifies Items 2.1 and 2.2 of Exception #2 but not Items 2.3, 2.4 and 2.5. Section 1707.2 requires continuous special inspection for structural welding except for two items permitted to be inspected periodically. These items match Items 2.1 and 2.2 in Exception #2 to Section 1704.3 but Items 2.3, 2.4 and 2.5 are similarly not specified. The purpose for this proposal is to correlate these provisions by adding listings to Table 1704.3 and the Exception to Section 1707.2 for consistency with Exception #2 to Section 1704.3. The listing of welded sheet steel for cold-formed steel framing members in Exception #2 to Section 1704.3 is appropriate because Section 1704.3 applies to cold-formed steel construction as well structural steel. Adding a listing to Item #5 of Table 1704.3 or the Exception to Section 1707.2, however, would not be appropriate because both are limited to structural steel.

At the listings for welded studs, “structural diaphragm” is changed to “structural diaphragms or composite systems” to correct what is evidently an oversight during the original drafting of the IBC (refer to Exception 2.3 to 1997 UBC Section 1701.5.5.1).

In Item #5a of Table 1704.3, “steel joists” is added to “structural steel” because the definitions of Section 2202.1 distinguish between cold-formed steel construction, steel joists, and structural steel members. The omission of steel joists from Item #5a of Table 1704.3 has the effect of exempting the welding of steel joist members to their supports from the requirements for special inspection. Note that Section 1704.2.2 exempts from the requirement for special inspection work done on the premises of a fabricator registered and approved to perform such work without special inspection.

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

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

S114–07/08
1704.3.4 (New), 1707.4

Proponent: D. Kirk Harman, The Harman Group, representing National Council of Structural Engineers Associations (NCSEA), Code Advisory Committee Quality Assurance and Special Inspection Subcommittee

1. Add new text as follows:

1704.3.4 Cold-formed steel framing. The special inspections for cold-formed steel framing shall be as required by Table 1704.3.4.

> **Exception:** Cold-formed steel framing for non-load bearing partitions, not exposed to wind loading or designed to act as shear walls, are exempt from special inspections.

TABLE 1704.3.4
REQUIRED VERIFICATION AND INSPECTION OF COLD-FORMED STEEL FRAMING

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC
1. Inspect size and thickness of members		X
2. Inspect mechanical connections including screws, powder actuated fasteners, bolting, anchor bolts, tie downs, and anchors.		X
3. Visually inspect all welds.		X
4. Inspect details of metal framing including framing layout, member sizes, bracing, bridging and bearing.		X

2. Delete without substitution:

~~**1707.4 Cold-formed steel framing.** Periodic special inspections is required during welding operations of elements of the seismic-force-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the seismic-force-resisting system, including struts, braces, and hold-downs.~~

(Renumber subsequent sections)

Reason: NCSEA believes that cold-formed metal framing has become more commonly used for load bearing applications in all seismic design categories and that this type of construction should be subject to Special Inspections in a similar manner to other systems. The new paragraph will make paragraph 1707.4 redundant therefore it is proposed that 1707.4 be deleted.

The code change proposal will increase the cost of construction by a modest amount as it will require some inspection of cold formed metal framing that is not presently required.

Cost Impact: The code change proposal will increase the cost of construction by a modest amount as it will require some inspection of cold formed metal framing that is not presently required.

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

S115–07/08
1704.3.4 (New), 1704.6.2 (New)

Proponent: Edwin Huston, National Council of Structural Engineers Association (NCSEA), representing NCSEA Code Advisory Committee – General Engineering Subcommittee

Add new text as follows:

1704.3.4 Cold-formed steel trusses spanning 60 feet or greater. Where a cold formed steel truss clear span is 60 feet (18288mm) or greater, the special inspector shall verify that the temporary installation restraint/bracing and the permanent individual truss member restraint/bracing are installed in accordance with the approved truss submittal package.

1704.6.2 Metal-plate connected wood trusses spanning 60 feet or greater. Where a truss clear span is 60 feet (18288 mm) or greater, the special inspector shall verify that the temporary installation restraint/bracing and the permanent individual truss member restraint/bracing are installed in accordance with the approved permit drawings.

Reason: This language is needed in order coordinate with Section 2210.3 (Cold Formed Steel Trusses) and Section 2303.4. (Wood Trusses) for criteria needed for long span truss conditions.

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

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

S116–07/08
Table 1704.4

Proponent: Philip Brazil, PE, SE, Reid Middleton, Inc., representing himself

Revise table as follows:

TABLE 1704.4
REQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD [a]	IBC REFERENCE
3. Inspect ~~bolts to be installed~~ anchors in concrete used to transmit structural loads, prior to and during placement of concrete, except where allowable loads ~~have been increased~~ or design strengths are based on design values without special inspection.	X	—	—	1911.5

(Portions of table not shown remain unchanged)

Reason: The purpose of this proposal is to expand the special inspection of anchors in concrete construction to include ones whose design values are determined other than by use of Table 1911.2. There are numerous types of anchors used for the connection of (typically) wood and steel construction to concrete. They can be cast-in-place or post-installed. The predominant example of cast-in-place installation is an anchor rod complying with ASTM F 1554 but materials complying with other ASTM standards can also be used. Welded head studs and deformed bar anchors are also commonly used. Prominent examples of post-installed anchors are expansion anchors, adhesive anchors, undercut anchors and screw anchors, but these are typically qualified for use through nationally recognized evaluation services (i.e., ICC-ES) and the evaluation reports detailing their qualified use typically specify special inspection requirements. In these cases, the requirements of Chapter 17 for special inspection would not apply unless the evaluation report specifically references Chapter 17 for special inspection requirements.

The design and installation of some anchors can be complex but the designer is rewarded with high-capacity connections that provide reliable and demonstrated levels of performance. This expected performance, however, will not occur unless the installation follows applicable specifications, which may also include the manufacturer's recommendations. Verification that the installation is done in accordance with applicable specifications and recommendations warrants higher levels of quality assurance than for anchors with less complex demands on design and installation. In the IBC, higher levels of quality assurance are provided through special inspection.

One would expect the provisions for special inspection in the IBC to be consistent with this and require special inspection for all anchors in concrete construction except those with less complex demands on their design and installation, but this is not the case. According to Item #3 of Table 1704.4, special inspection is required for bolts where allowable loads have been increased. A reference to Section 1911.5 is included. Section 1911.5 permits a 100-percent increase in the allowable tension values of Table 1911.2 for anchors where special inspection is provided. This means that special inspection is not required for anchors whose design values are derived directly from Table 1911.2 without any increases. Table 1911.2 provides allowable service loads for embedded bolts, referred to as headed anchors in the charging language of Section 1911.2. For all other cases of anchors used for the support of structural loads in concrete construction, including all applications of the procedures for strength design in Section 1912, which specifies compliance with Appendix D of ACI 318, special inspection is not required.

The purpose for this proposal seeks to correct what is judged to be an oversight by requiring special inspection for anchors in concrete used to transmit structural loads except where allowable loads or design strengths are based on design values without special inspection. The phrase "to be installed" is deleted because it is judged to be redundant given the current language specifying inspection "prior to and during placement of concrete." Specifying "design strengths" as well as "allowable loads" incorporates the terminology used for strength design along with allowable stress design. Replacing "bolts" with "anchors" provides a more commonly accepted term.

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

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

S117–07/08
Table 1704.4, 1912.2 (New)

Proponent: Randall Shackelford, P.E., Simpson Strong-Tie, Co., Inc. representing himself

1. Revise table as follows:

TABLE 1704.4
REQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARDâ	IBC REFERENCE
3. Where allowable loads have been increased or design strengths have not been decreased, inspect headed bolts, headed studs, and hooked bolts to be installed in concrete prior to and during placement of concrete ~~where allowable loads have been increased~~ and inspect expansion and undercut anchors during installation in hardened concrete.	X	—	ACI 318: Appendix D	1911.5, 1912.2

(Portions of table not shown remain unchanged)

2. Add new text as follows:

1912.2 Strength reduction for no special inspection. Where special inspection is not provided for the installation of anchors designed in accordance with this section, a 50-percent decrease in the tension design strength shall be taken. No decrease in shear design strength is required.

Reason: The purposes of the proposed code changes are to:
1) Utilize consistent language when referring to anchors, bolts, studs, etc between Table 1704.4 and IBC Sections 1911.1 and 1912.1.
2) Clarify that design strengths calculated under IBC Section 1912 presume that special inspection is provided.
3) Allow the design professional to eliminate the special inspection requirement provided that the design tension strengths calculated under IBC Section 1912 are decreased by 50 percent.

Justification (Reference the numbers above):
1) Use of inconsistent names for anchors between the code sections causes confusion and can lead users of the code to believe that some types of cast-in-place and post-installed anchors require special inspection while other types do not. The intent of the code is that the requirements for special inspection apply uniformly to all types of cast in place anchors and post-installed anchors, regardless of name.
2) The design strengths calculated under IBC Section 1912 (i.e. ACI 318 Appendix D) are based on the 5% fractile strengths of cast-in-place and post-installed anchors in concrete from research, theory, and testing. Unlike anchors in IBC Section 1911, no reductions have been pre-applied to the design strengths calculated in accordance with ACI 318 Appendix D to account for the removal of special inspection.
3) IBC Section 1911.5 allows design professionals the option of requiring or not requiring special inspection for anchors. If special inspection is not provided, the allowable tension load for anchors designed under Section 1911 is effectively reduced by 50 percent. The proposal permits design professionals to similarly eliminate the requirement for special inspection for anchors designed under Section 1912 by reducing design strength by 50 percent.

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

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

S118–07/08
1704.5, Table 1704.5.1, Table 1704.5.3, 1708.1 through Table 1708.1.4

Proponent: Jason Thompson, Concrete Masonry Association, representing Masonry Alliance for Codes and Standards

1. Revise as follows:

1704.5 Masonry construction. Masonry construction shall be inspected and ~~evaluated~~ verified 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 designed by Section 2109, 2110 or Chapter 14, respectively, or by Chapter 5, 7 or 6 of ACI 530/ASCE 5/TMS 402, 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.

TABLE 1704.5.1
~~LEVEL 1 SPECIAL INSPECTION~~
LEVEL 1 REQUIRED VERIFICATION AND INSPECTION OF MASONRY CONSTRUCTION

	FREQUENCY OF INSPECTION		REFERENCE FOR CRITERIA		
VERIFICATION AND INSPECTION ~~TASK~~	**~~Continuous during task listed~~ CONTINUOUS**	**~~Periodically during task listed~~ PERIODIC**	**IBC ~~Section~~ SECTION**	**ACI 530/ASCE 5/TMS 402[a]**	**ACI 530.1/ASCE 6/TMS 602[a]**
1. Verify compliance with the approved submittals	—	X	—	—	Art. 1.5
2. Verification of f'_m and f'_{AAC} prior to construction except where specifically exempted by this code	—	X	—	—	Art. 1.4B
3. Verification of slump flow and VSI as delivered to the site for self-consolidating grout	X	—	—	—	Art 1.5B.1.b.3
~~1~~4. As masonry construction begins, the following shall be verified to ensure compliance:					
a. Proportions of site-prepared mortar.	—	X	—	—	Art. 2.6A
b. Construction of mortar joints.	—	X	—	—	Art. 3.3B
c. Location of reinforcement, connectors, prestressing tendons and anchorages.	—	X	—	—	Art. 3.4, 3.6A
d. Prestressing technique.	—	X	—	—	Art. 3.6B
e. Grade and size of prestressing tendons and anchorages.	—	X	—	—	Art. 2.4B, 2.4H
~~2~~5. ~~The~~ During construction the inspection program shall verify:					
a. Size and location of structural elements.	—	X	—	—	Art. 3.3G
b. Type, size and location of anchors, including other details of anchorage of masonry to structural members, frames or other construction.	—	X	—	Sec. 1.2.2(e), 2.1.4, 3.1.6	—
c. Specified size, grade and type of reinforcement, anchor bolts, prestressing tendons and anchorages.	—	X	—	Sec. 1.13	Art. 2.4, 3.4
d. Welding of reinforcing bars	X	—	—	Sec. 2.1.10.7.2, 3.3.3.4(b)	—
e. Preparation, construction and protection of masonry during cold weather (temperature below 40°F) or hot weather (temperature above 90°F).	—	X	Sec 2104.3, 2104.4	—	Art. 1.8C, 1.8D
f. Application and measurement of prestressing force.	X ~~—~~	— ~~X~~	—	—	Art. 3.6B

VERIFICATION AND INSPECTION ~~TASK~~	FREQUENCY OF INSPECTION		REFERENCE FOR CRITERIA		
	~~Continuous during task listed~~ CONTINUOUS	~~Periodically during task listed~~ PERIODIC	IBC ~~Section~~ SECTION	ACI 530/ASCE 5/TMS 402[a]	ACI 530.1/ASCE 6/TMS 602[a]
~~3~~6. Prior to grouting, the following shall be verified to ensure compliance:					
a. Grout space is clean	—	X	—	—	Art. 3.2D
b. Placement of reinforcement and connectors~~,~~ and prestressing tendons and anchorages.	—	X	—	Sec. 1.13	Art. 3.4
c. Proportions of site-prepared grout and prestressing grout for bonded tendons.	—	X	—	—	Art. 2.6B
d. Construction of mortar joints.	—	X	—	—	Art. 3.3B
~~4~~7. Grout placement shall be verified to ensure compliance ~~with code and construction document provisions.~~	X	—	—	—	Art 3.5
a. Grouting of prestressing bonded tendons.	X	—	—	—	Art. 3.6C
~~5~~8. Preparation of any required grout specimens, mortar specimens and/or prisms shall be observed.	—~~X~~	X~~—~~	Sec. 2105.2.2, 2105.3	—	Art. 1.4
~~6. Compliance with required inspection provisions of the construction documents and the approved submittals shall be verified~~	~~—~~	~~X~~	~~—~~	~~—~~	~~Art. 1.5~~

For SI: °C = (°F- 32)/1.8.

a. The specific standards referenced are those listed in Chapter 35.

TABLE 1704.5.~~3~~2
~~LEVEL 2 SPECIAL INSPECTION~~
LEVEL 2 REQUIRED VERIFICATION AND INSPECTION OF MASONRY CONSTRUCTION

VERIFICATION AND INSPECTION ~~TASK~~	~~Continuous during task listed~~ CONTINUOUS	~~Periodically during task listed~~ PERIODIC	REFERENCE FOR CRITERIA		
			IBC ~~section~~ SECTION	TMS 402/ACI 530/ ASCE 5[a]	TMS 602/ACI 530.1/ ASCE 6[a]
1. Verify compliance with the approved submittals	—	X	—	—	Art. 1.5
2. Verification of f'_m and f'_{AAC} prior to construction and for every 5000 sq.ft. during construction.	—	X	—	—	Art. 1.4B
3. Verification of proportions of materials in premixed or preblended mortar and grout as delivered to the site.	—	X	—	—	Art. 1.5B
4 Verification of slump flow and VSI as delivered to the site for self-consolidating grout	X	—	—	—	Art. 1.5B.1.b.3
~~1~~5 ~~From the beginning of masonry construction, the~~ The following shall be verified to ensure compliance:					
a. Proportions of site-prepared mortar, grout and prestressing grout for bonded tendons.	—	X	—	—	Art. 2.6A
b. Placement of masonry units and construction of mortar joints.	—	X	—	—	Art. 3.3B

VERIFICATION AND INSPECTION ~~TASK~~	~~Continuous during task listed~~ CONTINUOUS	~~Periodically during task listed~~ PERIODIC	REFERENCE FOR CRITERIA		
			IBC ~~section~~ SECTION	TMS 402/ACI 530/ ASCE 5 [a]	TMS 602/ACI 530.1/ ASCE 6 [a]
c. Placement of reinforcement, connectors and prestressing tendons and anchorages.	X~~—~~	—~~X~~	—	Sec. 1.13	Art. 3.4, 3.6A
d. Grout space prior to grouting.	X	—	—	—	Art. 3.2D
e. Placement of grout.	X	—	—	—	Art. 3.5
f. Placement of prestressing grout.	X	—	—	—	Art. 3.6C
~~2. The inspection program shall verify:~~					
~~a~~g. Size and location of structural elements	—	X	—	—	Art. 3.3G
~~b~~h. Type, size and location of anchors, including other details of anchorage of masonry to structural members, frames or other construction.	X	—	—	Sec. 1.2.2(e), 2.1.4, 3.1.6	—
~~c~~i. Specified size, grade and type of reinforcement, anchor bolts, prestressing tendons and anchorages.	—	X	—	Sec. 1.13	Art. 2.4, 3.4
~~d~~j. Welding of reinforcing bars.	X	—	—	Sec. 2.1.10.7.2, 3.3.3.4(b)	—
~~e~~k. Preparation, construction and protection of masonry during cold weather (temperature below 40°F) or hot weather (temperature above 90°F).	—	X	Sec 2104.3, 2104.4	—	Art. 1.8C, 1.8D
~~f~~l. Application and measurement of prestressing force.	X	—	—	—	Art. 3.6B
~~3~~6 Preparation of any required grout specimens, mortar specimens and/or prisms shall be observed.	X	—	Sec. 2105.2.2, 2105.3	—	Art. 1.4
~~4. Compliance with required inspection provisions of the construction documents and the approved submittals shall be verified~~	~~—~~	~~X~~	~~—~~	~~—~~	~~Art. 1.5~~

For SI: °C = (°F - 32)/1.8.

a. The specific standards referenced are those listed in Chapter 35.

2. Delete without substitution as follows:

~~**1708.1 (Supp) Special inspections for wind requirements.** Special inspections itemized in Sections 1708.2 through 1708.4, unless exempted by the exceptions to Section 1704.1, are required for buildings and structures constructed in the following areas:~~

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

~~**1708.1.1 Empirically designed masonry and glass unit masonry in Occupancy Category I, II or III.** For masonry designed by Section 2109 or 2110 or by Chapter 5 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, 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 designed by Section 2109 or 2110 or by Chapter 5 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 the requirements of Table 1708.1.2.~~

~~TABLE 1708.1.2~~
~~LEVEL 1 QUALITY ASSURANCE~~

~~MINIMUM TESTS AND SUBMITTALS~~
~~Certificates of compliance used in masonry construction.~~ ~~Verification of f'_m and f'_{AAC} prior to construction, except where specifically exempted by this code.~~

~~**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 2107 or 2108 or by chapters other than Chapter 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 2107 or 2108 or by chapters other than Chapter 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.~~

~~TABLE 1708.1.4~~
~~LEVEL 2 QUALITY ASSURANCE~~

~~MINIMUM TESTS AND SUBMITTALS~~
~~Certificates of compliance used in masonry construction.~~ ~~Verification of f'_m and f'_{AAC} prior to construction and every 5,000 square feet during construction.~~ ~~Verification of proportions of materials in mortar and grout as delivered to the site.~~

(Renumber subsequent sections)

Reason: The revisions proposed in this code change reflect editorial and substantive revisions incorporated into the 2008 edition of the Building Code Requirements for Masonry Structures (TMS 402/ACI 530/ASCE 5), commonly referred to as the Masonry Standard Joint Committee (MSJC) Code. This code change proposal is one of several to harmonize the design and construction requirements for masonry within the IBC with those in the reference standard. A complete list of revisions incorporated into the reference standard is available for download at www.masonrystandards.org.

Specific revisions proposed above include:

IBC Section T1704.5.1 and 1704.5.3 have been revised to comply with the changes in the 2008 MSJC and to conform to the format of the tables for steel and concrete. In Section 1708 we are proposing to delete the Seismic Testing Provisions for masonry. 1708 doesn't indicate which SDC's require these tests. As such they are required in any SDC and are routinely preformed in all SDC's. This has caused significant confusion to many practitioners who don't know when to require this testing. They think it is only required in zones of moderate or high seismicity. Moving these requirements to T1704.5.1 and 1704.5.3 allows them to be eliminated from 1708 and follows the model of concrete.

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

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

S119–07/08
Table 1704.5.1, Table 1704.5.3

Proponent: D. Kirk Harman, PE, SE, The Harman Group, Inc., representing The National Council of Structural Engineers Associations (NCSEA) - Code Advisory Committee, Quality Assurance and Special Inspection Subcommittee

Revise tables as follows:

TABLE 1704.5.1
~~LEVEL 1 SPECIAL INSPECTION~~
<u>REQUIRED VERIFICATION AND INSPECTION OF MASONRY – LEVEL 1</u>

(Portions of table not shown remain unchanged)

TABLE 1704.5.3
~~LEVEL 2 SPECIAL INSPECTION~~
REQUIRED VERIFICATION AND INSPECTION OF MASONRY – LEVEL 2

(Portions of table not shown remain unchanged)

Reason: This change is to make the titles of these tables consistent with the other similar tables.

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

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

S120–07/08

1704.6, 1704.6.1, Table 1704.6 (New)

Proponent: D. Kirk Harman, PE, SE, The Harman Group, Inc., representing The NCSEA Code Advisory Committee, Quality Assurance and Special Inspection Subcommittee

Revise as follows:

1704.6 Wood construction. ~~Special inspections of the fabrication process of prefabricated wood structural elements and assemblies shall be in accordance with Section 1704.2. Special inspections of site-built assemblies shall be in accordance with this section.~~ Special inspections for wood construction shall be as required by Table 1704.6.

~~**1704.6.1 High-load diaphragms.** High-load diaphragms designed in accordance with Table 2306.3.2 shall be installed with special inspections as indicated in Section 1704.1. The special inspector shall inspect the wood structural panel sheathing to ascertain whether it is of the grade and thickness shown on the approved building plans. Additionally, the special inspector must verify the nominal size of framing members at adjoining panel edges, the nail or staple diameter and length, the number of fastener lines and that the spacing between fasteners in each line and at edge margins agrees with the approved building plans.~~

TABLE 1704.6
REQUIRED VERIFICATION AND INSPECTION OF WOOD CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC
1. Inspect grade stamp on framing lumber, plywood and OSB panels.		X
2. Inspect wood connections including nailing, bolting, anchor bolts, tie downs, beam hangers and framing anchors.		X
3. Inspect details of wood framing including framing, member sizes, blocking, bridging and bearing.		X
4. Inspect diaphragms and shearwalls for proper panel thickness and fastener pattern.	X	
5. Inspect prefabricated wood trusses for proper fabrication, installation and bracing.		X

Reason: The emphasis of the existing inspection requirements is on shop inspection of fabricated wood items rather than the field assembly of wood framing. Quality control problems with wood construction are most pronounced in the field work rather than in prefabricated components. The proposed provisions focus on the areas of wood construction that would benefit most from more comprehensive inspections.

The code change proposal will increase the cost of construction by a modest amount as it will require some inspection of wood buildings that is not presently required. This Code does not apply to single family home wood construction so it would not result in any increase in construction of those buildings.

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

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

S121–07/08
1704.7, 1704.8, 1704.9

Proponent: Philip Brazil, PE, SE, Reid Middleton, Inc., representing himself

Revise as follows:

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

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

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

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

Reason: The changes are proposed for consistency with the provisions of Section 106.1 on submittal documents and Sections 106.2 and 1603 on construction documents. Please refer to the 2007 IBC Supplement for the current provisions in Sections 106.1 and 106.2, which were revised by Proposal G222-06/07-AM (Part I).

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

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

S122–07/08
1704.7, Table 1704.7, 1704.8, Table 1704.8, 1704.9, Table 1704.9, 1704.10, 1707.5, 1803.5

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

1. Revise as follows:

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

> **Exception:** ~~Special inspection is not required during placement of controlled fill having a total depth of 12 inches (305 mm) or less.~~ <u>Where Section 1803 does not require reporting of materials and procedures for fill placement, the special inspector shall verify that fill is compacted to a minimum of 90 percent Modified Proctor in accordance with ASTM D 1557.</u>

TABLE 1704.7
REQUIRED VERIFICATION AND INSPECTION OF SOILS

VERIFICATION AND INSPECTION TASK	CONTINUOUS DURING TASK LISTED	PERIODICALLY DURING TASK LISTED
1. Verify materials below ~~footings~~ shallow foundations are adequate to achieve the design bearing capacity.	—	X
2. Verify excavations are extended to proper depth and have reached proper material.	—	X
3. Perform classification and testing of ~~controlled~~ compacted fill materials.	—	X
4. Verify use of proper materials, densities and lift thicknesses during placement and compaction of ~~controlled~~ compacted fill.	X	—
5. Prior to placement of ~~controlled~~ compacted fill, observe subgrade and verify that site has been prepared properly.	—	X

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

TABLE 1704.8
REQUIRED VERIFICATION AND INSPECTION OF ~~PILE~~ DRIVEN DEEP FOUNDATION ELEMENTS

VERIFICATION AND INSPECTION TASK	CONTINUOUS DURING TASK LISTED	PERIODICALLY DURING TASK LISTED
1. Verify ~~pile~~ element materials, sizes and lengths comply with the requirements.	X	—
2. Determine capacities of test ~~piles~~ elements and conduct additional load tests, as required.	X	—
3. Observe driving operations and maintain complete and accurate records for each ~~pile~~ element.	X	—
4. Verify placement locations and plumbness, confirm type and size of hammer, record number of blows per foot of penetration, determine required penetrations to achieve design capacity, record tip and butt elevations and document any ~~pile~~ damage to foundation elements.	X	—
5. For steel ~~piles~~ elements, perform additional inspections in accordance with Section 1704.3.	—	—
6. For concrete ~~piles~~ elements and concrete-filled ~~piles~~ elements, perform additional inspections in accordance with Section 1704.4.	—	—
7. For specialty ~~piles~~ elements, perform additional inspections as determined by the registered design professional in responsible charge.	—	—
~~8. For augered uncased piles and caisson piles, perform inspections in accordance with Section 1704.9.~~	—	—

1704.9 (Supp) ~~Pier~~ Cast-in-place deep foundations. Special inspections shall be performed during installation and testing of ~~pier~~ cast-in-place deep foundations elements as required by Table 1704.9. The approved ~~soils~~ geotechnical report~~, required by Section 1802.2,~~ and the documents prepared by the registered design professional shall be used to determine compliance.

TABLE 1704.9
REQUIRED VERIFICATION AND INSPECTION OF ~~PIER~~ CAST-IN-PLACE DEEP FOUNDATION ELEMENTS

VERIFICATION AND INSPECTION TASK	CONTINUOUS DURING TASK LISTED	PERIODICALLY DURING TASK LISTED
1. Observe drilling operations and maintain complete and accurate records for each element ~~pier~~.	X	—
2. Verify placement locations and plumbness, confirm element ~~pier~~ diameters, bell diameters (if applicable), lengths, embedment into bedrock (if applicable) and adequate end bearing strata capacity.	X	—
3. For concrete elements ~~piers~~, perform additional inspections in accordance with Section 1704.4.	—	—
4. ~~For masonry piers, perform additional inspections in accordance with Section 1704.5.~~	—	—

1704.10 Vertical masonry foundation elements. Special inspection shall be performed in accordance with Section 1704.5 for vertical masonry foundation elements.

(Renumber subsequent sections)

2. Delete without substitution:

~~**1707.5 Pier foundations.** Special inspection is required for pier foundations for buildings assigned to Seismic Design Category C, D, E or F in accordance with Section 1613. Periodic special inspection is required during placement of reinforcement and continuous special inspection is required during placement of the concrete.~~

3. Revise as follows:

1803.5 Compacted fill material. Where footings will bear on compacted fill material, the compacted fill shall comply with the provisions of an approved report, which shall contain the following:

1. Specifications for the preparation of the site prior to placement of compacted fill material.
2. Specifications for material to be used as compacted fill.
3. Test method to be used to determine the maximum dry density and optimum moisture content of the material to be used as compacted fill.
4. Maximum allowable thickness of each lift of compacted fill material.
5. Field test method for determining the in-place dry density of the compacted fill.
6. Minimum acceptable in-place dry density expressed as a percentage of the maximum dry density determined in accordance with Item 3.
7. Number and frequency of field tests required to determine compliance with Item 6.

Exception: Compacted fill material ~~less than~~ 12 inches (305 mm) in depth or less need not comply with an approved report, provided it has been compacted to a minimum of 90 percent Modified Proctor in accordance with ASTM D 1557. The compaction shall be verified by ~~a qualified inspector approved by the building official~~ special inspection in accordance with Section 1704.7.

Reason: Code clarification and update.

Removes conflict between Section 1704.7 (no special inspection) and Section 1803.5 (special inspection required). Corrects “controlled” as “compacted” in Table 1704.7.

Removes Section 1707.5, which is unnecessary. Section 1704.9 sets forth special inspection requirements for piers. Item 3 in Table 1704.9 requires compliance with Section 1704.4. For ALL seismic design categories Table 1704.4 requires periodic special inspection of reinforcement and continuous special inspection of concrete placement.

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

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

S123–07/08
Table 1704.9

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

Revise table as follows:

TABLE 1704.9
REQUIRED VERIFICATION AND INSPECTION OF PIER FOUNDATIONS

VERIFICATION AND INSPECTION TASK	CONTINUOUS DURING TASK LISTED	PERIODICALLY DURING TASK LISTED
1. Observe drilling operations and maintain complete and accurate records for each pier.	X	—
2. Verify placement locations and plumbness, confirm pier diameters, bell diameters (if applicable), lengths, embedment into bedrock (if applicable) and adequate end bearing strata capacity. <u>Record concrete or grout volumes.</u>	X	—
3. For concrete piers, perform additional inspections in accordance with Section 1704.4.	—	—
4. For masonry piers, perform additional inspections in accordance with Section 1704.5.	—	—

Reason: Code update. Adds an item to the required special inspections for piers to reflect typical practice. Continuous special inspection is already required during placement of concrete (Table 1704.4, item 6). Recorded volumes of concrete or grout placed are often the first indicator of potentially significant problems with the construction.

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

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

S124–07/08
1704.15 (New), Chapter 35 (New)

Proponent: Gilbert Gonzales, Murray City Corporation, representing Utah Chapter ICC

THIS CODE CHANGE PROPOSAL IS ON THE AGENDA OF THE IBC FIRE SAFETY CODE DEVELOPMENT COMMITTEE. SEE THE HEARING ORDER FOR THE IBC FIRE SAFETY CODE DEVELOPMENT COMMITTEE.

1. Add new text as follows:

<u>**1704.15 Fire-resistant penetration and joints.** Special inspection for through penetrations, membrane penetrations, joints and perimeter fire barrier systems of the types specified in Sections 712.3.1.2,712.4.1.2,713.3 and 713.4 respectively shall be in accordance with Section 1704.15.1 or 1704.15.2. Special inspections shall be based on fire-resistance rated design or system as designated in the approved construction documents.</u>

<u>**1704.15.1 Fire-resistant penetrations.** Inspections of fire-resistant penetrations systems of the types specified in Sections 712.3.1.2 and 712.4.1.2 shall be conducted by an inspection agency in accordance with ASTM E 2174.</u>

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

1. <u>Buildings less than 4 stories above grade plane or,</u>
2. <u>Installation by UL or FM certified contractors.</u>

<u>**1704.15.2 Fire-resistive joints.** Inspection of joints of the types specified in Sections 713.3 and 713.4 shall be conducted by an approved inspection agency in accordance with ASTM E 2393.</u>

Exceptions:

1. Buildings less than 4 stories above grade plane, or
2. Installation by UL or FM certified contractors.

2. Add standards to Chapter 35 as follows:

ASTM International

E 2174-04 Standard Practice for On-Site Inspection of Installed Fire Stops.

E 2393-04 Standard Practice for On-Site Inspection of Installed Fire Resistive Joint Systems and Perimeter Fire Barriers.

Reason: Installation of firestop systems is often installed by trades and or contractors who do not have the extensive knowledge or training needed to ensure that these critical life safety systems are installed correctly. At the same time, firestop and joint systems designs and materials are increasing in number and sophistication. Adding ASTM standard E2174-04 & ASTM E2393-04 outlines the inspection procedures for firestop inspectors. The addition of these new standards and certified contractors and or special inspection would provide and identify a means for building departments to have effective tools to instruct either their own staff or third party inspection agencies on good methodologies for inspection of these important systems. Requiring special inspection or certified contractors to perform the work will result in a proper installation.

This may or may not increase the cost of construction, depending on weather a UL of FM certified contractor is hired or requiring special inspection.

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

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM E 2174 and E 2393, for compliance with ICC criteria for referenced standards given in Section 3.6 of Council Policy #CP 28 will be posted on the ICC website on or before January 15, 2008.

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

S125–07/08

1705.3, 1707.1, 1708.2

Proponent: Bonnie Manley, American Iron and Steel Institute, representing American Institute of Steel Construction

1. Revise as follows:

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

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C, D, E or F, in accordance with Section 1613.
2. Designated seismic systems in structures assigned to Seismic Design Category D, E or F.
3. The following additional systems and components in structures assigned to Seismic Design Category C:
   3.1. Heating, ventilating and air-conditioning (HVAC) ductwork containing hazardous materials and anchorage of such ductwork.
   3.2. Piping systems and mechanical units containing flammable, combustible or highly toxic materials.
   3.3. Anchorage of electrical equipment used for emergency or standby power systems.
4. The following additional systems and components in structures assigned to Seismic Design Category D:
   4.1. Systems required for Seismic Design Category C.
   4.2. Exterior wall panels and their anchorage.
   4.3. Suspended ceiling systems and their anchorage.
   4.4. Access floors and their anchorage.
   4.5. Steel storage racks and their anchorage, where the importance factor is equal to 1.5 in accordance with Section 15.5.3 of ASCE 7.
5. The following additional systems and components in structures assigned to Seismic Design Category E or F:
   5.1. Systems required for Seismic Design Categories C and D.
   5.2. Electrical equipment.

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

   1. The structure consists of light-frame construction; the design spectral response acceleration at short periods, S_{DS}, is determined in Section 1613.5.4, does not exceed 0.5g; and the height of the structure does not exceed 35 feet (10 668 mm) above grade plane; or

2. The structure is constructed using a reinforced masonry structural system or reinforced concrete structural system; the design spectral response acceleration at short periods, S_{DS}, as determined in Section 1613.5.4, does not exceed 0.5g, and the height of the structure does not exceed 25 feet (7620 mm) above grade plane; or
3. Detached one- or two-family dwellings not exceeding two stories above grade plane, provided the structure does not have any of the following plan or vertical irregularities in accordance with Section 12.3.2 of ASCE 7:
   3.1. Torsional irregularity.
   3.2. Nonparallel systems.
   3.3. Stiffness irregularity extreme soft story and soft story.
   3.4. Discontinuity in capacity weak story.
4. Steel systems in structures that are assigned to Seismic Design Category C that are not specifically detailed for seismic resistance, with a response modification coefficient, R, of 3 or less, excluding cantilever column systems.

1707.1 Special inspections for seismic resistance. Special inspections itemized in Sections 1707.2 through 1707.10, unless exempted by the exceptions of Section 1704.1 or 1705.3, are required for the following:

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C, D, E or F, as determined in Section 1613.
2. Designated seismic systems in structures assigned to Seismic Design Category D, E or F.
3. Architectural, mechanical and electrical components in structures assigned to Seismic Design Category C, D, E or F that are required in Sections 1707.7 and 1707.8.

1708.2 (Supp) Testing and qualification for seismic resistance. The testing and qualification specified in Sections 1708.3 through 1708.6, unless exempted from special inspections by the exceptions of Section 1704.1 and 1705.3, are required as follows:

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C, D, E or F, as determined in Section 1613 shall meet the requirements of Sections 1708.3 and 1708.4, as applicable.
2. Designated seismic systems in structures assigned to Seismic Design Category D, E or F in Section 13.2.2 of ASCE 7 shall meet the requirements of Section 1708.5.
3. Architectural, mechanical and electrical components in structures assigned to Seismic Design Category C, D, E or F with an I_p = 1.0 shall be permitted to be seismically qualified by meeting the requirements of Section 1708.5.
4. The seismic isolation system in seismically isolated structures shall meet the testing requirements of Section 1708.6.

Reason: In Sections 1705.3, 1707.1 and 1708.2, a general reference to SDC C is included for seismic force resisting systems in order to recognize that many structural systems require special detailing because of their seismic response characteristics. However, this general requirement does not reflect the unique response characteristics of some steel buildings. ASCE 7-05, Table 12.2-1 assigns steel building structures a response modification coefficient of R = 3, if they are built in SDC A, B or C as a “steel system not specifically detailed for seismic resistance, excluding cantilever column systems.” For these building systems, the assigned seismic response coefficient reflects their inherent ductility. As a consequence, these structures are permitted to be constructed using only AISC 360 (that is, not detailed in accordance with the additional provisions of AISC 341). As these construction details and connections are the same as would be used in typical steel buildings following AISC 360, no additional inspection or testing should be required beyond that applied to typical steel buildings. The modifications to IBC Sections 1705.3, 1707.1 and 1708.2 reflect this concept.

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

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

S126–07/08
1705.4.2, 1708.4

Proponent: Michael D. Fischer, The Kellen Company, representing ARMA: The Asphalt Roofing Manufacturer’s Association

Revise as follows:

1705.4.2 Detailed requirements. The statement of special inspections shall include at least the following systems and components:

1. Roof deck connections. ~~cladding and roof framing connections.~~
2. Roof framing connections.
~~2~~3. Wall connections to roof and floor diaphragms and framing.
~~3~~4. Roof and floor diaphragm systems, including collectors, drag struts and boundary elements.
~~4~~5. Vertical windforce-resisting systems, including braced frames, moment frames and shear walls.
~~5~~6. Windforce-resisting system connections to the foundation.
~~6~~7. Fabrication and installation of systems or components required to meet the impact-resistance requirements of Section 1609.1.2.

Exception: Fabrication of manufactured systems or components that have a label indicating compliance with the wind-load and impact-resistance requirements of this code.

1708.4 (Supp) Wind-resisting components. Periodic special inspection is required for the following systems and components:

1. Roof deck connections. ~~cladding.~~
2. Roof framing connections.
~~2~~ 3. Wall cladding.

Exception: Fabrication of manufactured systems or components that have a label indicating compliance with the wind-load and impact-resistance requirements of this code.

Reason: The addition of the requirement for special inspections for roof cladding in the last cycle is inconsistent with the existing "statement of special inspections" provisions found in 1705.4.2, most notably the exception for labeled components. This proposal is necessary to clarify that the connections between the roof covering and the roof framing are subject to special inspections, not the roof covering itself. By removing "roof cladding" from the IBC text and substituting it with "roof deck connections", and adding the requirement for "roof framing", the proposal is more easily interpreted. The exception for labeled products precludes the need for a special inspector to perform plant visits for roof coverings. This proposal solves the definition problems between ASCE-7 and the code.

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

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

S127–07/08
1706.1

Proponent: D. Kirk Harman, PE, SE, The Harman Group, Inc., representing The National Council of Structural Engineers Associations (NCSEA) Code Advisory Committee, Quality Assurance and Special Inspection Subcommittee

Delete without substitution:

~~SECTION 1706~~
~~CONTRACTOR RESPONSIBILITY~~

~~**1706.1 Contractor responsibility.** Each contractor responsible for the construction of a main wind- or seismic-force-resisting system, designated seismic system or a wind- or seismic-resisting component listed in the statement of special inspections shall submit a written statement of responsibility to the building official and the owner prior to the commencement of work on the system or component. The contractor's statement of responsibility shall contain the following: 1. Acknowledgment of awareness of the special requirements contained in the statement of special inspections; 2. Acknowledgment that control will be exercised to obtain conformance with the construction documents approved by the building official; 3. Procedures for exercising control within the contractor's organization, the method and frequency of reporting and the distribution of the reports; and 4. Identification and qualifications of the person(s) exercising such control and their position(s) in the organization.~~

(Renumber subsequent sections)

Reason: This requirement was originally to go along with the Quality Assurance Plan, which has now been deleted from the code. The requirement is unenforceable, is not followed typically by contractors and is often ignored by jurisdictions.

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

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

S128–07/08
1707.2

Proponent: Bonnie Manley, American Iron and Steel Institute, representing American Institute of Steel Construction

Revise as follows:

1707.2 Structural steel. ~~Continuous special inspection is required for structural welding in accordance with AISC 341.~~ Special inspection for structural steel members shall be in accordance with the quality assurance plan requirements of AISC 341.

> ~~**Exceptions:**~~
>
> 1. ~~Single-pass fillet welds not exceeding 5/16 inch (7.9 mm) in size.~~
> 2. ~~Floor and roof deck welding.~~

Reason: Between the 2003 and 2006 editions of the IBC, the terminology in Section 1705 was changed from "quality assurance plan" to "statement of special inspection". Unfortunately, the change in terminology was not picked up in time for the 2005 edition of AISC 341, *Seismic Provisions for Structural Steel Buildings*. In order to ensure that there is no confusion, a direct reference to the quality assurance plan requirements in AISC 341 is recommended for structural steel members. Part 1, Appendix Q of the 2005 AISC Seismic Provisions provides a comprehensive Quality Assurance Plan including Tables of QC and QA inspection requirements. For structures designed according to AISC 341, it is required that QC and QA be provided as specified in that section.

Earlier versions of AISC 341 did not specifically address the frequency of welding inspection. However, the Quality Assurance Plan in Appendix Q of AISC 341-05 now addresses frequency of inspection. The first exception for single pass fillet welds is recommended for deletion. Fillet welds are now covered in Appendix Q of AISC 341-05, so the exception is no longer necessary. Also, the second exception for floor and roof deck welding is recommended for deletion. This section requires adequate special inspections for seismic resistance of structural steel only. Section 1704.3 Exception 2.2 already sufficiently addresses the welding of the floor and roof deck in a general manner.

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

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

S129–07/08
1707.4

Proponent: Bonnie Manley, American Iron and Steel Institute, American Iron and Steel Institute

Revise as follows:

1707.4 Cold-formed steel ~~framing~~ and cold-formed steel light frame construction. Periodic special inspection is required during welding operations of elements of the seismic-force-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the seismic-force-resisting system, including struts, braces, and hold-downs.

> **Exception:** Special inspection is not required for cold-formed steel light frame shearwalls, shear panels and diaphragms, including screw attachment, bolting, anchoring and other fastening to other components of the seismic-force-resisting system, where either of the following apply:
>
> 1. The sheathing is gypsum board or fiberboard.
> 2. The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c.).

Reason: The title of the section has been modified editorially to match the terminology used in IBC Section 2209 and Section 2210.

Wood and cold-formed steel light frame construction have similar requirements for their lateral force resisting systems. Therefore, the exception in IBC Section 1707.3 should also apply, with the appropriate adaptation, to cold-formed steel light frame construction. A quick historical review indicated that the 4" spacing for wood construction roughly translates to a minimum capacity of 380lb/ft. Cold-formed steel light frame shear walls, shear panels and diaphragms meeting the above requirements satisfy this minimum capacity, per AISI S213-07, *North American Standard for Cold-Formed Steel Framing – Lateral Design*.

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

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

S130–07/08
1707.7

Proponent: Andy Williams, Alcan Composites USA, Inc., representing himself

Revise as follows:

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

Exceptions:

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

Reason: This code change proposal is merely intended to editorially clarify the exceptions to this section which was revised during the last code development cycle. It is not intended to make any technical changes. It simply utilizes wording within the exceptions that is consistent with the wording in the charging paragraph to which the exceptions are made. Thus, the language in the exceptions will parallel the language in the main paragraph as the exceptions apply to the specific cases indicated. This should make these exceptions much more user friendly and easier to enforce.

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

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

S131–07/08
1707.8

Proponent: Philip Brazil, PE, SE, Reid Middleton, Inc., representing himself

Revise as follows:

1707.8 Mechanical and electrical components. Special inspection for mechanical and electrical equipment shall be as follows:

1. Periodic special inspection is required during the anchorage of electrical equipment for emergency or standby power systems in structures assigned to Seismic Design Category C, D, E or F;
2. Periodic special inspection is required during the installation of anchorage of all other electrical equipment in structures assigned to Seismic Design Category E or F;
3. Periodic special inspection is required during installation of piping ~~systems intended to carry flammable, combustible or highly toxic contents~~ and tubing for the conveyance of hazardous materials and their associated mechanical units in structures assigned to Seismic Design Category C, D, E or F;
4. Periodic special inspection is required during the installation of ~~HVAC~~ ductwork ~~that will contain~~ for the conveyance of hazardous materials and its associated mechanical units in structures assigned to Seismic Design Category C, D, E or F; and
5. Periodic special inspection is required during the installation of vibration isolation systems in structures assigned to Seismic Design Category C, D, E or F where the construction documents require a nominal clearance of 0.25 inches (6.4 mm) or less between the equipment support frame and restraint.

Reason: The purpose of this proposal is to clarify the intent of the requirements for periodic special inspection of mechanical and electrical equipment and to align the terminology used with Section 13.6 of ASCE 7-05 on electrical and mechanical components. The intent of Items #3 and #4 is judged to be a requirement for periodic special inspection during the installation of mechanical systems for the conveyance of materials that, if released, would pose a physical or health hazard to the occupants of the building or structure where the mechanical systems are located.

In Item #3, "piping systems" is changed to "piping and tubing" for consistency with the terminology for distribution systems in Table 13.6-1 of ASCE 7-05. In the same item, "flammable, combustible or highly toxic contents" is changed to "hazardous materials" for consistency with Item #4.

In Item #4, "and their associated mechanical units" is added for consistency with Item #3. In same item, "HVAC" is deleted since it is unlikely that a duct for heating, ventilating and air conditioning (HVAC) will convey hazardous materials and Item #4 is currently limited in scope to such ducts. An example of ductwork conveying hazardous materials is a hazardous exhaust system, which captures and controls hazardous emissions generated from product handling or similar process and conveys the emissions to the outdoors (refer to IMC Section 510.1).

Note that "hazardous materials" are defined in IBC Section 307.2 and include materials that are cryogenic, explosive, oxidizing, pyrophoric, unstable-reactive, water-reactive or corrosive in addition to flammable, combustible or highly toxic.

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

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

S132–07/08
1708.2

Proponent: Gary J. Ehrlich, PE, National Association of Home Builders, representing National Association of Home Builders

Revise as follows:

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

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

Reason: Proposal S44-06/07 in the last code cycle introduced a new section on Special Inspections for Wind Requirements. The inspection requirements for wood framing were copied in their entirety from the existing requirements in Section 1707.3 for seismic resistance. However, the requirement for continuous inspection of field gluing is not warranted for the main wind-force resisting system. Adhesives present a known problem for seismic resistance as they affect the stiffness and energy dissipation of the system under seismic loading. We are not aware of any similar performance problems of adhesives under wind loads, where the load cycles are much longer and the energy level of a high-seismic event is not being imparted to the structure. Thus the continuous inspection requirement for adhesives should be deleted from the wind section.

The primary purpose for adhesives in structural wood framing is to improve the serviceability performance in floor systems and reduce vibration and deflection under standard floor live loads. An additional benefit is that adhesives can used to bond sheathing to supporting members in a roof assembly, providing substantial added resistance to hurricane winds. In fact, several adhesive products (FoamSeal, for example) are being touted as a cost-effective retrofit measure as well as in new construction, and insurance companies in hurricane-prone areas are offering discounts and incentives for use of the products. These mitigation efforts should not be penalized by imposing a costly and onerous special inspection requirement on new construction or substantial remodeling and retrofit work which would offset the incentives for using the adhesives.

NAHB asks for your support of this proposal.

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

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

S133–07/08
1708.3

Proponent: Bonnie Manley, American Iron and Steel Institute, representing American Iron and Steel Institute

Revise as follows:

1708.3 (Supp) Cold-formed steel ~~framing~~ light-frame construction. Periodic special inspection is required during welding operations of elements of the main wind-force-resisting system. Periodic special inspection is required for screw attachment, bolting, anchoring and other fastening of components within the main wind-force-resisting system, including struts, braces, and holddowns.

> **Exception:** Special inspection is not required for cold-formed steel light-frame shearwalls, shear panels and diaphragms, including screw attachment, bolting, anchoring and other fastening to other components of the main wind-force-resisting system, where either of the following apply:

1. The sheathing is gypsum board or fiberboard.
2. The sheathing is wood structural panel or steel sheets on only one side of the shear wall, shear panel or diaphragm assembly and the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c.).

Reason: The title of the section has been modified editorially to match the terminology used in IBC Section 2210.
Wood and cold-formed steel light frame construction have similar requirements for their lateral force resisting systems. Therefore, the exception in IBC Section 1707.3 should also apply, with the appropriate adaptation, to cold-formed steel light frame construction. A quick historical review indicated that the 4" spacing for seismic detailing of wood construction, which is the presumed source of the wood construction exception, roughly translates to a minimum capacity of 380lb/ft. Cold-formed steel light frame shear walls, shear panels and diaphragms meeting the above requirements satisfy this minimum capacity, per AISI S213-07, *North American Standard for Cold-Formed Steel Framing – Lateral Design*.

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

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

S134–07/08
1708.4

Proponent: Michael D. Fischer, The Kellen Company, representing ARMA: The Asphalt Roofing Manufacturer's Association

Revise as follows:

1708.4 (Supp) Wind-resisting components. Periodic special inspection is required for the following systems and components:

1. Roof cladding and roof framing connections.
2. Wall cladding.

Exception: Fabrication of manufactured systems or components that have a label indicating compliance with the wind-load and impact-resistance requirements of this code.

Reason: The addition of the requirement for special inspections for roof cladding in the last ICC cycle is inconsistent with the existing "statement of special inspections" provisions found in 1705.4.2, most notably the exception for labeled components. This proposal is necessary to clarify that the connections between the roof covering and the roof framing are subject to special inspections, not the roof cladding itself. The ambiguity between roof covering and roof cladding- defined in ASCE-7 but not in the IBC- is resolved in a separate code proposal. The exception for labeled products precludes the need for a special inspector to perform plant visits for roof coverings.

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

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

S135–07/08
1708.2

Proponent: Jim W. Sealy, FAIA; Robert E. Bachman, SE; and John D. Gillengerten, Building Seismic Safety Council of the National Institute of Building Sciences, representing FEMA/BSSC Code Resource Support Committee

Revise as follows:

1708.2 (Supp) Testing for seismic resistance. ~~The tests and qualification specified in Sections 1708.3 through 1708.6 are required as follows:~~

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C, D, E or F, as determined in Section 1613, shall meet the requirements of Sections 1708.3 and 1708.4, as applicable.
2. Designated seismic systems in structures assigned to Seismic Design Category C, D, E or F ~~in~~ subject to the special certification requirements of ASCE 7 Section 13.2.2 ~~of ASCE shall meet the requirements of~~ are required to be tested in accordance with Section 1708.5.

3. Architectural, mechanical and electrical components in structures assigned to Seismic Design Category C, D, E or F with an Ip=1.0 ~~shall be permitted to be seismically qualified by meeting the requirements of Section 1708.5~~ are required to be tested in accordance with Section 1708.5 where the general design requirements of ASCE 7 Section 13.2.1.2 for manufacturer's certification are satisfied by testing.
4. Seismic Isolation Systems are required to be tested in accordance with Section 1708.6.

Reason: This proposal clarifies the requirements for special certification of designated seismic systems. In ASCE 7, all nonstructural components must comply with general design provisions of Section 13.2.1. This section permits justification of components by project-specific design or certification by the manufacturer. The manufacturer can use analysis, testing, or experience data.

Special certification is only required for active mechanical and electrical components that must remain operable following an earthquake, and components with hazardous contents. Obtaining this certification requires shake table testing or use of experience data.

The changes to Items 1 and 4 provide pointers to the appropriate code sections. The changes to item 2 clarify when the special seismic qualification procedures (i.e., shake table testing) are required – only for those components identified in ASCE 7. Item 3 clarifies the process when the basic seismic design requirements (anchorage and bracing) are addressed by a manufacturer's certification.

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

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

S136–07/08
1708.3

Proponent: Philip Brazil, PE, SE, Reid Middleton, Inc., representing himself

Revise as follows:

1708.3 Reinforcing and prestressing steel. Certified mill test reports shall be provided for each shipment of ~~reinforcing steel~~ reinforcement used to resist flexural~~, shear~~ and axial forces in reinforced concrete intermediate ~~frames,~~ and special moment frames and boundary elements of special reinforced concrete ~~or reinforced~~ and masonry shear walls. Where reinforcement complying with ASTM A 615 ~~reinforcing steel~~ is used to resist earthquake-induced flexural and axial forces in special moment frames and ~~in wall boundary elements of~~ special reinforced concrete shear walls in structures assigned to Seismic Design Category C, D, E or F, as determined in Section 1613, the testing requirements of Section 21.2.5 of ACI 318 shall be met. Where reinforcement complying with ASTM A 615 ~~reinforcing steel~~ is to be welded, chemical tests shall be performed to determine weldability in accordance with Section 3.5.2 of ACI 318.

> **Exception**: Certified mill test reports are not required to be provided for reinforcement complying with ASTM A 706.

Reason: The purpose of this proposal is to align the provisions of IBC Section 1708.3 with related provisions in ASCE 7 and ACI 318. The reference to axial forces is deleted for consistency with Section 21.2.5 of ACI 318-05. The references to types of seismic-force resisting systems are revised for consistency with Table 12.1-1 of ASCE 7-05. The exception for reinforcement complying with ASTM A 706 is proposed in recognition of the exemption from any special requirements for the use of such bars by Sections 3.5.2 and 21.2.5 of ACI 318-05. Note that Section 16 of ASTM A 706 specifies requirements for the marking of individual reinforcing bars complying with the standard for ready identification during construction.

The references in the proposal to sections of ACI 318, current and proposed, are to the 2005 edition. I assume the 2008 edition of ACI 318 will be the edition that is referenced in the 2009 IBC. The sections in the public draft of ACI 318-08 corresponding to the sections in the proposal are 21.1.5.2 for Section 21.2.5 and 3.5.2 for Section 3.5.2.

The reference to the testing requirements in Section 21.2.5 of ACI 318-05 ought to specify all frame members and structural wall boundary elements, which could conceivably include intermediate and special reinforced concrete moment frames and shear walls. Section 21.1.5.2 of the public draft of ACI 318-08, however, revises the requirement so that it applies to special moment frames, special structural walls and coupling beams. Section 21.1.1.4 of the public draft on structures assigned to Seismic Design Category C specifies compliance with the applicable provisions of Sections 21.1.3 through 21.1.7 for structures using special moment frames or special structural walls. The proposed revisions incorporate these upcoming changes in ACI 318.

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

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

S137–07/08
1708.4

Proponent: Bonnie Manley, American Iron and Steel Institute, representing American Institute of Steel Construction

1. Revise as follows:

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

2. Revise Chapter 35 as follows:

AISC
341-05 Seismic Provisions for Structural Steel Buildings, including Supplement No. 1 dated 2005~~6~~. . . .. . 1613.6.2, 1707.2, 1709~~8~~.4, 2205.2.1, 2205.2.2, 2205.3, 2205.3.1

ASTM INTERNATIONAL
~~A 435/A 435M—90 (2001) Specification for Straight-beam Ultrasonic Examination of Steel Plates . . . . .1708.4~~
~~A 898/A 898M—91 (2001) Specification for Straight Beam Ultrasonic Examination of Rolled Steel Shapes . 1708.4~~

AWS
D1.1—04 Structural Welding Code—Steel . . . . .Table 1704.3, 1704.3.1~~, 1708.4~~

Reason: *Section 1709.4, 1st paragraph (Numbering based upon IBC-06 w/2007 Supplement):* Between the 2003 and 2006 editions of the IBC, the terminology in Section 1705 was changed from "quality assurance plan" to "statement of special inspection". Unfortunately, the change in terminology was not picked up in time for the 2005 edition of AISC 341, *Seismic Provisions for Structural Steel Buildings*. In order to ensure that there is no confusion, a direct reference to the quality assurance plan requirements in AISC 341 is recommended for structural steel. In fact, AISC 341-05 Appendix Q provides the user with the minimum acceptable requirements for a quality assurance plan that applies to the construction of welded joints, bolted joints, and other details in the seismic load resisting system. The requirements of AISC 341, Appendix Q are recommended for implementation without revision. Where appropriate, AISC 341-05 Appendix Q references AWS D1.1 for specific acceptance criteria. Thus, the second sentence of the first paragraph is unnecessary and redundant with language that currently exists in AISC 341-05.

Section 1709.4, 2nd paragraph: The requirements of this paragraph are recommended for deletion. This paragraph focuses on the ultrasonic testing of base metal that may be subject to lamellar tearing or have laminations present. However, AISC 341-05, Section 5.2(2)(c) currently addresses this specific topic by stating when non-destructive testing (NDT) is needed, where it is needed and the appropriate acceptance criteria as follows:

Q5.2(2)(c) Base Metal NDT for Lamellar Tearing and Laminations. After joint completion, base metal thicker than 1-1/2 in. (38 mm) loaded in tension in the through thickness direction in tee and corner joints, where the connected material is greater than 3/4 in. (19 mm) and contains CJP groove welds, shall be ultrasonically tested for discontinuities behind and adjacent to the fusion line of such welds. Any base metal discontinuities found within t/4 of the steel surface shall be accepted or rejected on the basis of criteria of AWS D1.1 Table 6.2, where t is the thickness of the part subjected to the through thickness strain.

Referenced in AISC-341, Section Q5.2(2)(c), AWS D1.1 Table 6.2 provides the acceptance criteria for ultrasonically tested joints when statically loaded. The criteria is similar to that used prior to adoption of the current language in IBC 2000, which had used the term of "larger reflector criteria" in the UBC, and left it to the engineer in the NBC. The "larger reflector criteria", a termed used in the 1970s, is now identified as a "Class A" discontinuity in Table 6.2. By referencing only Table 6.2, and not referencing Class A, the additional considerations of flaw length and reflector height is made.

Finally, the direct references to ASTM A 435 and ASTM A898 are no longer needed because the AISC 341 criteria has been made more restrictive regarding permitted flaws, and more properly reflects the angle-beam ultrasonic methodology used for post-welding examinations. The prior reference to ASTM A 435 and A 898 were straight-beam ultrasonic tests to detect laminations in base metal prior to welding, and have been deemed inadequate for post-welding lamellar tearing checks.

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

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

S138–07/08
1708.5

Proponent: Jim W. Sealy, FAIA; Robert E. Bachman, SE; and John D. Gillengerten, Building Seismic Safety Council of the National Institute of Building Sciences, representing FEMA/BSSC Code Resource Support Committee

Revise as follows:

1708.5 (Supp) Seismic ~~qualification~~ certification of ~~mechanical and electrical equipment~~ nonstructural components. The registered design professional shall state the applicable seismic ~~qualification~~ certification requirements for nonstructural components and designated seismic systems on the construction documents. ~~Each~~

1. The manufacturer of each designated seismic system component~~s~~ subject to the provisions of ASCE 7 Section 13.2.2 shall test or analyze the component and its mounting system or anchorage and submit a certificate of compliance for review and acceptance by the registered design professional for the design of the designated seismic system and for approval by the building official. ~~Qualification shall be by~~ Certification shall be based on an actual test on a shake table, by three-dimensional shock tests, by an analytical method using dynamic characteristics and forces, by the use of experience data (i.e., historical data demonstrating acceptable seismic performance) or by a more rigorous analysis providing for equivalent safety.
2. Manufacturers certification of compliance for the general design requirements of ASCE 7 Section 13.2.1 shall be based on analysis, testing, or experience data.

Reason: This proposal clarifies the requirements for special certification of designated seismic systems, and clarifies the distinctions between seismic certification and special seismic certification. In ASCE 7, all nonstructural components must comply with general design provisions of Section 13.2.1, which permits justification of components by project-specific design or certification by the manufacturer (through analysis, resting, or experience data). Special certification is only required for active mechanical and electrical components that must remain operable following an earthquake, and components with hazardous contents. Obtaining this certification requires shake table testing or use of experience data. The term "seismic qualification" is replaced, since it does not apply.

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

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

S139–07/08
1708.5

Proponent: Philip Brazil, PE, SE, Reid Middleton, Inc., representing himself

Revise as follows:

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

Reason: The change is proposed for consistency with other instances in the IBC when a registered design professional's specific responsibilities are specified. This is also being proposed to correct an oversight on my part during development of Proposal S37-06/07-AM.

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

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

S140–07/08
1709.2, [F] 903.3.5.2, 1802.2.6, 1802.2.7, 1805.5.1.3, 2306.4.2, 2306.4.3, 2306.4.4, 2306.4.5

Proponent: Philip Brazil, PE, SE, Reid Middleton, Inc., representing himself

Revise as follows:

[F] 903.3.5.2 Secondary water supply. A secondary on-sitewater supply equal to the hydraulically calculated sprinkler demand, including the hose stream requirement, shall be provided for high-rise buildings ~~in~~ assigned to Seismic Design Category C, D, E or F as determined by this code. The secondary water supply shall have a duration of not less than 30 minutes as determined by the occupancy hazard classification in accordance withNFPA13.

Exception: Existing buildings.

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

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

1802.2.6 Seismic Design Category C. Where a structure is ~~determined to be in~~ assigned to Seismic Design Category C in accordance with Section 1613, an investigation shall be conducted and shall include an evaluation of the following potential hazards resulting from earthquake motions: slope instability, liquefaction and surface rupture due to faulting or lateral spreading.

1802.2.7 Seismic Design Category D, E or F. Where the structure is ~~determined to be in~~ assigned to Seismic Design Category D, E or F, in accordance with Section 1613, the soils investigation requirements for Seismic Design Category C, given in Section 1802.2.6, shall be met, in addition to the following. The investigation shall include:

1. A determination of lateral pressures on basement and retaining walls due to earthquake motions.
2. An assessment of potential consequences of any liquefaction and soil strength loss, including estimation of differential settlement, lateral movement or reduction in foundation soil-bearing capacity, and shall address mitigation measures. Such measures shall be given consideration in the design of the structure and can include but are not limited to ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements or any combination of these measures. The potential for liquefaction and soil strength loss shall be evaluated for site peak ground acceleration magnitudes and source characteristics consistent with the design earthquake ground motions. Peak ground acceleration shall be determined from a site-specific study taking into account soil amplification effects, as specified in Chapter 21 of ASCE 7.

   Exception: A site-specific study need not be performed, provided that peak ground acceleration equal to $SDS/2.5$ is used, where SDS is determined in accordance with Section 21.2.1 of ASCE 7.

1805.5.1.3 Rubble stone. Foundation walls of rough or random rubble stone shall not be less than 16 inches (406 mm) thick. Rubble stone shall not be used for foundations ~~for~~ of structures ~~in~~ assigned to Seismic Design Category C, D, E or F.

2306.4.2 (Supp) Lumber sheathed shear walls. Single and double diagonally sheathed lumber shear walls shall be designed and constructed in accordance with AF&PA SDPWS. Single and double diagonally sheathed lumber walls shall not be used to resist seismic ~~loads~~ forces in structures ~~in~~ assigned to Seismic Design Category E or F.

2306.4.3 (Supp) Particleboard shear walls. Particleboard shear walls shall be designed and constructed in accordance with AF&PA SDPWS. Particleboard shear walls shall be permitted to resist horizontal forces using the allowable shear capacities set forth in Table 2306.4.3. Allowable capacities in Table 2306.4.3 are permitted to be increased 40 percent for wind design. Particleboard shall not be used to resist seismic forces in structures ~~in~~ assigned to Seismic Design Category D, E or F.

2306.4.4 (Supp) Fiberboard shear walls. Fiberboard shear walls shall be designed and constructed in accordance with AF&PA SDPWS. Fiberboard shear walls are permitted to resist horizontal forces using the allowable shear capacities set forth in Table 2306.4.4. Allowable capacities in Table 2306.4.4 are permitted to be increased 40 percent for wind design. Fiberboard shall not be used to resist seismic forces in structures ~~in~~ assigned to Seismic Design Category D, E or F.

2306.4.5 (Supp) Shear walls sheathed with other materials. Shear walls sheathed with portland cement plaster, gypsum lath, gypsum sheathing, or gypsum board shall be designed and constructed in accordance with AF&PA SDPWS. Shear walls sheathed with these materials are permitted to resist horizontal forces using the allowable shear capacities set forth in Table 2306.4.5. Shear walls sheathed with portland cement plaster, gypsum lath, gypsum sheathing, or gypsum board shall not be used to resist seismic ~~loads~~ forces in structures ~~in~~ assigned to Seismic Design Category E or F.

Reason: The changes are proposed for consistency with the use of "assigned to" in conjunction with structures and Seismic Design Category elsewhere in the 2006 IBC (more than 60 code sections) and with Proposal S39-04/05-AM. The sections in the proposal contain the only such instances in the 2007 Supplement and 2006 IBC with respect to structures and Seismic Design Category that merit consideration by the Correlating Committee. Other instances should be modified through the code development process. In Sections 2306.4.2 and 2306.4.5, "seismic loads" is changed to "seismic forces" for consistency with use of the latter term throughout the IBC.

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

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

S141–07/08
1714.5; IRC R613.4

Proponent: John Woestman, The Kellen Company, representing Window and Door Manufacturers Association

THESE PROPOSALS ARE ON THE AGENDA OF THE IBC STRUCTURAL AND THE IRC BUILDING/ENERGY CODE DEVELOPMENT COMMITTEES AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

1714.5 Exterior window and door assemblies. The ~~design pressure~~ performance rating of exterior windows and doors in buildings shall be determined in accordance with Section 1714.5.1 or 1714.5.2.

> **Exception:** Structural wind load design pressures for exterior window and door units smaller than the size tested in accordance with Section 1714.5.1 or 1714.5.2 shall be permitted to be higher than the design value of the tested unit provided such higher pressures are determined by accepted engineering analysis. All components of the small unit shall be the same as the tested unit. Where such calculated design pressures are used, they shall be validated by an additional test of the window unit having the highest allowable design pressure.

1714.5.1 Exterior windows and doors. Exterior windows, ~~and~~ sliding doors, and side-hinged doors shall be tested and labeled as conforming to AAMA/WDMA/CSA101/I.S.2/A440. The label shall state the name of the manufacturer, the approved labeling agency and the product designation as specified in AAMA/ WDMA/CSA101/I.S.2/A440. ~~Exterior side-hinged doors shall be tested and labeled as conforming to AAMA/WDMA/CSA101/I.S.2/A440 or comply with Section 1714.5.2.~~ Products tested and labeled as conforming to AAMA/WDMA/CSA 101/I.S.2/A440 shall not be subject to the requirements of Sections 2403.2 and 2403.3.

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

PART II – IRC B/E

Revise as follows:

R613.4 Testing and labeling. Exterior windows, ~~and~~ sliding doors, and side-hinged doors shall be tested ~~by an approved independent laboratory, and bear a label identifying manufacturer, performance characteristics and approved inspection agency to indicate compliance with AAMA/WDMA/CSA 101/I.S.2/A440. Exterior side-hinged doors shall be tested~~ and labeled as conforming to AAMA/ WDMA/CSA 101/I.S.2/A440 ~~or comply with Section R613.6.~~ The label shall state the name of the manufacturer, the approved labeling agency and the product designation as specified in AAMA/WDMA/CSA 101/I.S.2/A440.

Exception: Decorative glazed openings.

Reason: (IBC) This proposal adds testing and labeling requirements for side-hinged door assemblies that are included within the scope of AAMA/WDMA/CSA 101/I.S.2/A440. Starting with the 2000 IBC (and IRC), exterior windows and exterior sliding doors have been required to be tested and labeled as conforming to AAMA/WDMA/CSA 101/I.S.2/A440 (or to the 1997 or 2002 versions of this standard) requiring window and sliding door assemblies to meet air infiltration, water infiltration, structural, operational, and forced entry performance requirements. This proposal adds side-hinged doors to the list of exterior fenestration products which will be required to meet air infiltration, water infiltration, operational, and forced entry performance requirements in addition to structural performance requirements currently required by the IBC and IRC.

It is important to note that the following products are not within the scope of AAMA/WDMA/CSA 101/I.S.2/A440, as listed in this industry consensus standard, and would not be affected by this proposal: curtain wall and storefront, storm doors, commercial entrance systems, revolving doors, site-built door systems, and commercial steel doors.

This proposal clarifies in the IBC that exterior window and door performance is not just design pressure. This clarification is important as exterior fenestration assemblies tested and labeled as conforming to AAMA/WDMA/CSA 101/I.S.2/A440 are required by this industry consensus standard to meet numerous performance requirements in addition to design pressure performance.

There are several editorial changes which clarify the code.

This proposal will increase complexity and cost of manufacturing side-hinged door assemblies because it requires side-hinged door assemblies to be tested, and labeled, to performance requirements previously not required.

(IRC) This proposal adds testing and labeling requirements for side-hinged door assemblies that are included within the scope of AAMA/WDMA/CSA 101/I.S.2/A440. Starting with the 2000 IRC (and IBC), exterior windows and exterior sliding doors have been required to be tested and labeled as conforming to AAMA/WDMA/CSA 101/I.S.2/A440 (or to the 1997 or 2002 versions of this standard) requiring window and sliding door assemblies to meet air infiltration, water infiltration, structural, operational, and forced entry performance requirements. This proposal adds side-hinged doors to the list of exterior fenestration products which will be required to meet air infiltration, water infiltration, operational, and forced entry performance requirements in addition to structural performance requirements currently required by the IBC and IRC.

This proposal also revises the language describing labeling requirements in IRC Section R613.4 to be consistent with the IBC.

It is important to note that storm doors and site-built door systems are not within the scope of AAMA/WDMA/CSA 101/I.S.2/A440, as listed in this industry consensus standard, and would not be affected by this proposal.

This proposal will increase complexity and cost of manufacturing side-hinged door assemblies because it requires side-hinged door assemblies to be tested, and labeled, to performance requirements previously not required.

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

PART I – IBC STRUCTURAL

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

PART II – IRC B/E

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

S142–07/08
1714.5.1

Proponent: William E. Koffel, PE, Koffel Associates, Inc., representing Glazing Industry Code Committee

Revise as follows:

1714.5.1 Exterior windows and doors. Exterior windows and sliding doors shall be tested and labeled as conforming to AAMA/WDMA/CSA101/I.S.2/A440. The label shall state the name of the manufacturer, the approved labeling agency and the product designation as specified in AAMA/ WDMA/CSA101/I.S.2/A440. Exterior side-hinged doors shall be tested and labeled as conforming to AAMA/WDMA/CSA101/I.S.2/A440 or comply with Section 1714.5.2. ~~Products tested and labeled as conforming to AAMA/WDMA/CSA 101/I.S.2/A440 shall not be subject to the requirements of Sections 2403.2 and 2403.3.~~

Reason: The purpose of this proposal is to remove the exemption that fenestration products labeled to AAMA/WDMA/CSA 101/I.S.2/A440 do not have to meet the requirements of sections 2403.2 and 2403.3, which ensure safe performance through proper support of glass. Specifically, section 2403.3 requires that the deflection of framing members supporting glass may not exceed 1/175 of the glass edge length (or ¾ inch, whichever is less) when subjected to the design load. Chapter 24 of the IBC relies on glass design curves that are contained in ASTM E 1300. This ASTM standard recognizes the importance of limiting edge deflection of the glass and also recommends a limitation of 1/175 of the glass edge length. Prior to the IBC, the legacy codes required deflection limitations of 1/175 of the span for glass holding members. It was not until the IBC was published that this exemption was allowed.

AAMA/WDMA/CSA 101/I.S.2/A440 does require testing in accordance with ASTM E330 and measurement of deflection. However, AAMA/WDMA/CSA 101/I.S.2/A440 only places a limit on the frame and sash deflection for heavy commercial (HC) and architectural products (AW), and has no requirement on deflection for residential (R), light commercial (LC), and commercial (C) products. Excessive deflection of the frame or sash can have an adverse effect on stress in the glass and could result in glass breakage at or below design loads creating a safety concern. The single ASTM E330 load test required in AAMA/WDMA/CSA 101/I.S.2/A440 is not statistically significant in ensuring that the stress does not increase the probability of breakage beyond the industry standard of eight lites per thousand when the deflection limitation of 1/175 is exceeded. While this deflection exemption remains in the IRC for residential buildings, it is inappropriate to have an exemption for these products when used in more diverse and larger buildings built to the IBC. This proposal would ensure that an appropriate limit on frame deflection is placed on fenestration products from all performance classes. Because the deflection is already being measured for all these products (but not limited for R, LC, and C classes), there is no cost impact except for products which do not comply with this more conservative and appropriate requirement.

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

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

S143–07/08
1714.5.2, Chapter 35 (New)

Proponent: John Woestman, The Kellen Company, representing Door Safety Council

1. Revise as follows:

1714.5.2 (Supp) Exterior windows and door assemblies not provided for in Section 1714.5.1. Exterior window and door assemblies shall be tested in accordance with ASTM E 330. <u>Exterior side-hinged door assemblies shall be permitted to be tested in accordance with ANSI/SDI A250.13.</u> Structural performance of garage doors shall be determined in accordance with either ASTM E 330 or ANSI/DASMA 108, and shall meet the acceptance criteria of ANSI/DASMA 108. Exterior window and door assemblies containing glass shall comply with Section 2403. The design pressure for testing shall be calculated in accordance with Chapter 16. Each assembly shall be tested for <u>a minimum of</u> 10 seconds at a load equal to 1.5 times the design pressure.

2. Add standard to Chapter 35 as follows:

ANSI

<u>ANSI/SDI A250.13-XX</u> <u>Testing and Rating of Severe Windstorm Resistant Components for Swinging Door Assemblies.</u>

Reason: This proposed change allows an alternative method to demonstrate structural performance for side-hinged doors by requiring doors to be tested per ANSI/SDI A250.13-XX. A250.13-XX, which is under development to update A250.13-03, will contain language that prescribes how door components are to be selected to create door assemblies expected to perform equivalently to a door assembly tested to ASTM E 330.

This proposal helps resolve performance and code compliance issues when doors are assembled from components from multiple sources and include interchangeable elements.

Through the ASTM standards development process, members of the Steel Door Institute (SDI) and members of the Builders Hardware Manufacturers' Association (BHMA) developed a national standard for a component-based approach to testing for windstorm resistance of swinging door assemblies. The test procedures used in this standard represent the most severe requirements found in the windstorm resistance standards in use in building codes. However, the procedures are designed to isolate, as much as possible, the loads and conditions that a particular component is subjected to in the full assembly test and duplicate these specific conditions. Using a combination of worst-case assembly design and safety factors, this testing was designed to provide a component rating that related directly to the component's ability to withstand the conditions that occur in a full assembly test.

Prior to releasing the current ANSI/SDI A250.13 standard, the BHMA/SDI task group conducted validation testing where components were expected to be rated at three design-load target values. Those components were tested to establish their ratings by the proposed procedure. The results of this process confirmed that assemblies made up of rated components would perform as expected. In addition, the validation test showed that where a component was identified as the weakest element of an assembly, based on the component tests, the same component would fail in a similar manner when tested as part of an assembly to levels exceeding the component's rated capacity.

Building designers will use performance criteria of door components, per ANSI/SDI A250.13, to select appropriate components to create door assemblies by conducting an opening-by-opening design analysis, specify components, verify code compliance, and submit the results through the normal plans review process. Code Authorities will thus need only to verify that the design load and compliance analysis has been correctly carried out and that the specified components are actually installed during construction in accordance with the manufacturer's instructions and project specifications.

Cost Impact: The code change proposal will not increase the cost of construction.
Analysis: A review of the standard(s) proposed for inclusion in the code, ANSI/SDI A250.13, for compliance with ICC criteria for referenced standards given in Section 3.6 of Council Policy #CP 28 will be posted on the ICC website on or before January 15, 2008.

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

S144–07/08
1715.1.1, 1715.1.2, 1715.1.3, 1715.1.4, Chapter 35 (New)

Proponent: Greg Greenlee, PE, USP Structural Connectors representing himself

1. Revise as follows:

1715.1.1 Test standards for joist hangers. The vertical load-bearing capacity, torsional moment capacity and deflection characteristics of joist hangers shall be determined in accordance with ASTMD1761 <u>ASTM D 7147 or an approved method.</u> ~~using lumber having a specific gravity of 0.49 or greater, but not greater than 0.55, as determined in accordance with AF&PA NDS for the joist and headers.~~

~~**Exception:** The joist length shall not be required to exceed 24 inches (610 mm).~~

~~**1715.1.2 Vertical load capacity for joist hangers.** The vertical load capacity for the joist hanger shall be determined by testing a minimum of three joist hanger assemblies as specified in ASTM D 1761. If the ultimate vertical load for any one of the tests varies more than 20 percent from the average ultimate vertical load, at least three additional tests shall be conducted. The allowable vertical load of the joist hanger shall be the lowest value determined from the following:~~

1. ~~The lowest ultimate vertical load for a single hanger from any test divided by three (where three tests are conducted and each ultimate vertical load does not vary more than 20 percent from the average ultimate vertical load).~~
2. ~~The average ultimate vertical load for a single hanger from all tests divided by three (where six or more tests are conducted).~~
3. ~~The average from all tests of the vertical loads that produce a vertical movement of the joist with respect to the header of 0.125 inch (3.2 mm).~~
4. ~~The sum of the allowable design loads for nails or other fasteners utilized to secure the joist hanger to the wood members and allowable bearing loads that contribute to the capacity of the hanger.~~
5. ~~The allowable design load for the wood members forming the connection.~~

~~**1715.1.3 Torsional moment capacity for joist hangers.** The torsional moment capacity for the joist hanger shall be determined by testing at least three joist hanger assemblies as specified in ASTM D 1761. The allowable torsional moment of the joist hanger shall be the average torsional moment at which the lateral movement of the top or bottom of the joist with respect to the original position of the joist is 0.125 inch (3.2 mm).~~

1715.1.~~4~~ <u>2</u> Design value modifications for joist hangers. Allowable design values for joist hangers that are determined by ~~Item 4 or 5 in Section 1715.1.2~~ <u>calculation</u> shall be permitted to be modified by the appropriate duration of loading factors as specified in AF&PA NDS but shall not exceed the direct loads as determined by ~~Item 1, 2 or 3 in Section 1715.1.2.~~ <u>testing conducted in accordance with ASTM D 7147 or an approved method.</u> Allowable design values determined by ~~Item 1, 2 or 3 in Section 1715.1.2~~ <u>testing conducted in accordance with ASTM D 1761, ASTM D 7147 or an approved method</u> shall not be modified by duration of loading factors.

<u>**1715.1.3 Design values for holdowns and other structural wood connectors.** Allowable design values for holdowns and other structural wood connectors shall be determined in accordance with an approved method.</u>

2. Add standard to Chapter 35 as follows:

ASTM

<u>D 7147–05</u> <u>Standard Specification for Testing and Establishing Allowable Loads of joist Hangers ;1715.1.1, 1715.1.2</u>

Reason: In 2005 ASTM adopted the standard D7147 for testing and establishing allowable loads for joist hangers which replaced the joist hanger testing procedures prescribed in ASTM D1761. In addition, the ICC-ES recently revised its acceptance criteria (AC-13) to reference ASTM D7147 as well as ASTM D1761. While the content in ASTM D1761, D7147 and ICC EC AC-13 is similar, there are differences between the test standards. In addition, manufactures have been working with ICC-ES in recent years to develop additional acceptance criteria for the different types of connection hardware available. Manufacturers of these products most often test and submit their products according to the provisions of the appropriate ICC-ES acceptance criteria. This revision updates the ASTM reference as well as eliminated the duplicate language contained in either the ASTM standard or ICC-ES acceptance criteria. This proposed change also addresses requirements for structural connectors used in wood construction besides joist hangers.

The proposed language allows for testing to be conducted in accordance with ASTM D1761, ASTM D7147 or an approved standard. An approved standard would be an ICC-ES acceptance criteria; in this case AC-13. Although ASTM D7147 was created to replace the joist hanger test method included in D1761, the reference to D1761 has been retained in the code language because it is referenced in the most recent version of AC-13.

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

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM D 7147, for compliance with ICC criteria for referenced standards given in Section 3.6 of Council Policy #CP 28 will be posted on the ICC website on or before January 15, 2008.

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

S145–07/08

1801.2.1

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

Revise as follows:

1801.2.1 Foundation design for seismic overturning. Where the foundation is proportioned using the load combinations of Section 1605.2 or 1605.3.1, and the computation of the seismic overturning moment is by the equivalent lateral-force method or the modal analysis method, the proportioning shall be in accordance with Section 12.13.4 of ASCE 7.

Reason: Code consistency. ASCE 7 permits the reduction of seismic overturning for foundation design where either strength design or allowable stress design load combinations are used. The load combinations of Sections 1605.2 and 1605.3.1 correspond to the two sets of ASCE 7 load combinations. Because the load combinations in Section 1605.3.2 include 0.9*D* where overturning is assessed (rather than 0.6*D* as in Section 1605.3.1), reduction of seismic overturning in accordance with Section 12.13.4 of ASCE 7 would be unconservative where those load combinations are used.

In a related proposal Section 1801.2.1 is moved to new Section 1808.3.1. If both proposals are approved this change should be made in the new section.

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

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

S146–07/08

106.1, 1610.1, 1802, 1803, 1805.3.5, 1808.2.2, 1808.2.8.4, 1808.2.10, 3304.1.4, Appendix J101.1, J104.3, J106.1, J107.1, J107.6

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

Revise as follows:

106.1 (Supp) General. Submittal documents consisting of construction documents, statement of special inspections, geotechnical report and other data shall be submitted in one or more sets with each permit application. The construction documents shall be prepared by a registered design professional where required by the statutes of the jurisdiction in which the project is to be constructed. Where special conditions exist, the building official is authorized to require additional construction documents to be prepared by a registered design professional.

> **Exception:** The building official is authorized to waive the submission of construction documents and other data not required to be prepared by a registered design professional if it is found that the nature of the work applied for is such that review of construction documents is not necessary to obtain compliance with this code.

1610.1 General. Basement, foundation and retaining walls shall be designed to resist lateral soil loads. Soil loads specified in Table 1610.1 shall be used as the minimum design lateral soil loads unless ~~specified~~ determined otherwise ~~in~~ by a ~~soil~~ geotechnical investigation ~~report approved by the building official~~ in accordance with Section 1803. Basement walls and other walls in which horizontal movement is restricted at the top shall be designed for at-rest pressure. Retaining walls free to move and rotate at the top ~~are~~ shall permitted to be designed for active pressure. Design lateral pressure from surcharge loads shall be added to the lateral earth pressure load. Design lateral pressure shall be increased if soils with expansion potential are present at the site.

Exception: Basement walls extending not more than 8 feet (2438 mm) below grade and supporting flexible floor systems shall be permitted to be designed for active pressure.

SECTION ~~1802~~ 1803
~~FOUNDATION AND SOILS~~ GEOTECHNICAL INVESTIGATIONS

~~1802.1~~ 1803.1 General. Geotechnical ~~Foundation and soils~~ investigations shall be conducted in accordance ~~conformance~~ with Section~~s~~ 1803.2 ~~1802.2 through 1802.6~~ and reported in accordance with Section 1803.6. Where required by the building official, or where geotechnical investigations involve in-situ testing, laboratory testing, or engineering calculations, such investigations ~~the classification and investigation of the soil~~ shall be ~~made~~ conducted by a registered design professional.

~~1802.2~~ 1803.2 Investigations required ~~Where required~~. Geotechnical investigations shall be conducted in accordance with ~~The owner or applicant shall submit a foundation and soils investigation to the building official where required in~~ Sections 1803.3 through 1803.5 ~~1802.2.1 through 1802.2.7~~.

Exception: The building official ~~need not require~~ shall be permitted to waive the requirement for a geotechnical ~~foundation or soils~~ investigation where satisfactory data from adjacent areas is available that demonstrates an investigation is not necessary for any of the conditions in Sections 1803.5.1 through 1803.5.6 and Sections 1803.5.10 and 1803.5.11 ~~1802.2.1 through 1802.2.6~~.

2. Delete without substitution:

~~**1802.2.1 Questionable soil.** Where the classification, strength or compressibility of the soil are in doubt or where a load-bearing value superior to that specified in this code is claimed, the building official shall require that the necessary investigation be made. Such investigation shall comply with the provisions of Sections 1802.4 through 1802.6.~~

~~**1802.2.2 Expansive soils.** In areas likely to have expansive soil, the building official shall require soil tests to determine where such soils do exist.~~

~~**1802.2.3 Ground-water table.** A subsurface soil investigation shall be performed to determine whether the existing ground-water table is above or within 5 feet (1524 mm) below the elevation of the lowest floor level where such floor is located below the finished ground level adjacent to the foundation.~~

~~**Exception:** A subsurface soil investigation shall not be required where waterproofing is provided in accordance with Section 1807.~~

~~**1802.2.4 Pile and pier foundations.** Pile and pier foundations shall be designed and installed on the basis of a foundation investigation and report as specified in Sections 1802.4 through 1802.6 and Section 1808.2.2.~~

~~**1802.2.5 Rock strata.** Where subsurface explorations at the project site indicate variations or doubtful characteristics in the structure of the rock upon which foundations are to be constructed, a sufficient number of borings shall be made to a depth of not less than 10 feet (3048 mm) below the level of the foundations to provide assurance of the soundness of the foundation bed and its load-bearing capacity.~~

~~**1802.2.6 Seismic Design Category C.** Where a structure is determined to be in Seismic Design Category C in accordance with Section 1613, an investigation shall be conducted and shall include an evaluation of the following potential hazards resulting from earthquake motions: slope instability, liquefaction and surface rupture due to faulting or lateral spreading.~~

~~**1802.2.7 Seismic Design Category D, E or F.** Where the structure is determined to be in Seismic Design Category D, E or F, in accordance with Section 1613, the soils investigation requirements for Seismic Design Category C, given in Section 1802.2.6, shall be met, in addition to the following.~~

~~The investigation shall include:~~

1. ~~A determination of lateral pressures on basement and retaining walls due to earthquake motions.~~
2. ~~An assessment of potential consequences of any liquefaction and soil strength loss, including estimation of differential settlement, lateral movement or reduction in foundation soil-bearing capacity, and shall address mitigation measures. Such measures shall be given consideration in the design of the structure and~~

~~can include but are not limited to ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements or any combination of these measures. The potential for liquefaction and soil strength loss shall be evaluated for site peak ground acceleration magnitudes and source characteristics consistent with the design earthquake ground motions. Peak ground acceleration shall be determined from a site-specific study taking into account soil amplification effects, as specified in Chapter 21 of ASCE 7.~~

~~**Exception:** A site-specific study need not be performed, provided that peak ground acceleration equal to S_{DS}/2.5 is used, where S_{DS} is determined in accordance with Section 21.2.1 of ASCE 7.~~

~~**1802.3 Soil classification.** Where required, soils shall be classified in accordance with Section 1802.3.1 or 1802.3.2.~~

~~**1802.3.1 General.** For the purposes of this chapter, the definition and classification of soil materials for use in Table 1804.2 shall be in accordance with ASTM D 2487.~~

~~**1802.3.2 Expansive soils.** Soils meeting all four of the following provisions shall be considered expansive, except that tests to show compliance with Items 1, 2 and 3 shall not be required if the test prescribed in Item 4 is conducted:~~

1. ~~Plasticity index (PI) of 15 or greater, determined in accordance with ASTM D 4318.~~
2. ~~More than 10 percent of the soil particles pass a No. 200 sieve (75 _m), determined in accordance with ASTM D 422.~~
3. ~~More than 10 percent of the soil particles are less than 5 micrometers in size, determined in accordance with ASTM D 422.~~
4. ~~Expansion index greater than 20, determined in accordance with ASTM D 4829.~~

3. Revise as follows:

~~**1802.4**~~ **1803.3 Basis of investigation.** Soil classification shall be based on observation and any necessary tests of the materials disclosed by borings, test pits or other subsurface exploration made in appropriate locations. Additional studies shall be made as necessary to evaluate slope stability, soil strength, position and adequacy of load-bearing soils, the effect of moisture variation on soil-bearing capacity, compressibility, liquefaction and expansiveness.

~~**1802.4.1**~~ **1803.3.1 Scope of investigation** ~~**Exploratory boring.**~~ The scope of the geotechnical ~~soil~~ investigation including the number and types of borings or soundings, the equipment used to drill ~~and~~ or sample, the in-situ testing equipment and the laboratory testing program shall be determined by a registered design professional.

~~**1802.5**~~ **1803.4 Qualified representative** ~~**Soil boring and sampling.**~~ The investigation ~~soil boring and sampling~~ procedure and apparatus shall be in accordance with generally accepted engineering practice. The registered design professional shall have a fully qualified representative on the site during all boring ~~and~~ or sampling operations.

4. Add new text as follows:

1803.5 Investigated conditions. Geotechnical investigations shall be conducted as indicated in Sections 1803.5.1 through 1803.5.12.

1803.5.1 Classification. Soil materials shall be classified in accordance with ASTM D 2487.

1803.5.2 Questionable soil. Where the classification, strength or compressibility of the soil are in doubt or where a load-bearing value superior to that specified in this code is claimed, the building official shall be permitted to require that a geotechnical investigation be conducted.

1803.5.3 Expansive soil. In areas likely to have expansive soil, the building official shall require soil tests to determine where such soils do exist. Soils meeting all four of the following provisions shall be considered expansive, except that tests to show compliance with Items 1, 2 and 3 shall not be required if the test prescribed in Item 4 is conducted:

1. Plasticity index (PI) of 15 or greater, determined in accordance with ASTM D 4318.
2. More than 10 percent of the soil particles pass a No. 200 sieve (75 µm), determined in accordance with ASTM D 422.
3. More than 10 percent of the soil particles are less than 5 micrometers in size, determined in accordance with ASTM D 422.
4. Expansion index greater than 20, determined in accordance with ASTM D 4829.

1803.5.4 Ground-water table. A subsurface soil investigation shall be performed to determine whether the existing ground-water table is above or within 5 feet (1524 mm) below the elevation of the lowest floor level where such floor is located below the finished ground level adjacent to the foundation.

> **Exception:** A subsurface soil investigation to determine the location of the ground-water table shall not be required where waterproofing is provided in accordance with Section 1807.

1803.5.5 Deep foundations. Where deep foundations will be used, a geotechnical investigation shall be conducted and shall include all of the following, unless sufficient data upon which to base the design and installation is otherwise available:

1. Recommended deep foundation types and installed capacities.
2. Recommended center-to-center spacing of deep foundation elements.
3. Driving criteria.
4. Installation procedures.
5. Field inspection and reporting procedures (to include procedures for verification of the installed bearing capacity where required).
6. Load test requirements.
7. Suitability of deep foundation materials for the intended environment.
8. Designation of bearing stratum or strata.
9. Reductions for group action, where necessary.

1803.5.6 Rock strata. Where subsurface explorations at the project site indicate variations or doubtful characteristics in the structure of the rock upon which foundations are to be constructed, a sufficient number of borings shall be made to a depth of not less than 10 feet (3048 mm) below the level of the foundations to provide assurance of the soundness of the foundation bed and its load-bearing capacity.

1803.5.7 Excavation near foundations. Where excavation will remove lateral support from any foundation, an investigation shall be conducted to assess the potential consequences and address mitigation measures.

1803.5.8 Compacted fill material. Where shallow foundations will bear on compacted fill material more than 12 inches (305 mm) in depth, a geotechnical investigation shall be conducted and shall include all of the following:

1. Specifications for the preparation of the site prior to placement of compacted fill material.
2. Specifications for material to be used as compacted fill.
3. Test methods to be used to determine the maximum dry density and optimum moisture content of the material to be used as compacted fill.
4. Maximum allowable thickness of each lift of compacted fill material.
5. Field test method for determining the in-place dry density of the compacted fill.
6. Minimum acceptable in-place dry density expressed as a percentage of the maximum dry density determined in accordance with Item 3.
7. Number and frequency of field tests required to determine compliance with Item 6.

1803.5.9 Controlled low-strength material (CLSM). Where shallow foundations will bear on controlled low-strength material (CLSM), a geotechnical investigation shall be conducted and shall include all of the following:

1. Specifications for the preparation of the site prior to placement of the CLSM.
2. Specifications for the CLSM.
3. Laboratory or field test method(s) to be used to determine the compressive strength or bearing capacity of the CLSM.
4. Test methods for determining the acceptance of the CLSM in the field.
5. Number and frequency of field tests required to determine compliance with Item 4.

1803.5.10 Alternate setback and clearance. Where setbacks or clearances other than those required in Section 1805.3 are desired, the building official shall be permitted to require a geotechnical investigation by a registered design professional to demonstrate that the intent of Section 1805.3 would be satisfied. Such an investigation shall include consideration of material, height of slope, slope gradient, load intensity and erosion characteristics of slope material.

1803.5.11 Seismic Design Categories C through F. For structures assigned to Seismic Design Category C, D, E, or F in accordance with Section 1613, a geotechnical investigation shall be conducted, and shall include an evaluation of all the following potential geologic and seismic hazards:

1. <ins>Slope instability.</ins>
2. <ins>Liquefaction.</ins>
3. <ins>Differential settlement.</ins>
4. <ins>urface displacement due to faulting or lateral spreading.</ins>

<ins>**1803.5.12 Seismic Design Categories D through F.** For structures assigned to Seismic Design Category D, E or F in accordance with Section 1613, the geotechnical investigation required by Section 1803.5.11 shall also include:</ins>

1. <ins>The determination of lateral pressures on basement and retaining walls due to earthquake motions.</ins>
2. <ins>The potential for liquefaction and soil strength loss evaluated for site peak ground accelerations, magnitudes, and source characteristics consistent with the design earthquake ground motions. Peak ground acceleration shall be permitted to be determined based on a site-specific study taking into account soil amplification effects, as specified in Chapter 21 of ASCE 7, or, in the absence of such a study, peak ground accelerations shall be assumed equal to S_{DS} / 2.5, where S_{DS} is determined in accordance with Section 11.4 of ASCE 7.</ins>
3. <ins>An assessment of potential consequences of liquefaction and soil strength loss, including estimation of differential settlement, lateral movement, lateral loads on foundations, reduction in foundation soil-bearing capacity, increases in lateral pressures on retaining walls, and flotation of buried structures.</ins>
4. <ins>Discussion of mitigation measures such as, but not limited to, ground stabilization, selection of appropriate foundation type and depths, selection of appropriate structural systems to accommodate anticipated displacements and forces, or any combination of these measures and how they shall be considered in the design of the structure.</ins>

5. Revise as follows:

~~1802.6~~ <ins>1803.6</ins> ~~Reports~~ <ins>Reporting</ins>. The soil classification and design load-bearing capacity shall be shown on the construction documents. Where <ins>geotechnical investigations are</ins> required ~~by the building official~~, a written report of the investigation<ins>s</ins> shall be submitted <ins>to the building official by the owner or authorized agent at the time of permit application. This geotechnical report shall</ins> ~~that~~ includes, but need not be limited to, the following information:

1. A plot showing the location of ~~test borings and/or excavations~~ <ins>the soil investigations</ins>.
2. A complete record of the <ins>soil boring and penetration test logs and</ins> soil samples.
3. A record of the soil profile.
4. Elevation of the water table, if encountered.
5. Recommendations for foundation type and design criteria, including but not limited to: bearing capacity of natural or compacted soil; provisions to mitigate the effects of expansive soils; mitigation of the effects of liquefaction liquefaction, differential settlement and varying soil strength; and the effects of adjacent loads.
6. Expected total and differential settlement.
7. ~~Pile and pier~~ <ins>Deep</ins> foundation information in accordance with Section ~~1808.2.2~~ <ins>1803.5.5</ins>.
8. Special design and construction provisions for ~~footings or~~ foundations <ins>of structures</ins> founded on expansive soils, as necessary.
9. Compacted fill material properties and testing in accordance with Section ~~1803.5~~ <ins>1803.5.7</ins>.
10. <ins>Controlled low-strength material properties and testing in accordance with Section 1803.5.8.</ins>

SECTION ~~1803~~ <ins>1804</ins>
EXCAVATION, GRADING AND FILL

~~1803.1~~ <ins>1804.1</ins> ~~Excavations~~ <ins>Excavation</ins> near ~~footings or~~ foundations. ~~Excavations~~ <ins>Excavation</ins> for any purpose shall not remove lateral support from any ~~footing or~~ foundation without first underpinning or protecting the ~~footing or~~ foundation against settlement or lateral translation.

~~1803.2~~ <ins>1804.2</ins> Placement of backfill. The excavation outside the foundation shall be backfilled with soil that is free of organic material, construction debris, cobbles and boulders or a controlled low-strength material (CLSM). The backfill shall be placed in lifts and compacted, in a manner that does not damage the foundation or the waterproofing or dampproofing material.

Exception: Controlled low-strength material need not be compacted.

~~1803.3~~ <ins>1804.3</ins> Site grading. The ground immediately adjacent to the foundation shall be sloped away from the building at a slope of not less than one unit vertical in 20 units horizontal (5-percent slope) for a minimum distance of 10 feet (3048 mm) measured perpendicular to the face of the wall. If physical obstructions or lot lines prohibit 10 feet (3048 mm) of horizontal distance, a 5-percent slope shall be provided to an approved alternative method of diverting water away from the foundation. Swales used for this purpose shall be sloped a minimum of 2 percent where located within 10 feet (3048 mm) of the building foundation. Impervious surfaces within 10 feet (3048 mm) of the building foundation shall be sloped a minimum of 2 percent away from the building.

Exception: Where climatic or soil conditions warrant, the slope of the ground away from the building foundation is permitted to be reduced to not less than one unit vertical in 48 units horizontal (2-percent slope). The procedure used to establish the final ground level adjacent to the foundation shall account for additional settlement of the backfill.

~~**1803.4**~~ **1804.4** **Grading and fill in flood hazard areas.** In flood hazard areas established in Section 1612.3, grading and/or fill shall not be approved:

1. Unless such fill is placed, compacted and sloped to minimize shifting, slumping and erosion during the rise and fall of flood water and, as applicable, wave action.
2. In floodways, unless it has been demonstrated through hydrologic and hydraulic analyses performed by a registered design professional in accordance with standard engineering practice that the proposed grading or fill, or both, will not result in any increase in flood levels during the occurrence of the design flood.
3. In flood hazard areas subject to high-velocity wave action, unless such fill is conducted and/or placed to avoid diversion of water and waves toward any building or structure.
4. Where design flood elevations are specified but floodways have not been designated, unless it has been demonstrated that the cumulative effect of the proposed flood hazard area encroachment, when combined with all other existing and anticipated flood hazard area encroachment, will not increase the design flood elevation more than 1 foot (305 mm) at any point.

~~**1803.5**~~ **1804.5** **Compacted fill material.** Where ~~footings~~ shallow foundations will bear on compacted fill material, the compacted fill shall comply with the provisions of an approved geotechnical report, as set forth in Section 1803 ~~which shall contain the following~~:

~~1. Specifications for the preparation of the site prior to placement of compacted fill material.~~
~~2. Specifications for material to be used as compacted fill.~~
~~3. Test method to be used to determine the maximum dry density and optimum moisture content of the material to be used as compacted fill.~~
~~4. Maximum allowable thickness of each lift of compacted fill material.~~
~~5. Field test method for determining the in-place dry density of the compacted fill.~~
~~6. Minimum acceptable in-place dry density expressed as a percentage of the maximum dry density determined in accordance with Item 3.~~
~~7. Number and frequency of field tests required to determine compliance with Item 6.~~

Exception: Compacted fill material less than 12 inches (305 mm) in depth need not comply with an approved report, provided it has been compacted to a minimum of 90 percent Modified Proctor in accordance with ASTM D 1557. The compaction shall be verified by a qualified inspector approved by the building official.

~~**1803.6**~~ **1804.6** **Controlled low-strength material (CLSM).** Where ~~footings~~ shallow foundations will bear on controlled low-strength material (CLSM), the CLSM shall comply with the provisions of an approved geotechnical report, as set forth in Section 1803. ~~which shall contain the following:~~

~~1. Specifications for the preparation of the site prior to placement of the CLSM.~~
~~2. Specifications for the CLSM.~~
~~3. Laboratory or field test method(s) to be used to determine the compressive strength or bearing capacity of the CLSM.~~
~~4. Test methods for determining the acceptance of the CLSM in the field.~~
~~5. Number and frequency of field tests required to determine compliance with Item 4.~~

6. Revise as follows:

1805.3.5 Alternate setback and clearance. Alternate setbacks and clearances are permitted, subject to the approval of the building official. The building official ~~is~~ shall be permitted to require ~~an~~ a geotechnical investigation as set forth in Section 1803.5.10. ~~and recommendation of a registered design professional to demonstrate that the intent of this section has been satisfied. Such an investigation shall include consideration of material, height of slope, slope gradient, load intensity and erosion characteristics of slope material.~~

1808.2.2 General. Pier and pile foundations shall be designed and installed on the basis of a ~~foundation~~ geotechnical investigation as set forth ~~defined~~ in Section ~~1802~~ 1803. ~~unless sufficient data upon which to base the design and installation is available. The investigation and report provisions of Section 1802 shall be expanded to include, but not be limited to, the following:~~

1. ~~Recommended pier or pile types and installed capacities.~~
2. ~~Recommended center-to-center spacing of piers or piles.~~
3. ~~Driving criteria.~~
4. ~~Installation procedures.~~
5. ~~Field inspection and reporting procedures (to include procedures for verification of the installed bearing capacity where required).~~
6. ~~Pier or pile load test requirements.~~
7. ~~Durability of pier or pile materials.~~

1808.2.8.4 Allowable frictional resistance. The assumed frictional resistance developed by any pier or uncased cast-in-place pile shall not exceed one-sixth of the bearing value of the soil material at minimum depth as set forth in Table 1804.2, up to a maximum of 500 psf (24 kPa), unless a greater value is allowed by the building official after a geotechnical ~~soil~~ investigation, as specified in Section 1802, is submitted or a greater value is substantiated by a load test in accordance with Section 1808.2.8.3. Frictional resistance and bearing resistance shall not be assumed to act simultaneously unless ~~recommended~~ determined by a ~~soil~~ geotechnical investigation ~~as specified~~ in accordance with Section 1802.

1808.2.10 Use of higher allowable pier or pile stresses. Allowable stresses greater than those specified for piers or for each pile type in Sections 1809 and 1810 are permitted where supporting data justifying such higher stresses is filed with the building official. Such substantiating data shall include:

1. A ~~soils~~ geotechnical investigation in accordance with Section ~~1802~~ 1803.
2. Pier or pile load tests in accordance with Section 1808.2.8.3, regardless of the load supported by the pier or pile.

The design and installation of the pier or pile foundation shall be under the direct supervision of a registered design professional knowledgeable in the field of soil mechanics and pier or pile foundations who shall certify to the building official that the piers or piles as installed satisfy the design criteria.

3304.1.4 Fill supporting foundations. Fill to be used to support the foundations of any building or structure shall comply with Section ~~1803.5~~ 1804.5. Special inspections of compacted fill shall be in accordance with Section 1704.7.

J101.1 Scope. The provisions of this chapter apply to grading, excavation and earthwork construction, including fills and embankments. Where conflicts occur between the technical requirements of this chapter and the ~~soils~~ geotechnical report, the ~~soils~~ geotechnical report shall govern.

J104.3 Soils report. A ~~soils~~ geotechnical report prepared by a registered design ~~professionals~~ professional shall be provided. The report ~~which~~ shall ~~identify~~ contain at least the following:

1. The nature and distribution of existing soils;
2. Conclusions and recommendations for grading procedures;
3. Soil design criteria for any structures or embankments required to accomplish the proposed grading; and,
4. Where necessary, slope stability studies, and recommendations and conclusions regarding site geology.

Exception: A ~~soils~~ geotechnical report is not required where the building official determines that the nature of the work applied for is such that a report is not necessary.

J106.1 Maximum slope. The slope of cut surfaces shall be no steeper than is safe for the intended use, and shall be no steeper than 2 horizontal to 1 vertical (50 percent) unless the ~~applicant~~ owner or authorized agent furnishes a ~~soils~~ geotechnical report justifying a steeper slope.

Exceptions:

1. A cut surface may be at a slope of 1.5 horizontal to 1 vertical (67 percent) provided that all the following are met:
   1.1. It is not intended to support structures or surcharges.
   1.2. It is adequately protected against erosion.

1.3. It is no more than 8 feet (2438 mm) in height.
1.4. It is approved by the building official.
1.5. Ground-water is not encountered.
2. A cut surface in bedrock shall be permitted to be at a slope of 1 horizontal to 1 vertical (100 percent).

J107.1 General. Unless otherwise recommended in the ~~soils~~ geotechnical report, fills shall comply with the ~~conform to~~ provisions of this section.

J107.6 Maximum slope. The slope of fill surfaces shall be no steeper than is safe for the intended use. Fill slopes steeper than 2 horizontal to 1 vertical (50 percent) shall be justified by ~~soils~~ a geotechnical report ~~reports~~ or engineering data.

Reason: Code update and clarification.

Reorganizes and clarifies the sections related to geotechnical investigations and excavation, grading and fill. Provides consistent use of "geotechnical" as related to investigations and reports.

Section 1802.1 allows the building official to require that investigations be conducted by a registered design professional (RDP), but does NOT allow the building official to remove such a requirement that appears elsewhere. Several sections of the code do require investigations by a RDP. The text of 1802.1 is revised to clarify those requirements. Section 1802.4.1 requires that a RDP establish the scope of investigations that involve borings and soundings, drilling and sampling, in-situ testing, and laboratory testing. Since the purpose of borings, soundings, drilling, and sampling is related to "in-situ testing, laboratory testing, or engineering calculations", the scope is slightly revised by use of those terms in new Section 1803.1. Several sections outside 1802 set forth requirements for geotechnical investigations; those items are all collected and coordinated in this proposal.

Where excavation will remove lateral support for a foundation, current Section 1803.1 requires underpinning or protection against settlement or lateral translation. In practice, compliance requires a geotechnical investigation. Section 1803.5.7 is added to reflect that reality.

The addition of a tenth item in the section on reporting relates to an item that already requires investigation.

The requirements related to seismic design categories are recast (and slightly revised) for better agreement with Section 11.8.2 and 11.8.3 of ASCE 7-05.

The change is made to Section J106.1 because cuts below the ground-water table are less stable than those above.

Bibliography:
Composite of Chapter 18 reorganization assuming all of proponent's proposals are approved.

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

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

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1804, 1804.1, 1804.2, Table 1804.2, 1804.3, 1804.3.1, 1806.3.2, 1806.3.3, 1806.3.4

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

Revise as follows:

SECTION ~~1804~~ 1806
~~ALLOWABLE~~ PRESUMPTIVE LOAD-BEARING VALUES OF SOILS

~~1804.1 Design.~~ 1806.1 Load combinations. The presumptive load-bearing values provided in Table ~~1804.2~~ 1806.2 shall be used with the allowable stress design load combinations specified in Section 1605.3. The values of vertical foundation pressure and lateral bearing pressure given in Table 1806.2 shall be permitted to be increased by one-third where used with the alternative basic load combinations of Section 1605.3.2 that include wind or earthquake loads.

~~1804.2~~ 1806.2 Presumptive load-bearing values. The ~~maximum allowable foundation pressure, lateral pressure or lateral sliding-resistance~~ load-bearing values used in design for supporting soils near the surface shall not exceed the values specified in Table ~~1804.2~~ 1806.2 unless data to substantiate the use of ~~a~~ higher ~~value~~ values are submitted and approved. Where the building official has reason to doubt the classification, strength, or compressibility of the soil, the requirements of Section 1802.2.1 shall be satisfied.

Presumptive load-bearing values shall apply to materials with similar physical characteristics and dispositions. Mud, organic silt, organic clays, peat or unprepared fill shall not be assumed to have a presumptive load-bearing capacity unless data to substantiate the use of such a value are submitted.

Exception: A presumptive load-bearing capacity ~~is~~ shall be permitted to be used where the building official deems the load-bearing capacity of mud, organic silt or unprepared fill is adequate for the support of lightweight and temporary structures.

TABLE ~~1804.2~~ 1806.2
~~ALLOWABLE FOUNDATION AND LATERAL PRESSURE~~
PRESUMPTIVE LOAD-BEARING VALUES

CLASS OF MATERIALS	~~ALLOWABLE~~ VERTICAL FOUNDATION PRESSURE (psf)~~[d]~~	LATERAL BEARING PRESSURE (psf/f below natural grade)~~[d]~~	LATERAL SLIDING RESISTANCE	
			Coefficient of friction[a]	Cohesion ~~Resistance~~ (psf)[b]
1. Crystalline bedrock	12,000	1,200	0.70	—
2. Sedimentary and foliated rock	4,000	400	0.35	—
3. Sandy gravel and/or gravel (GW and GP)	3,000	200	0.35	—
4. Sand, silty sand, clayey sand, silty gravel and clayey gravel (SW, SP, SM, SC, GM and GC)	2,000	150	0.25	—
5. Clay, sandy clay, silty clay, clayey silt, silt and sandy silt (CL, ML, MH and CH)	1,500~~[c]~~	100	—	130

For SI: 1 pound per square foot = 0.0479 kPa, 1 pound per square foot per foot = 0.157 kPa/m.
a. Coefficient to be multiplied by the dead load.
b. ~~Lateral sliding resistance~~ Cohesion value to be multiplied by the contact area, as limited by Section 1806.3.2 ~~1804.3~~.
~~c. Where the building official determines that in-place soils with an allowable bearing capacity of less than 1,500 psf are likely to be present at the site, the allowable bearing capacity shall be determined by a soils investigation.~~
~~d. An increase of one-third is permitted when using the alternate load combinations in Section 1605.3.2 that include wind or earthquake loads.~~

~~1804.3~~ 1806.3 Lateral ~~sliding~~ load resistance. Where the presumptive values of Table 1806.2 are used to determine resistance to lateral loads, the calculations shall be in accordance with Sections 1806.3.1 through 1806.3.4.

1806.3.1 Combined resistance. The total resistance ~~of structural walls~~ to lateral loads ~~sliding~~ shall be permitted to be determined ~~calculated~~ by combining the values derived from the lateral bearing pressure and the lateral sliding resistance specified ~~shown~~ in Table ~~1804.2~~ 1806.2 ~~unless data to substantiate the use of higher values are submitted for approval.~~

1806.3.2 Lateral sliding resistance limit. For clay, sandy clay, silty clay, ~~and~~ clayey silt, silt and sandy silt, in no case shall the lateral sliding resistance exceed one-half the dead load.

~~**1804.3.1 Increases in allowable lateral sliding resistance.** The resistance values derived from the table are~~
1806.3.3 Increase for depth. The lateral bearing pressures specified in Table 1806.2 shall be permitted to be increased by the tabular value for each additional foot (305 mm) of depth to a maximum of 15 times the tabular value.

1806.3.4 Increase for poles. Isolated poles for uses such as flagpoles or signs and poles used to support buildings that are not adversely affected by a 0.5 inch (12.7 mm) motion at the ground surface due to ~~short-term lateral~~ wind or earthquake loads ~~are~~ shall be permitted to be designed using ~~lateral-bearing values~~ lateral bearing pressures equal to two times the tabular values.

Reason: Code clarification. Changes are editorial. Moves the footnote (d) concerning load combinations to the introductory section with the same scope. Recasts the content of footnote c by referring to the broader requirement in Section 1802.2.1. Makes the terminology more consistent throughout the section. Removes the unnecessary text concerning substantiation of higher values, which is already covered in Section 1802.2.1 and new Section 1806.2. Fixes the apparent oversight of two soil types that appear in item 5 of the table for the list in the section limiting lateral sliding resistance. Clarifies that "short-term lateral loads" are "wind or earthquake loads".
Bibliography:
Composite of Chapter 18 reorganization assuming all of proponent's proposals are approved.

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

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