

S90-09/10
1609.1.2.2, Chapter 35

Proposed Change as Submitted

Proponent: John Woestman, The Kellen Company, representing the Door Safety Council (DSC)

1. Add new text as follows:

1609.1.2.2 Side-hinged doors. Side-hinged door glazed opening protection for wind-borne debris shall meet the requirements of an *approved* impact-resistant standard or ANSI A250.13.

(Renumber remaining sections)

2. Add standard to Chapter 35 as follows:

ANSI

A250.13-08 Testing and Rating of Severe Windstorm Resistant Components for Swinging Door Assemblies

Reason: This proposal helps resolve performance and code compliance issues when exterior side-hinged door openings are comprised of components from multiple sources and include interchangeable elements (ie: doors, frames, hinging and latching hardware, etc.).

This proposed change allows an alternative method to demonstrate performance to impact-resistant requirements for side-hinged door openings by requiring components to be tested to ANSI A250.13-2008. ANSI A250.13 contains language that prescribes how components are to be selected to create complete door openings expected to perform equivalently to door assemblies tested to ASTM E 1996 / E 1886 for impact resistance.

Through the ANSI standards development process stake-holders comprising most major manufacturing associations, testing and certification organizations, specifiers, code officials and end users, developed a national standard for a component-based approach to testing for windstorm resistance of swinging door openings. The test procedures used in this standard represent the most severe requirements found in the windstorm resistance standards referenced in today's building codes. These procedures are designed to isolate the loads, conditions and critical performance requirements that a particular component is subjected to in full assembly tests and duplicate these specific conditions. Using a combination of worst-case scenario design and safety factors, this standard is designed to provide a component rating that relates directly to the component's ability to withstand the conditions that occur in full assembly tests.

Prior to releasing the current revision of ANSI A250.13, validation tests of the large missile impact test specified by ASTM E1886/E1996 were conducted through Intertek Testing Services, a Nationally Recognized Test Laboratory. The study was conducted to quantify the energy that would tend to shear the latch bolt in assembly tests and compare it to the energy delivered to the latch bolt in the ANSI A250.13 component test procedure which uses a relatively rigid fixture and a pendulum type impactor. The impact energy applied to the test sample was varied and the actual energy imparted to the lock and hinge was measured. The component test fixture is more efficient at transferring the energy applied to the system into the test samples than the ASTM E1996 assembly test fixture. This results in higher impact energy at the lock or hinge. For example, only 4% of the impact energy applied in the ASTM E1996 test transfers to the lock. Whereas, 15% of the impact energy is delivered to the lock mounted in the A250.13 test fixture.

Results demonstrated that this test specified in ANSI A250.13 for latches was indeed much more severe (approximately 3.75 times more) than the exposure provided in door assembly tests conducted per ASTM E1996 and similar wind borne debris impact tests. The current impact test requirements of ANSI A250.13 were therefore adjusted to be two times more severe (maintaining a 2 times safety factor) to the current requirements of ASTM E1996.

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 A250 13-08, 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 September 24, 2009.

ICCFILENAME: WOESTMAN-S1-1609.1.2.2

Public Hearing Results

Note: The following analysis was not in the Code Change monograph but was published on the ICC website at <http://www.iccsafe.org/cs/codes/Documents/2009-10cycle/ProposedChanges/Standards-Analysis.pdf> :

Analysis: Review of proposed new standard ANSI A250.12 indicated that, in the opinion of ICC Staff, the standard complies with ICC standards criteria.

Committee Action:

Approved as Submitted

Committee Reason: With the addition of ANSI A250.12 to regulate the parts of a side-hinged door, there will be at least a requirement for their testing. It can be better to have tests on each part of the assembly. This component approach is not a novel idea, but is something that is done all the time. There is a consensus standard and it's a good option to have in the code.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment 1:

Jeff Inks, Window and Door Manufacturers Association, requests Disapproval.

Commenter's Reason: After subsequent consideration of the Structural Committee's approval of this code change proposal, WDMA believes it should be disapproved.

While as the committee pointed out in their reason for approving the proposal, the "*component approach is not a novel idea*," it has not evolved to the point that it can be relied upon to the extent intended by this proposal which is to ensure side-hinged door assemblies constructed of components that have been tested, but not together as an assembly, are compliant with the performance requirements for these assemblies in wind borne debris regions. ANSI 250.13 allows substitutions that go far beyond what is allowed in the ASTM E-1996 standard.

While we are also supportive of the concept, more guidance should be in place with respect to component substitution and the extent to which it can be safely employed in the construction of assemblies that are not tested as assemblies. That guidance should be available before provisions such as those proposed by S90 are approved for the code.

Public Comment 2:

Julie Ruth, JRuth Code Consulting, representing American Architectural Manufacturers Association, requests Disapproval.

Commenter's Reason: S90 would require glazed opening protection in side hinged doors to meet the requirements of an approved impact resistant standard, or ANSI A250.13. Since glazed opening protections are already required to meet the requirements of an approved impact resistant standard elsewhere in Section 1609.1.2, the net effect of S90 would be to permit the alternate use of ANSI A250.13 to determine the impact resistance of components of side hinged doors rather than testing complete systems as required by other impact resistance standards.

It has been AAMA's experience that the testing and rating of individual components of a fenestration assembly, without testing a full assembly to establish a baseline, does not provide adequate information on the performance of the resultant assembly under load. This is as true for resistance to impact load as it is for resistance to design wind pressure.

During the testimony on S90 at the code development hearings in Baltimore, the proponents of S90 pointed to the fact that the standard requires the door slab to be stiffened prior to testing, to more fully impart the impact load onto the hardware that is securing the door slab into the opening. While such stiffening may in fact provide a more meaningful test of the hardware, it almost certainly does not give an accurate view of the performance of the door slab itself. There is also some question as to whether the most appropriate test for the hardware is in fact also the most appropriate for the door slab. Perhaps two different test methods, or two different methods of preparing the test assembly, should be used.

AAMA also has a concern that sections 9.1.1 and 9.1.2 of ANSI A250.13 permit alternate means of testing glazing in impact rated doors. Section 9.1.1 specifies that the glazing is to be tested in the largest size that can accommodate the glazing system. Section 9.1.2 permits the glazing system to be tested as part of door assemblies that are defined elsewhere in the standard, which are not required to include glazing in the largest size that can be accommodated. The members of the AAMA Door Council know from years of testing that these glazing options will react differently in different door slab and framing systems (assemblies). Therefore, permitting alternate means of testing the glazing options will not result in consistent test results, or consistent product performance.

Finally, it should be noted that validation test data from proponents of ANSI A250.13 has not been made available for review. Confirmation of the validity of a proposed new testing and rating method, either through peer review, round robin testing, or some other method of verifying the validity of the results, is the hallmark of meaningful standards development. The need for it should not be dismissed or lightly set aside.

Considering the importance of impact protection in maintaining the integrity of the building envelope, it is prudent that building codes remain conservative in their approach to specifying the means of qualifying impact protective products. Allowing a method such as A250.13, that does not require a full assembly test, is not a conservative approach. We urge disapproval of the use of this method, as provided for in S90.

Public Comment 3:

Larry J. Tanner P.E., Texas Tech University, representing Wind Sciences & Engineering Center, Debris Impact Test Laboratory, requests Disapproval.

Commenter's Reason: The ANSI A250 Standard, along with the ASTM 1886/1996 standards, were developed to prevent the proliferation of envelope perforations and the resulting inundation of rainwater from hurricane events. Evidence from hurricane investigations has revealed that indeed buildings designed to these standards performed better than buildings without said protection. However, it should be understood and specifically included in technical specifications by the manufacturers and advertisements to the consumers, that such products are intended only for non-catastrophic property protection from rainwater inundation and not for the protection of building occupants (Life Safety). I was a coauthor of both FEMA 320 and FEMA 361 which utilize Tornado and Hurricane Saferoom Design Wind Speed Maps. Never were the above referenced ANSI and ASTM standards considered suitable for FEMA 320 Saferooms or FEMA 361 Community Shelters.

Specifics to the proposed changes to the ANSI A250.13-2008 Standard:

1. From a quantitative standpoint the "stiffness theory" appears reasonable; however laboratory tests have proven otherwise. Texas Tech University has been the leading "storm debris impact researcher" for over 35 years. Tests on door assemblies have proven that success or failure from wind pressures and debris impacts is unique to the door (or window unit) and the hardware components installed. A heavily constructed door absorbs little energy and directs most of the energy to the attaching components and has proven to fail components that previously passed on other less massive doors. Lighter constructed doors can bend excessively and either pull out locking bolts or cause bolting bending and ultimate failure. Doors passing the impact tests must have a unique set of hardware that matches the door performance, thus doors are rated as a complete assembly, inclusive of frame, door(s), hinging, and locking hardware. Window lites in doors compromise the strength of the door and present another set of unique circumstances which require the unit to be rated as a complete assembly. Window unit performance is unique to the opening size, frame type, and the glazing. The elasticity of the glazing is a function of size and type. Based on size, some glazing is so elastic that it bounces

out of the frame. Smaller is not always better; some glazing will destroy the framing system and be pushed out. Thus, the only way to predict window behavior under impact is by full scale testing in the laboratory in "as specified and installed" condition.

2. Though these Standards were developed for "envelope" protection to reduce rainwater intrusion, these components that are now rated as "Hurricane Tested" are now being used in hardened "Hurricane Shelters" which are intended to protect lives. This is the result of misleading specification sheets, and uninformed dealers and consumers.

3. The "component rating" system does not consider the size of doors or glazed openings; the stiffness of doors with various sizes of lites, nor the quantity of hinges or latches required per size to carry the loads.

4. The Standard requires the component to be rated by ultimate load, but there is no guidance regarding the "assembly rating" based upon mixture of components with various ratings.

5. The Standard does not require engineering review or oversight.

6. According to the Test Procedure stated in Section 5.2.2, the impact energy should be 350 foot-pounds. However, for hinges, Section 6.1.1.2 and Latching Hardware, Section 7.1.1.3, the impact kinetic energy has been reduced to 125 foot-pounds. I understand the "stiffness" theory of the test fixture and product configuration, but this assumes that every laboratory will have the same fixtures and the same laboratory conditions.

7. The wind speed range has changed from 110-150 mph for the 2003 Standard to 110-170 mph for the 2008 Standard, but the test loads and impact criteria has not changed.

8. The impact location 6" above the floor on Figure 4, page 6 is unrealistic. In researching most all of the severe tornadoes and hurricanes since 1989, I have never seen an impact lower than 2.5 feet on a vertical surface.

9. Although there is not the opportunity at this time, to prevent the misuse of these products as describe above, I would suggest that the title of the standard be changed to: *Testing and Rating of Windstorm Resistant Components for Swinging Doors for Non-Life Safety Uses*.

Public Comment 4:

Gordon Thomas P.E., representing self, requests Disapproval.

Commenter's Reason: The ANSI A250.13 standard is a component rating standard not an assembly design standard. By referencing a component standard in the code, it would allow any non-engineering professional to inappropriately assemble components into an unqualified door assembly.

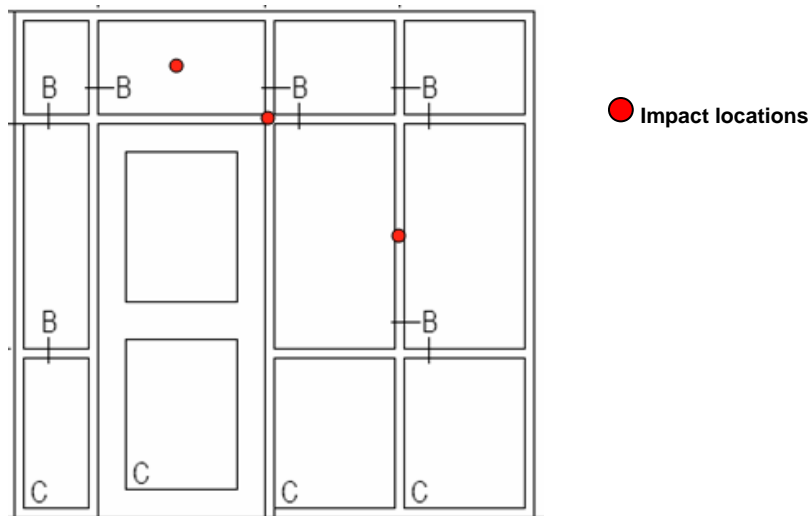
In addition, the proponent's *Reason Statement* for S90 contained the following statement regarding ANSI A250.13; "The test procedures used in this standard represent the most severe requirements found in the windstorm resistance standards referenced in today's building codes."

As a Professional Engineer with extensive experience in design, testing and analysis of debris impact-resistant opening protectives, I have researched this claim and have concluded that this is not an accurate statement. Please consider the following facts;

ANSI A250.13 Section 6.1.1.2 – Hinges, and Section 7.1.1.3 – Latching Hardware, have impact energy requirements reduced to 125 foot-pounds. This represents a 64% reduction from the 350 foot-pounds impact energy requirements of the Florida Building Code or ASTM 1996.

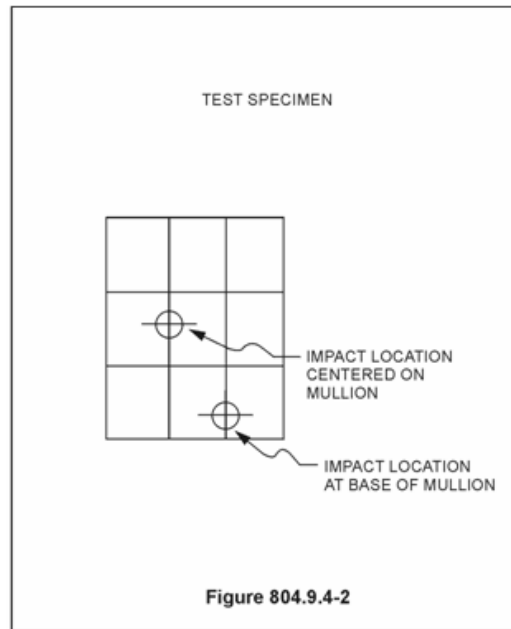
The 2003 edition of A250.13 defined its scope as hurricanes with wind speeds of 110 to 150 MPH. The 2008 edition has expanded its scope to encompass wind speeds up to 170 MPH, while reducing impact energy requirements and the safety factor in section 6.1.3.1

ANSI A250.13 - 2008 section 10.1 prohibits missile impacts to framing members of glazed transom / sidelite openings, as is typically conducted under ASTM 1996, ICC 500 or Florida Building Code protocol TAS 201 testing. A graphic comparison of impact test methods follows;



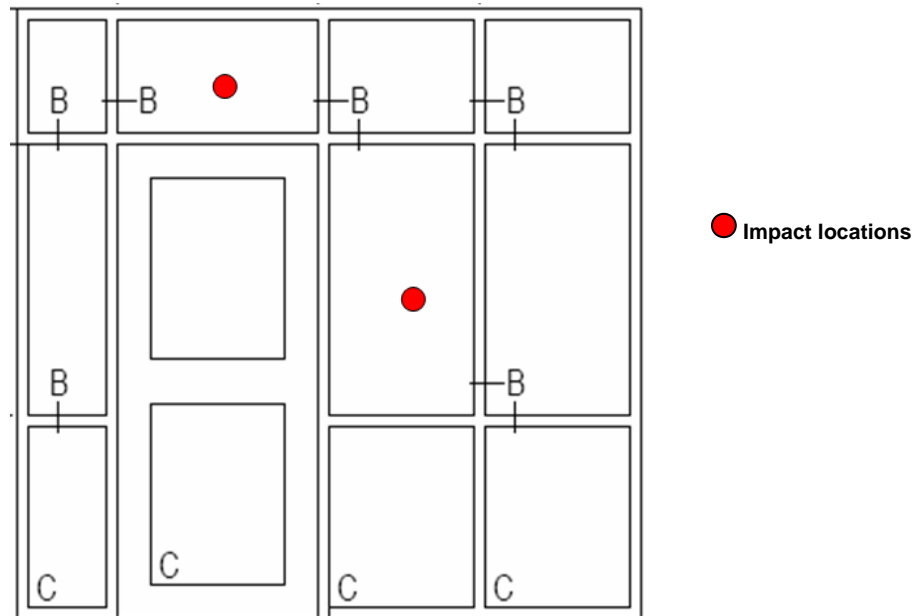
Florida Building Code - TAS 201

6.3.2.2 When testing any specimen with more than one component, in addition to complying with the impacts required by Section 1626.2 of the Florida Building Code, the framing member connecting these components shall be impacted at one half the span of such member with the large missile at a speed indicated in Section 1626.2.4 of the Florida Building Code.



ICC 500

804.9.4 Windows and other Glazed Openings. All window assemblies and other glazed openings shall be impacted in the center of the smallest glazed section, and at one interface corner as detailed in Figure 804.9.4-1. Where interior mullions or other glazed section joints and/or latches are present, additional impacts shall be applied on these features as shown in Figure 804.9.4-2.



ANSI A 250.13

Sidelights and / or Transoms

10.1 Sidelights and/or transoms shall be tested with doors, to the largest total size (maximum area, height and width) to be rated. Testing shall be performed per ASTM E1886 and ASTM E1996 with the impacts applied only to the glazed portion of the assembly

Two similar proposals (S83 and S143 -07/08) to add ANSI A250.13 were disapproved in the last cycle by vote of the membership on September 20, 2008 at the Final Action Hearings in Minneapolis, MN

The Standard does not require engineering calculations validating openings configured from components, leaving the Plans Examiner or AHJ with the responsibility to do so, or the liability for not doing so

The October 30, 2009 issue of ANSI Standards Action identifies for public comment changes made to the A250.13 standard *after the approval of the document*. This matter is still open and pending with ANSI, subject to further review by the ANSI Board of Standards Review

Final Action: AS AM AMPC_____ D

S94-09/10 1612.6 (New), Chapter 35

Proposed Change as Submitted

Proponent: Michael Mahoney, Federal Emergency Management Agency, representing the National Tsunami Hazard Mitigation Program

1. Add a new text as follows:

1612.6 Tsunami-generated flood hazard. Construction within a Tsunami Hazard Inundation Zone shall be in accordance with this section.

1612.6.1 Definitions. The following words and terms shall, for the purposes of this section, have the meanings shown herein.

TSUNAMI HAZARD INUNDATION MAP. A map that designates the extent of inundation by a design event tsunami which is developed and provided to a community by either the State or the National Atmospheric and Oceanic Administration (NOAA) under the National Tsunami Hazard Mitigation Program, using NOAA mapping criteria.

TSUNAMI HAZARD INUNDATION ZONE. The area anticipated to be flooded or inundated by a design event tsunami as identified on a community's Tsunami Hazard Inundation Map.

1612.6.2 Establishment of Tsunami Hazard Inundation Zone. Where a community has adopted a Tsunami Hazard Inundation Map, that map shall be used to establish a community's Tsunami Hazard Inundation Zone.

1612.6.3 Construction within the Tsunami Hazard Inundation Zone. Buildings and structures designated Occupancy Category III or IV in accordance with Section 1604.5 shall be prohibited within a Tsunami Hazard Inundation Zone.

Exception: A vertical evacuation tsunami refuge shall be permitted to be located in a Tsunami Hazard Inundation Zone provided it is constructed in accordance with FEMA P646.

2. Add standard to Chapter 35 as follows:

Federal Emergency Management Agency
P646- 08 *Guidelines for Design of Structures for Vertical Evacuation from Tsunamis*

Reason: For coastal communities subject to tsunami waves, where the either the State or the National Oceanic and Atmospheric Administration (NOAA) have provided a Tsunami Hazard Inundation Map and that community has adopted that Map, the Map specifies a Tsunami Hazard Inundation Zone. This Zone is subject to inundation in a design event tsunami, which can result in significant damage. Most of these maps are deterministic in nature, using historical and best available scientific data, and it is currently difficult to assign a specific probability to the design event used for mapping purposes. However, given the potentially serious life safety risk presented to structures within this zone, this is sufficient justification to limit the presence of high hazard and high occupancy structures within the Zone.

Cost Impact: The potential cost impact would be requiring new high hazard and high occupancy structures to be located outside the Tsunami Inundation Zone. Given that this land is further away from the shore and therefore normally less expensive, the cost impact is believed to be minimal.

Analysis: A review of the standard(s) proposed for inclusion in the code, FEMA P646-08, 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 September 24, 2009.

ICCFILENAME: MAHONEY-S2-1612.6 NEW

Public Hearing Results

Note: The following analysis was not in the Code Change monograph but was published on the ICC website at <http://www.iccsafe.org/cs/codes/Documents/2009-10cycle/ProposedChanges/Standards-Analysis.pdf> :

Analysis: Review of proposed new standard, FEMA P646, indicated that, in the opinion of ICC Staff, the standard did not comply with ICC standards criteria, Section 3.6.2(1) Mandatory language, 3.6.3(2) Consensus process.

Committee Action:

Approved as Modified

Modify the proposal as follows:

~~1612.6 Tsunami-generated flood hazard. Construction within a Tsunami Hazard Inundation Zone shall be in accordance with this section.~~

APPENDIX L
TSUNAMI-GENERATED FLOOD HAZARD

L101.1 General. The purpose of this appendix is to provide tsunami regulatory criteria for those communities that have a tsunami hazard and have elected to develop and adopt a map of their tsunami hazard inundation zone.

~~1612.6.1~~ **L101.2 Definitions.** The following words and terms shall, for the purposes of this ~~section~~ appendix, have the meanings shown herein.

TSUNAMI HAZARD INUNDATION MAP. A map that designates the extent of inundation by a design event tsunami which is developed and provided to a community by either the State or the National Atmospheric and Oceanic Administration (NOAA) under the National Tsunami Hazard Mitigation Program, using NOAA mapping criteria.

TSUNAMI HAZARD INUNDATION ZONE. The area anticipated to be flooded or inundated by a design event tsunami as identified on a community's Tsunami Hazard Inundation Map.

~~1612.6.2~~ **L101.3 Establishment of Tsunami Hazard Inundation Zone.** Where a community has adopted a Tsunami Hazard Inundation Map, that map shall be used to establish a community's Tsunami Hazard Inundation Zone.

~~1612.6.3~~ **L101.4 Construction within the Tsunami Hazard Inundation Zone.** Buildings and structures designated Occupancy Category III or IV in accordance with Section 1604.5 shall be prohibited within a Tsunami Hazard Inundation Zone.

Exception: A vertical evacuation tsunami refuge shall be permitted to be located in a Tsunami Hazard Inundation Zone provided it is constructed in accordance with FEMA P646.

(Portions of proposal not shown are unchanged)

Committee Reason: This code change provides a good start, giving guidance on tsunami hazards. The modification places the provisions in an appendix, making them available for jurisdictions to adopt them.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment:

Michael Mahoney, FEMA, representing National Tsunami Hazard Mitigation Program, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

APPENDIX L
TSUNAMI-GENERATED FLOOD HAZARD

L101.1 General. The purpose of this appendix is to provide tsunami regulatory criteria for those communities that have a tsunami hazard and have elected to develop and adopt a map of their tsunami hazard inundation zone.

L101.2 Definitions.

TSUNAMI HAZARD INUNDATION ZONE MAP. A map adopted by the community that designates the extent of inundation by a design event tsunami. This map shall be based the tsunami inundation map which is developed and provided to a community by either the applicable State agency or the National Atmospheric and Oceanic Administration (NOAA) under the National Tsunami Hazard Mitigation Program, using NOAA mapping criteria but shall be permitted to utilize a different probability or hazard level.

TSUNAMI HAZARD INUNDATION ZONE. The area ~~anticipated to be~~ vulnerable to being flooded or inundated by a design event tsunami as identified on a community's Tsunami Hazard ~~Inundation Zone~~ Inundation Zone Map.

L101.3 Establishment of Tsunami Hazard Inundation Zone. Where applicable, if a community has adopted a Tsunami Hazard Inundation Zone Map, that map shall be used to establish a community's Tsunami Hazard Inundation Zone.

L101.4 Construction within the Tsunami Hazard Inundation Zone. Construction of structures designated Occupancy Category III and IV as specified under Section 1604.5 shall be prohibited within a Tsunami Hazard Inundation Zone.

Exceptions:

1. A vertical evacuation tsunami refuge shall be permitted to be located in a Tsunami Hazard Inundation Zone provided it is constructed in accordance with FEMA P646.
2. Community critical facilities shall be permitted to be located within the Tsunami Hazard Zone when such a location is necessary to fulfill their function, providing suitable structural and emergency evacuation measures have been incorporated..

Commenter's Reason: A subsequent review by the State representatives to the National Tsunami Hazard Mitigation Program (NTHMP) generated several comments, which have been condensed into this single public comment. The most significant of these was that the Tsunami Inundation Maps developed by either the State or the National Oceanic and Atmospheric Administration (NOAA) are generally worst case deterministic maps for emergency evacuation purposes. Those maps may be too severe for the purposes of this appendix, so the language in M101.2 has been modified to decouple the Tsunami Inundation Map from the Tsunami Hazard Zone Map referenced in this appendix so that a community can select a map using a more appropriate hazard level. A second comment was that some communities may have a situation where critical facilities may need to be located in the Tsunami Hazard Zone. Exception #2 was added for this situation, but only if evacuation measures have been incorporated.

Final Action: AS AM AMPC____ D

S97-09/10-PART I

1613.5.1, Figure 1613.5(1) - Figure 1613.5(14)

Proposed Change as Submitted

Proponent: Steven Winkel, FAIA, PE, Kelly Cobeen, PE, SE, and J. Daniel Dolan, PhD, PE, Building Seismic Safety Council (BSSC) of the National Institute of Building Sciences, representing the Federal Emergency Management Agency/BSSC Code Resource Support Committee

PART I – IBC STRUCTURAL

Revise as follows:

1613.5.1 Mapped Acceleration Parameters. The parameters S_S and S_I shall be determined from the 0.2 and 1 s spectral response accelerations shown on Figures 1613.5(1) and 1613.5(2) through 1613.5(14), respectively. Where S_I is less than or equal to 0.04 and S_S is less than or equal to 0.15, the structure is permitted to be assigned to Seismic Design Category A.

2. Delete and substitute as follows:

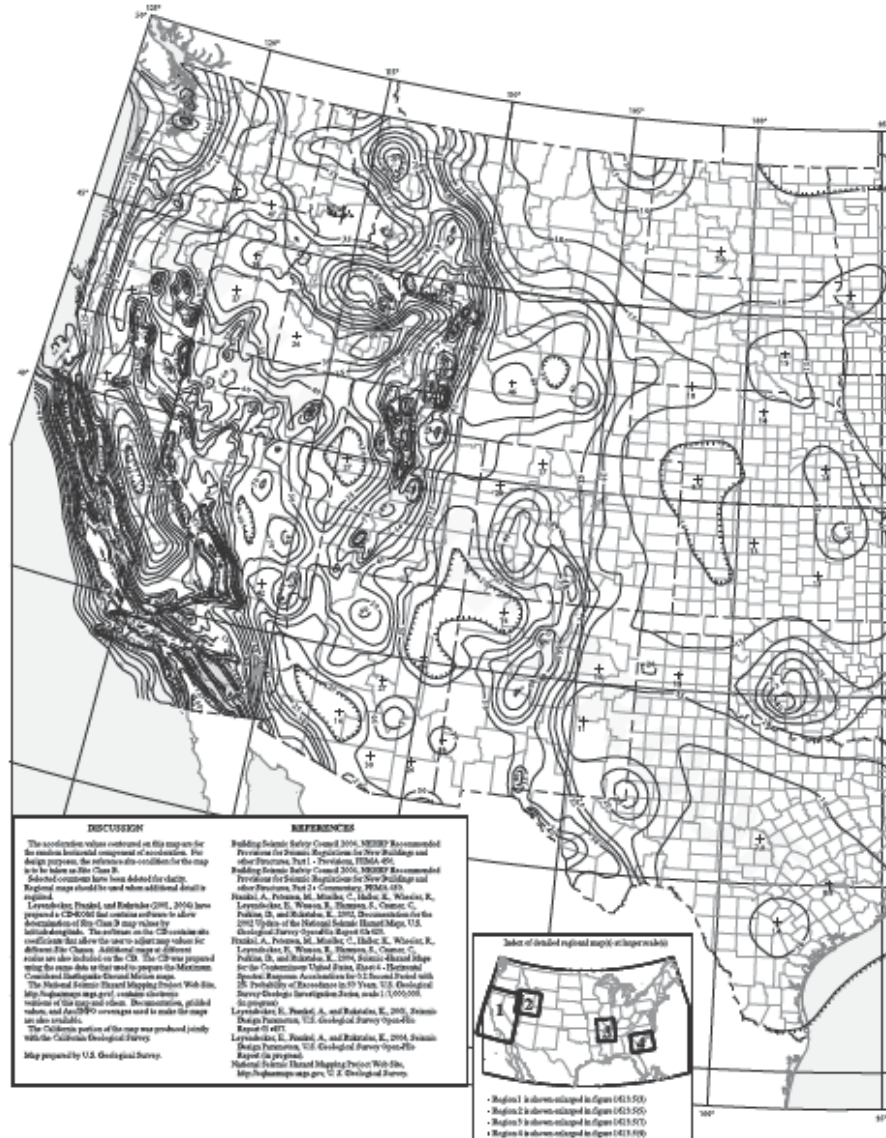


FIGURE 1613.5(1)
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES OF
0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

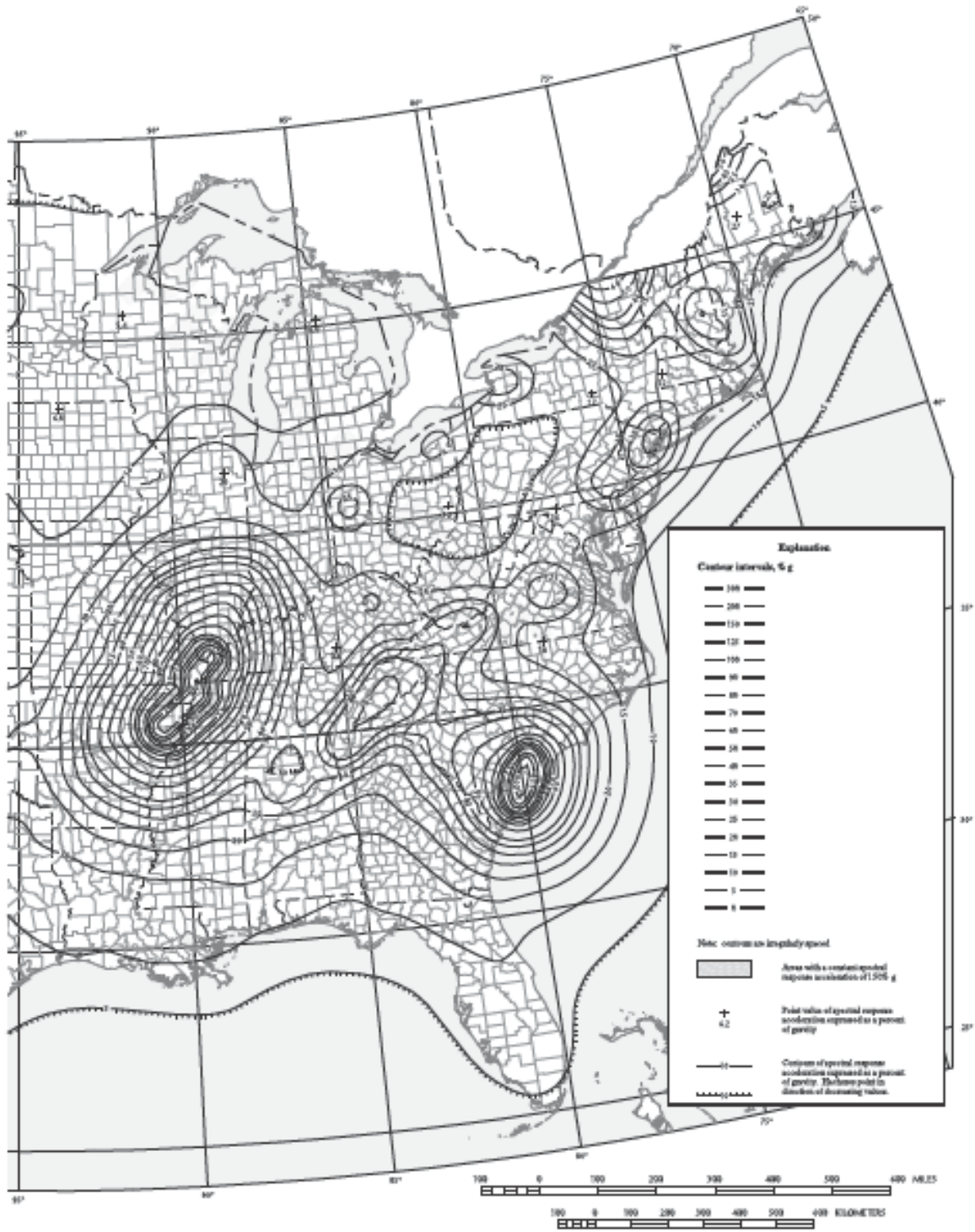


FIGURE 1613.5(1)-continued
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES OF
0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

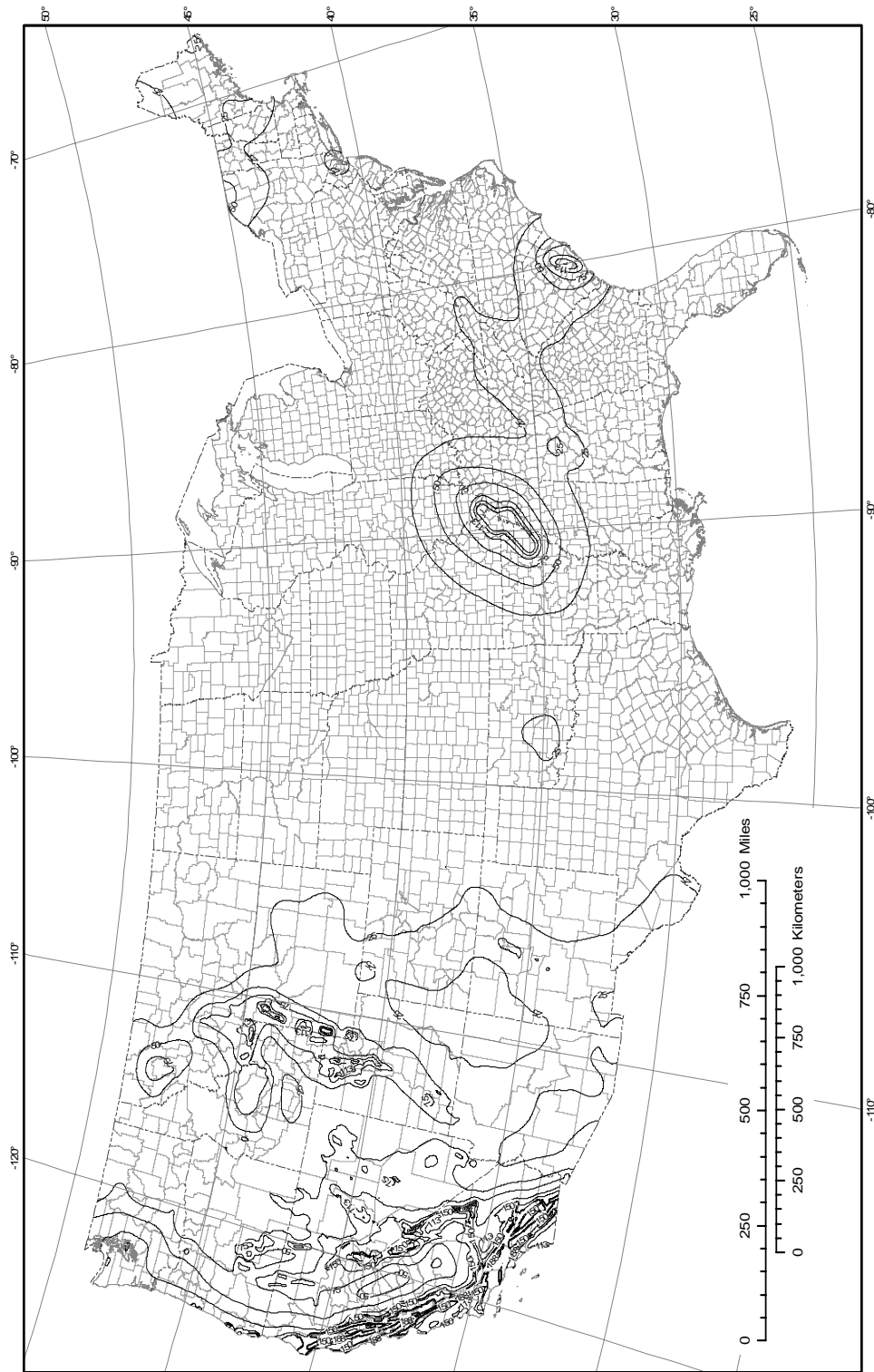
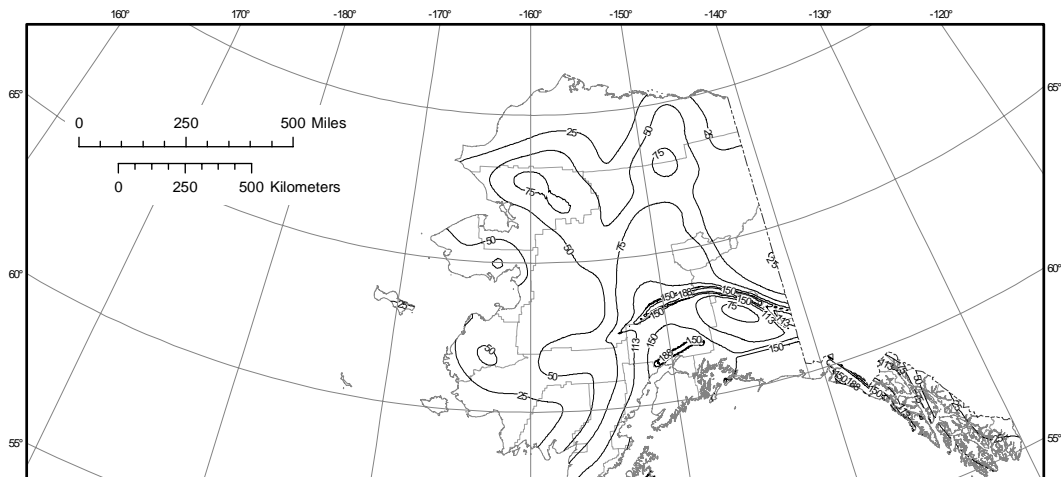
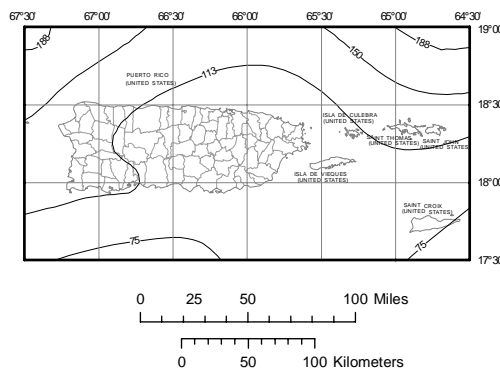
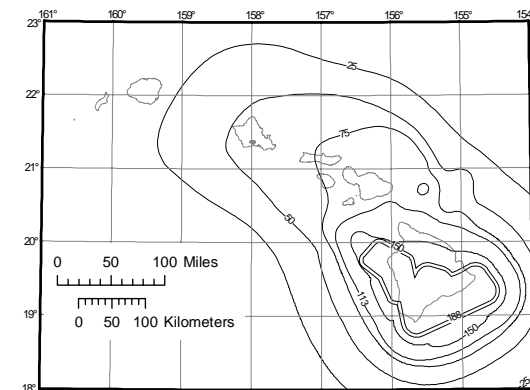
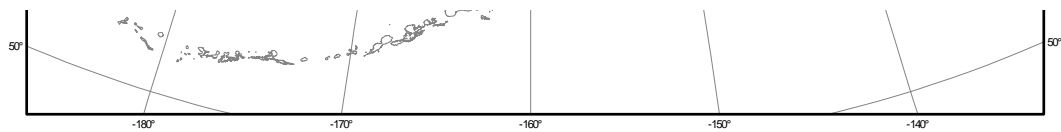


FIGURE 1613.5(1)
MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION OF 0.2 S SPECTRAL RESPONSE
ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



Contours are in units of %g



Notes:

Maps prepared by United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency funded work of the Building Seismic Safety Council (BSSC) and with the American Society of Civil Engineers (ASCE) 7 Seismic Subcommittee.

Ground motion values contoured on these maps incorporate risk-targeted and deterministic ground motions and a factor of 1.1 for the maximum direction of 0.2 s spectral response acceleration. As such, they are different from those on the uniform-hazard-based 2008 USGS National Seismic Hazard Maps posted at <http://earthquake.usgs.gov/research/hazmaps/>.

Larger, more detailed versions of these maps are not provided because it is recommended that a corresponding USGS web tool at <http://earthquake.usgs.gov/research/hazmaps/design/> be used to determine the mapped value for specific locations.

FIGURE 1613.5(1) (CONTINUED)
MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION OF 0.2 S SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

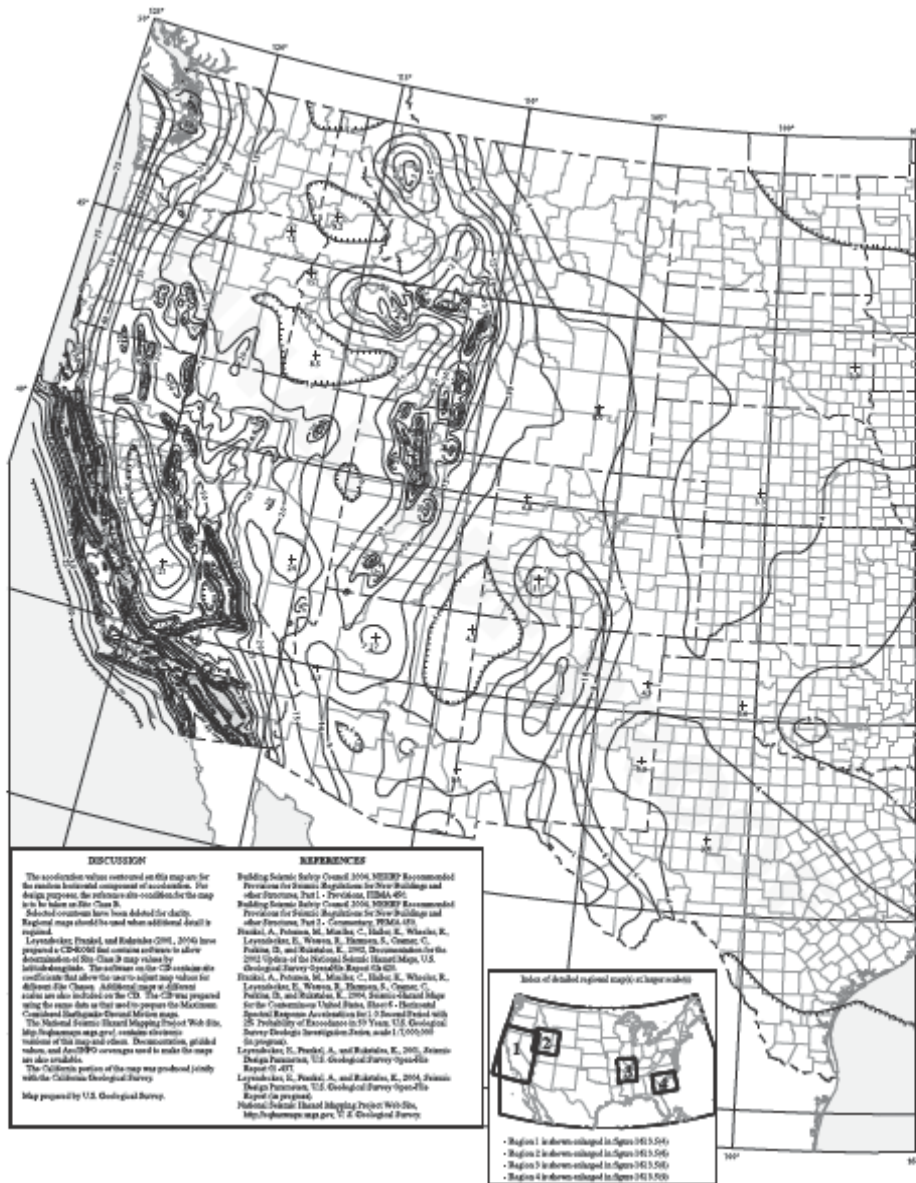


FIGURE 1613.5(2)
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES
OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

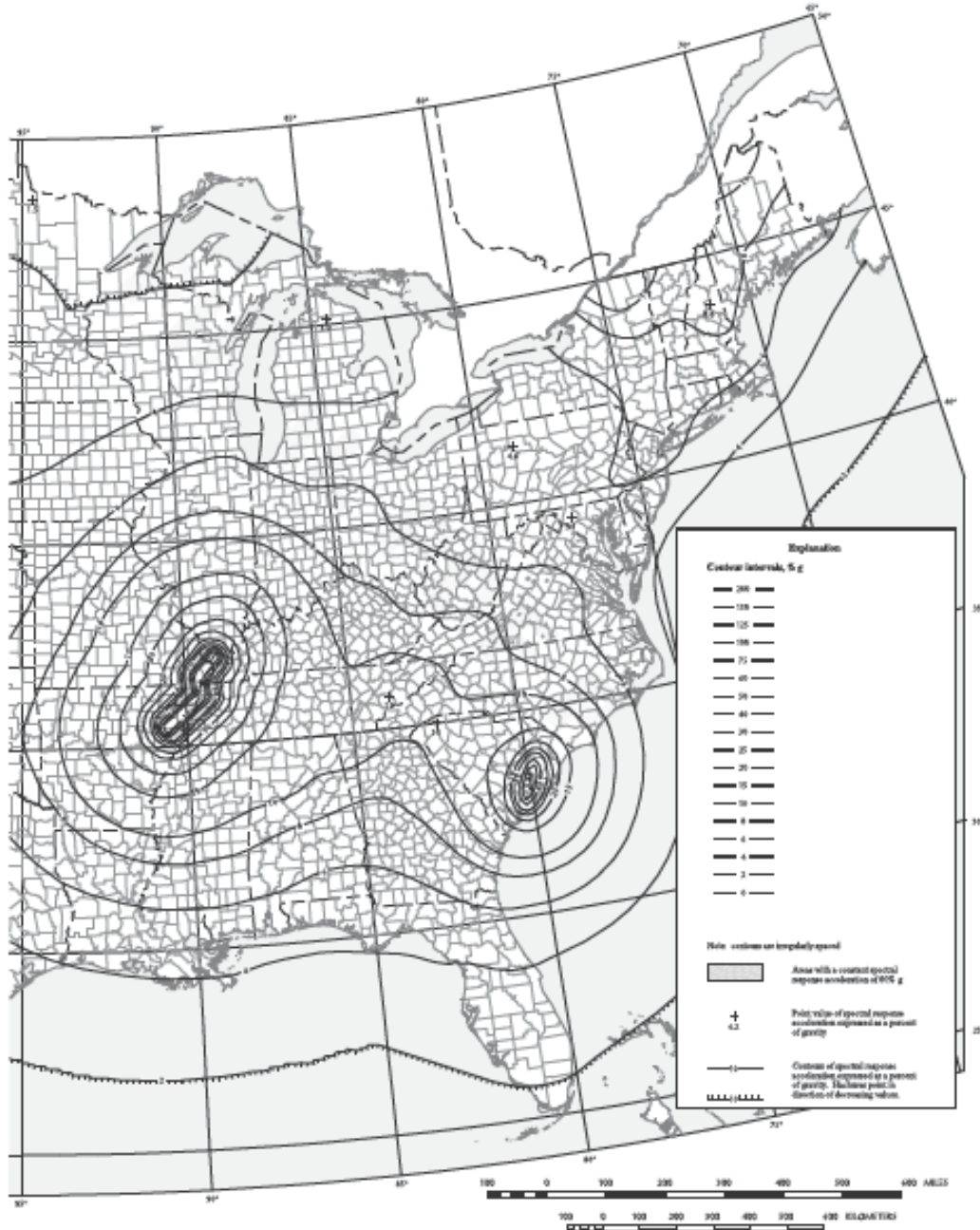


FIGURE 1613.5(2) — continued
MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES
OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

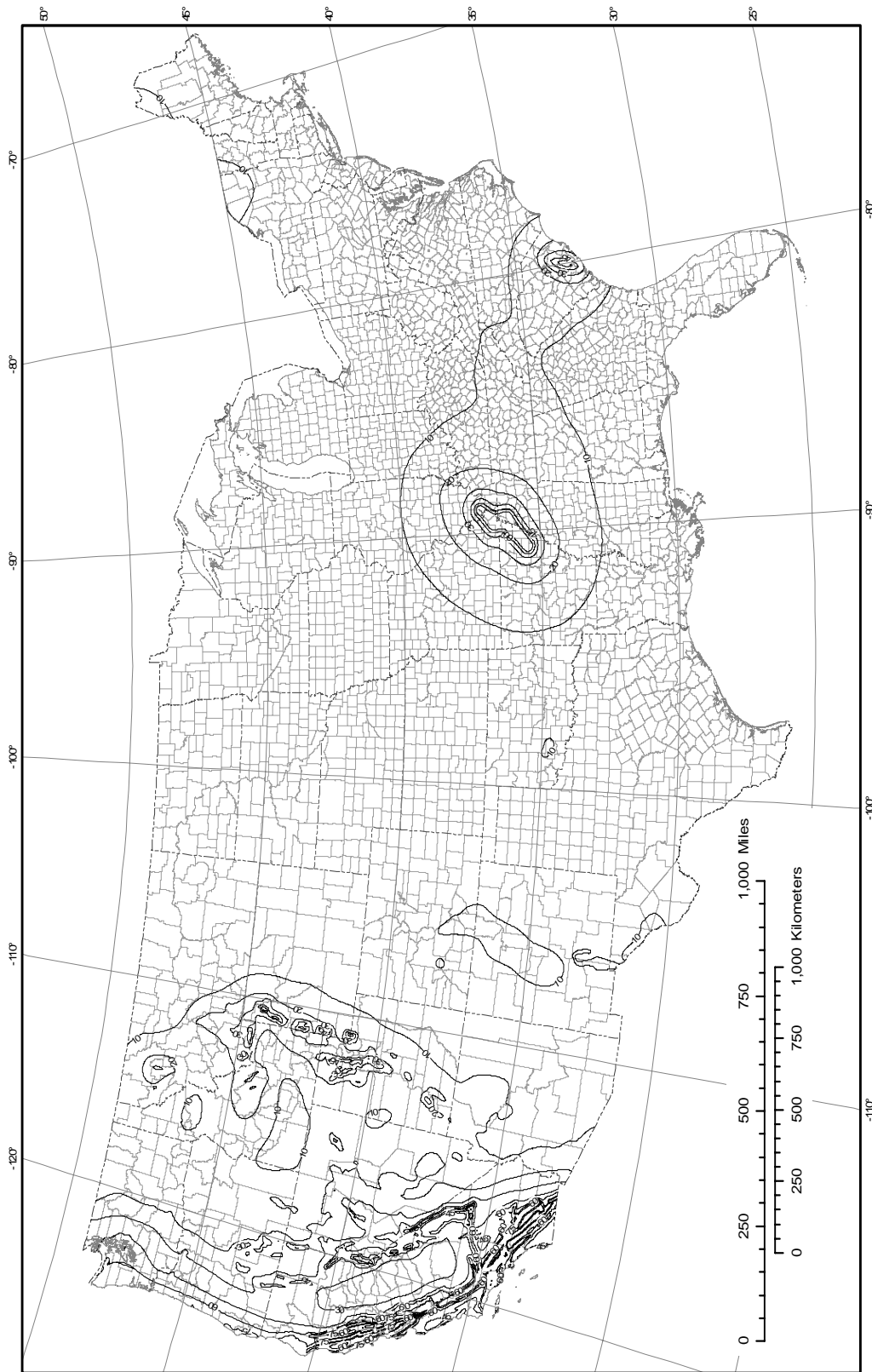
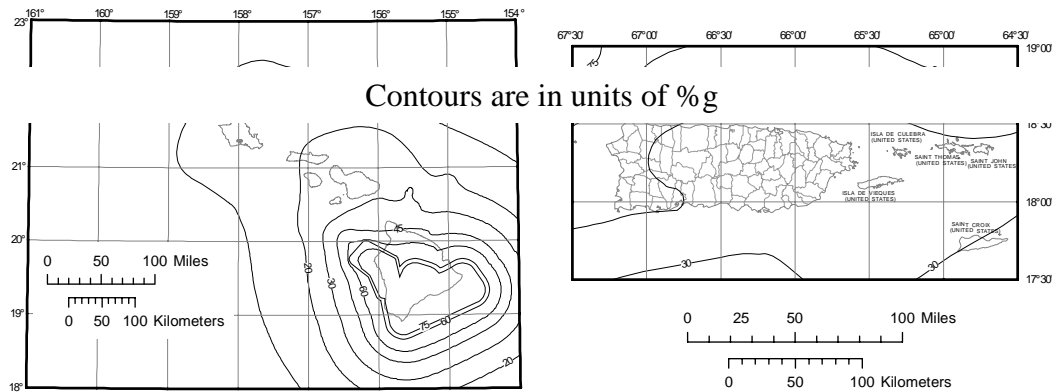
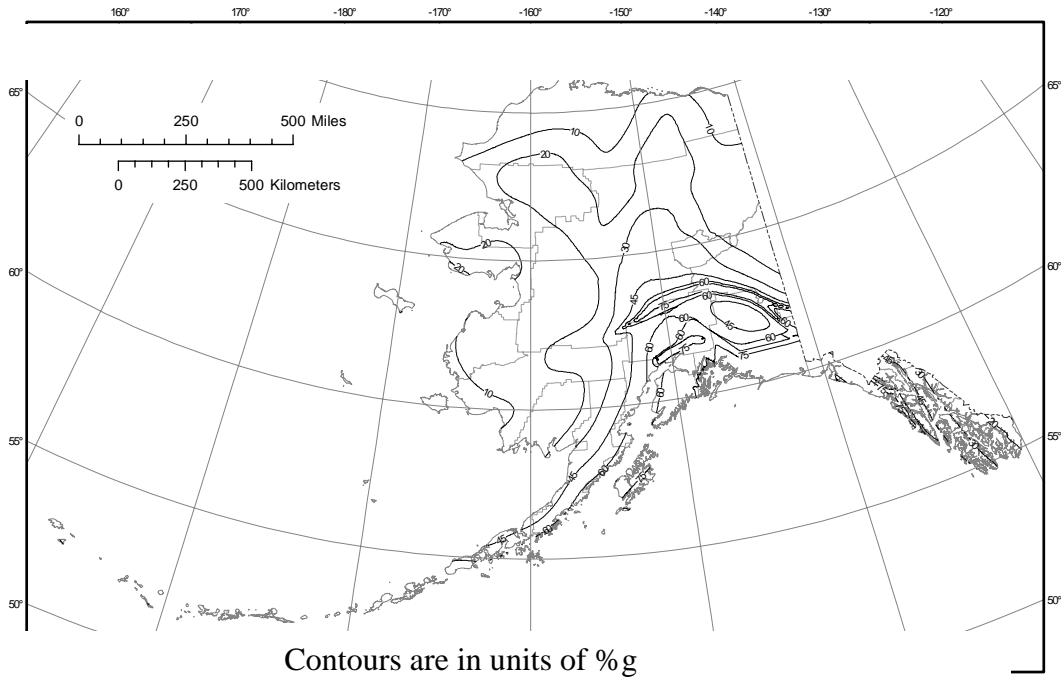


FIGURE 1613.5(2)
MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION OF 1 S SPECTRAL RESPONSE
ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



Notes:

Maps prepared by United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency funded work of the Building Seismic Safety Council (BSSC) and with the American Society of Civil Engineers (ASCE) 7 Seismic Subcommittee. Ground motion values contoured on these maps incorporate risk-targeted and deterministic ground motions and a factor of 1.3 for the maximum direction of 1.0 s spectral response acceleration. As such, they are different from those on the uniform-hazard-based 2008 USGS National Seismic Hazard Maps posted at <http://earthquake.usgs.gov/research/hazmaps/>. Larger, more detailed versions of these maps are not provided because it is recommended that a corresponding USGS web tool at <http://earthquake.usgs.gov/research/hazmaps/design/> be used to determine the mapped value for specific locations.

FIGURE 1613.5(2) (CONTINUED)
MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION OF 1 S SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

Reason: PART I- This proposal incorporates updated earthquake ground motion maps that reflect the 2008 maps developed by the United States Geological Survey (USGS) National Seismic Hazard Mapping Project as well as technical changes adopted for the 2009 *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* (FEMA P750), which was developed by the Building Seismic Safety Council with funding from the Federal Emergency Management Agency. Both projects are part of federal National Earthquake Hazard Reduction Program's (NEHRP) ongoing efforts to make the most current earthquake hazard information available to the building codes. If this code change is not moved forward, the ground motion maps in the IBC will reflect superseded seismic hazard information. The 2008 USGS seismic hazard maps incorporate new information on earthquake sources and ground motion prediction equations including the new Next Generation Attenuation (NGA) relations. The ground motion maps proposed for the IBC further incorporate technical changes adopted for the 2009 *NEHRP Provisions* that include use of: (1) risk-targeted ground motions, (2) maximum direction ground motions, and (3) near-source 84th percentile ground motions.

The proposed ground motion maps for the IBC also reflect a current draft proposal for change to the ASCE 7 (Minimum Design Loads for Buildings and other Structures) standard update process. Under normal circumstances, ASCE 7 would adopt ground motion map related changes drawn from the most current edition of the *NEHRP Recommended Seismic Provisions* prior to incorporation of the maps into the IBC; however, the recent changes to the ICC code development process and schedule have made it necessary to submit this working version of the ASCE 7 proposal in an effort to provide the regulatory community with the most up-to-date information available. It should be understood that, to the extent possible, this proposal will be updated to reflect any modifications to maps, maps titles or other Section 1613 content made during the ASCE 7 consensus standard process so the consistency between ASCE 7 and the IBC is maintained. In the NEHRP update process the title for these maps was revised from "Maximum Considered Earthquake (MCE) Ground Motions" to "Risk-Targeted Earthquake (RTE) Ground Motions." This proposal retains the MCE terminology because it is retained in the working version of the ASCE 7 proposal.

This proposal also reduces the number of printed maps to appear in the IBC from 14 to 2. Twelve of the maps included in earlier editions of the IBC provided enlargements of portions of two maps that covered the entire United States; this proposal eliminates the enlargements. This is being recommended because the maps printed in former editions of the IBC, while generally illustrative of the earthquake hazard, could not be read clearly enough to provide exact design values for specific building sites. Those in need of precise design values can easily obtain them from a USGS web site (<http://earthquake.usgs.gov/research/hazmaps/design/index.php>) using the longitude and latitude of the building site, obtained from GPS mapping programs or web sites.

Detailed descriptions of changes made for the 2009 *NEHRP Recommended Seismic Provisions* are available at www.bssconline.org under the explanation of changes made for the 2009 edition of the *Provisions*.

Cost Impact: The new maps may lower costs in some locations but may increase them in others.

ICCFILENAME: WINKEL-COBEEN-S1-1613.1

Public Hearing Results

PART I- IBC STRUCTURAL Committee Action:

Approved as Modified

Modify the proposal as follows:

1613.2 Definitions. The following words and terms shall, for the purposes of this section, have the meanings shown herein.

MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION (MCE_R). The most severe earthquake effects considered by this code.

(No changes to definitions not shown)

1613.5.1 Mapped Acceleration Parameters. The parameters S_S and S_I shall be determined from the 0.2 and 1 s spectral response accelerations shown on Figures 1613.5(1) and ~~1613.5(2)~~ through 1613.5(6). Where S_I is less than or equal to 0.04 and S_S is less than or equal to 0.15, the structure is permitted to be assigned to Seismic Design Category A.

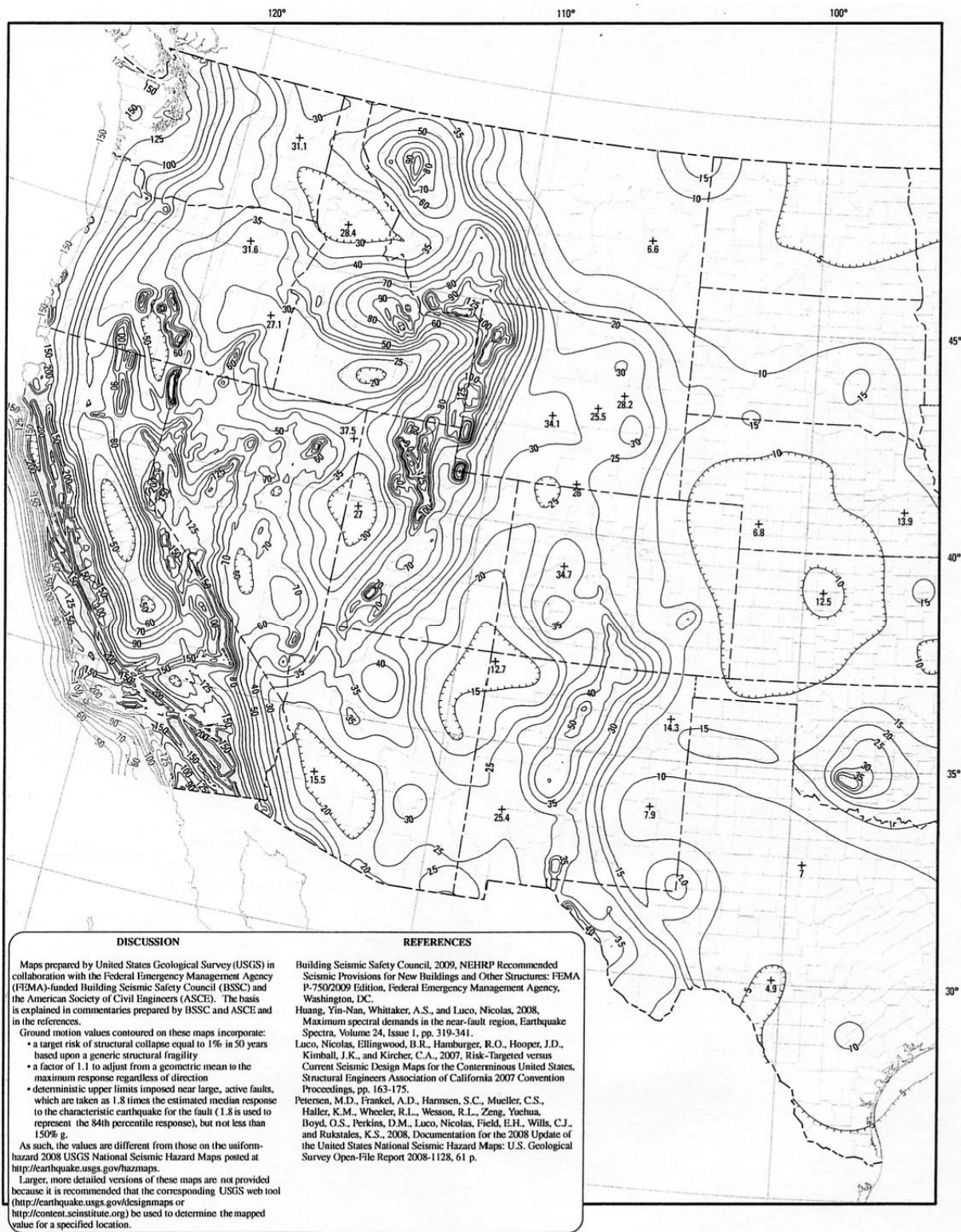


FIGURE 1613.5(1) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION (MCE_R) FOR THE CONTERMINOUS UNITED STATES OF 0.2 SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

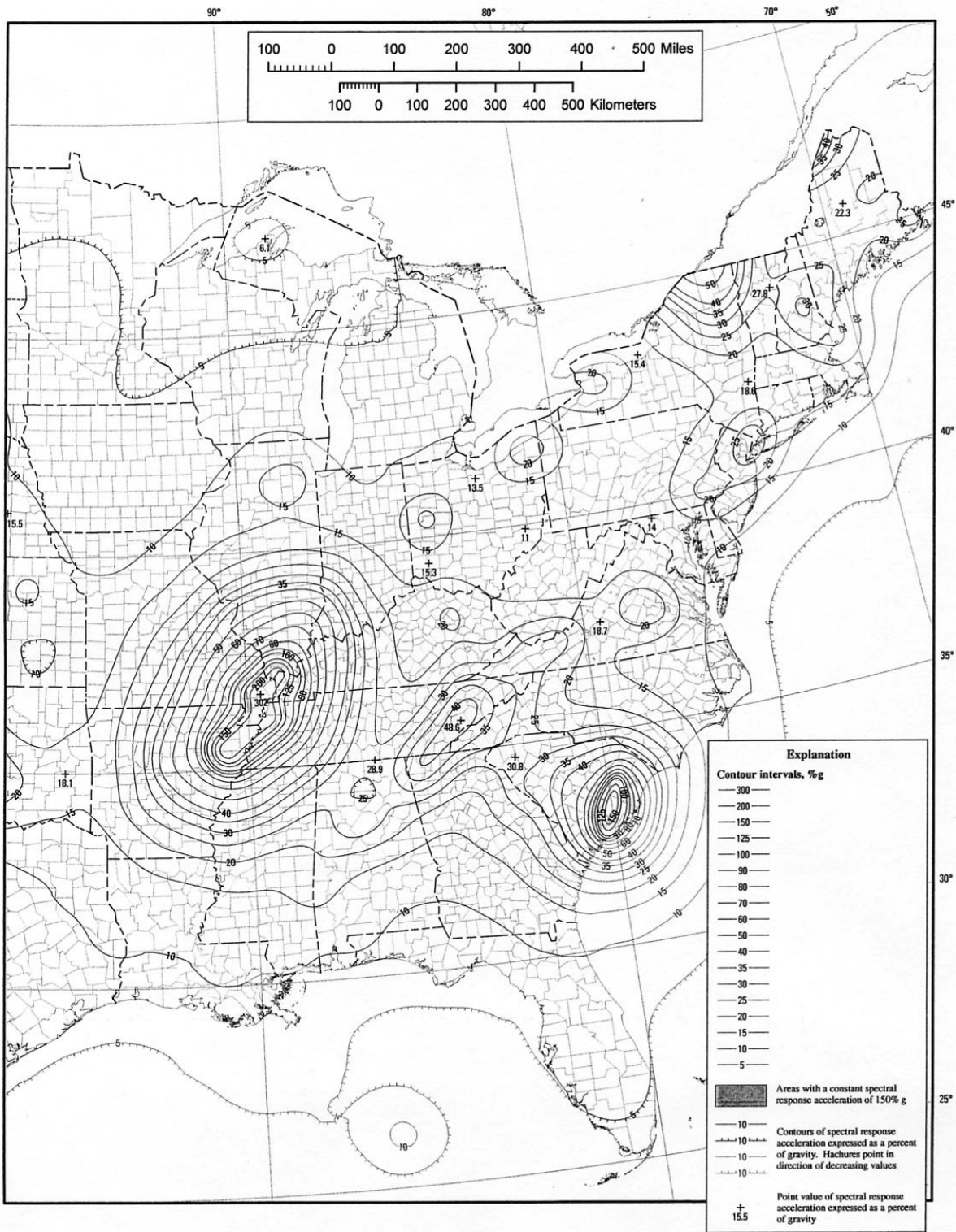


FIGURE 1613.5(1)(CONTINUED) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION (MCE_R) FOR THE CONTERMINOUS UNITED STATES OF 0.2 SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

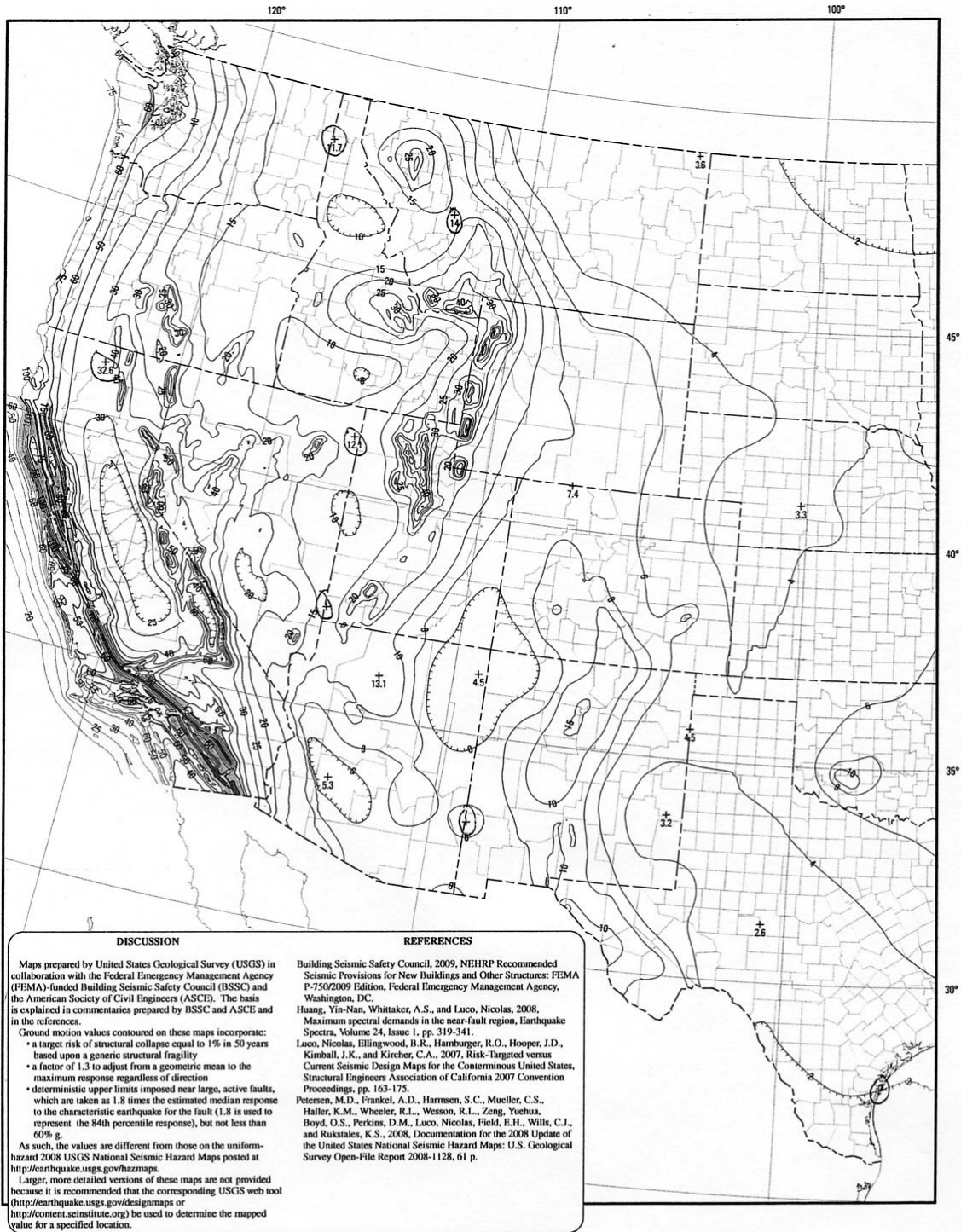


FIGURE 1613.5(2) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION (MCE_R) FOR THE CONTERMINOUS UNITED STATES OF 1 SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

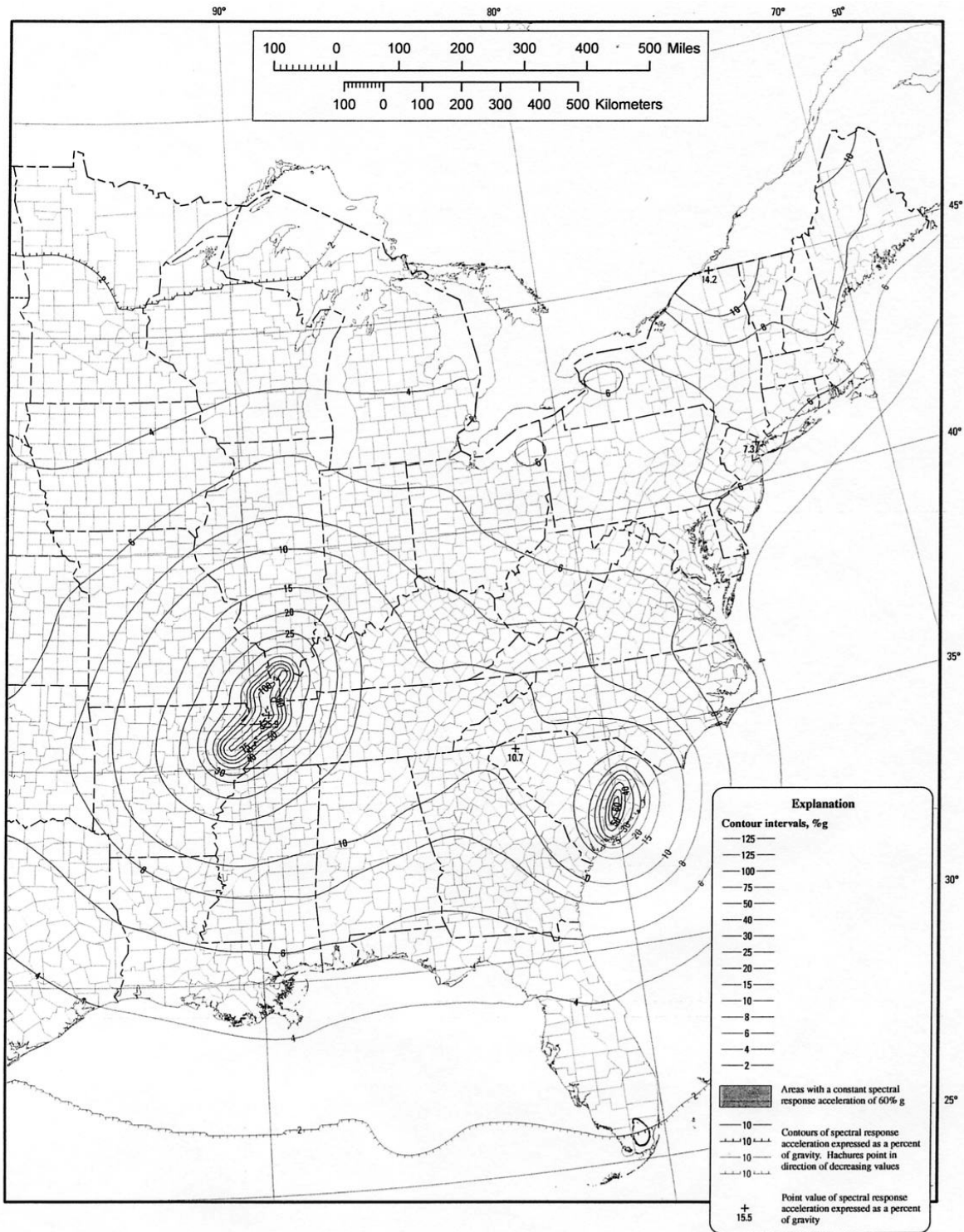
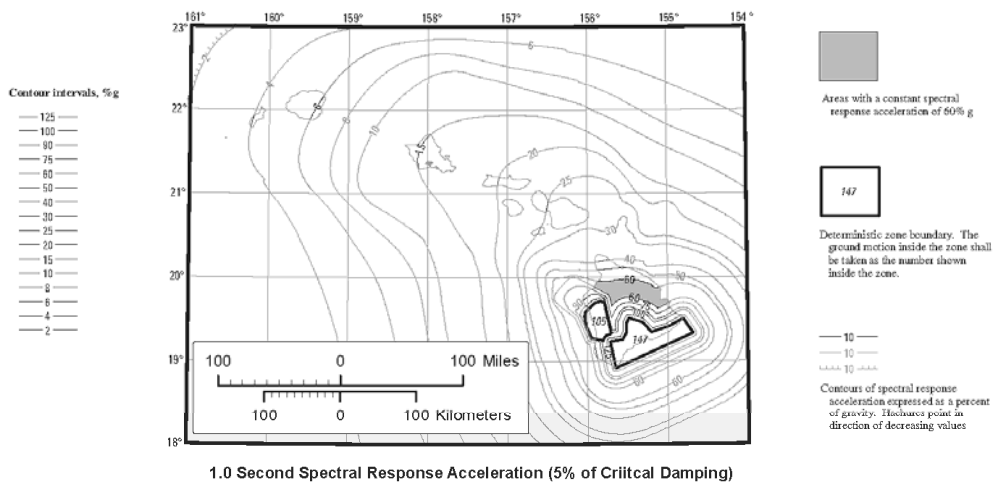
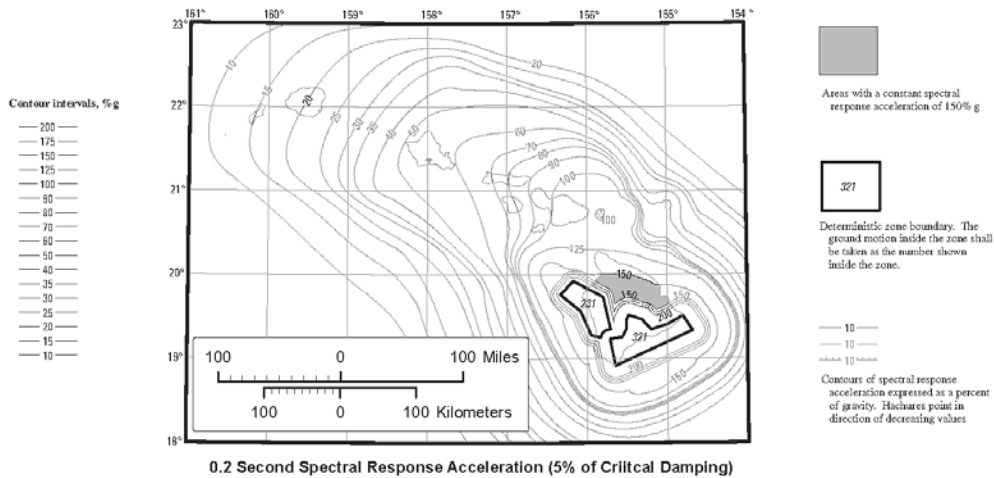


FIGURE 1613.5(2)(CONTINUED) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION (MCE_R) FOR THE CONTERMINOUS UNITED STATES OF 1 SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



DISCUSSION	REFERENCES
<p>Maps prepared by United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency (FEMA)-funded Building Seismic Safety Council (BSSC) and the American Society of Civil Engineers (ASCE). The basis is explained in commentaries prepared by BSSC and ASCE and in the references given below.</p> <p>Ground motion values contoured on these maps incorporate:</p> <ul style="list-style-type: none"> • a target risk of structural collapse equal to 1% in 50 years based upon a generic structural fragility • deterministic upper limits imposed near large, active faults, which are taken as 1.8 times the estimated median response to the characteristic earthquakes for the fault (1.8 is used to represent the 84th percentile response), but not less than 150 and 60% g for 0.2 and 1.0 sec, respectively. <p>As such, the values are different from those on the uniform-hazard 1998 USGS National Seismic Hazard Maps for Hawaii posted at http://earthquake.usgs.gov/hazmaps.</p> <p>Larger, more detailed versions of these maps are not provided because it is recommended that the corresponding USGS web tool (http://earthquake.usgs.gov/designmaps or http://content.seisintite.org) be used to determine the mapped value for a specified location.</p>	<p>Building Seismic Safety Council, 2009, NEHRP Recommended Seismic Provisions for New Buildings and Other Structures; FEMA P-750/2009 Edition, Federal Emergency Management Agency, Washington, DC.</p> <p>Huang, Yin-Nan, Whitaker, A.S., and Luco, Nicolas, 2008, Maximum spectral demands in the near-fault region, Earthquake Spectra, Volume 24, Issue 1, pp. 319-341.</p> <p>Klein, F., Frankel, A.D., Mueller, C.S., Wesson, R.L., and Otubo, P., 2001, Seismic hazard in Hawaii: high rate of large earthquakes and probabilistic ground-motion maps, Bulletin of the Seismological Society of America, Volume 91, pp. 479-498.</p> <p>Luco, Nicolas, Ellingwood, B.R., Hamburger, R.O., Hooper, J.D., Kimball, J.K., and Kircher, C.A., 2007, Risk-Targeted versus Current Seismic Design Maps for the Conterminous United States, Structural Engineers Association of California 2007 Convention Proceedings, pp. 163-175.</p>

FIGURE 1613.5(3) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION (MCE_R) FOR HAWAII OF 0.2 AND 1 SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

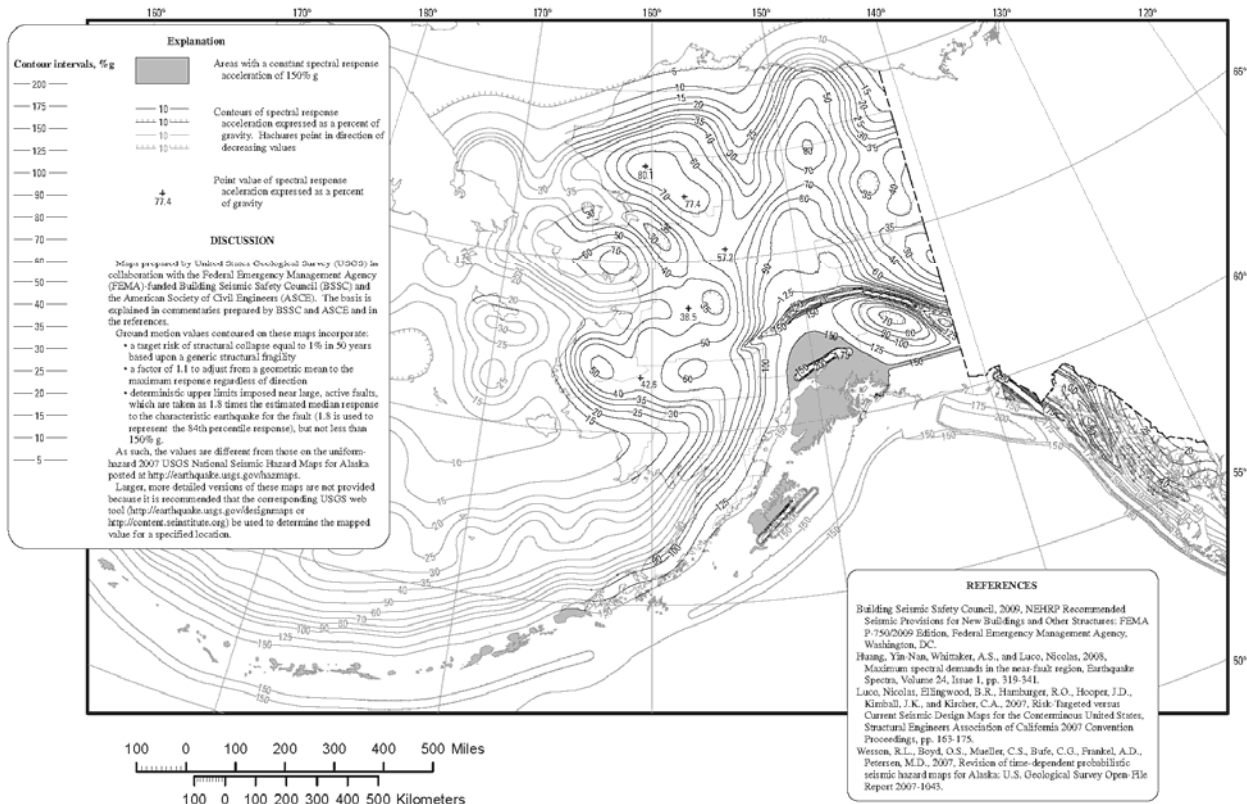


FIGURE 1613.5(4) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION (MCE_g) FOR ALASKA OF 0.2 SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

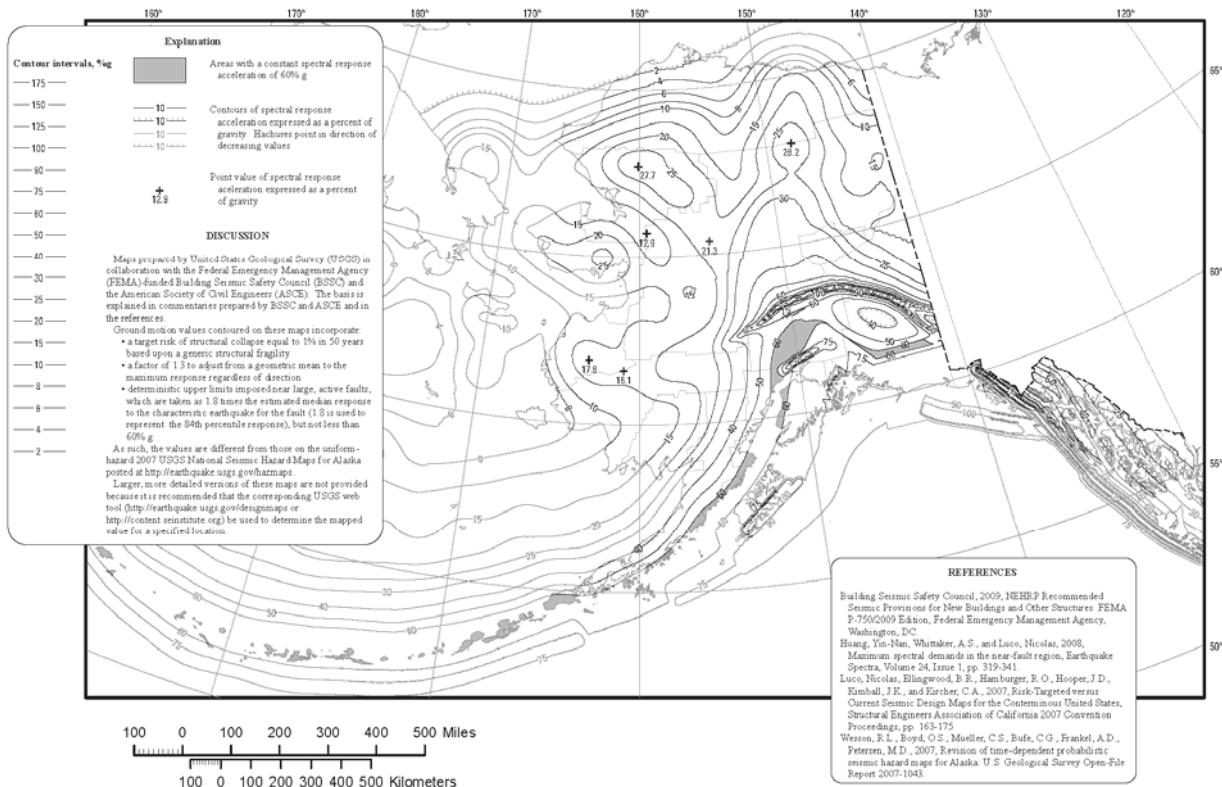
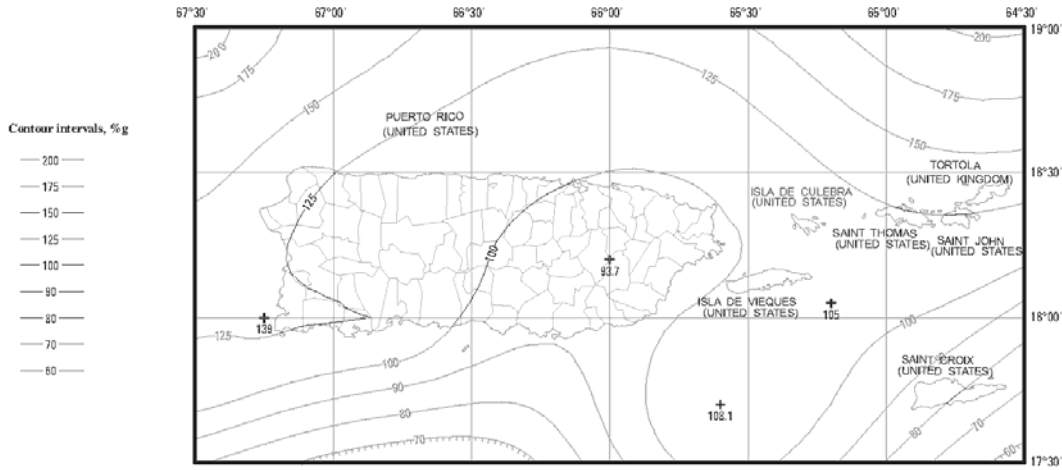
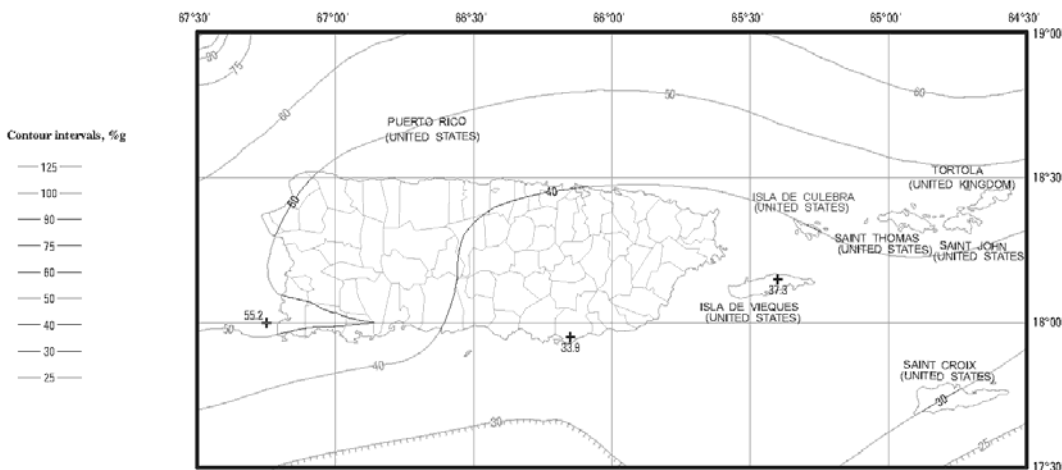


FIGURE 1613.5(5) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION (MCE_R) FOR ALASKA OF 1.0 SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



0.2 Second Spectral Response Acceleration (5% of Critical Damping)



1.0 Second Spectral Response Acceleration (5% of Critical Damping)

Explanation

— 10 —
 — 10 —
 — 10 —

Contours of spectral response acceleration expressed as a percent of gravity. Hashes point in direction of decreasing values

+
 93.7

Point value of spectral response acceleration expressed as a percent of gravity

DISCUSSION

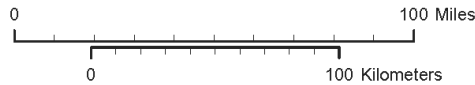
Maps prepared by United States Geological Survey (USGS) in collaboration with the Federal Emergency Management Agency (FEMA)-funded Building Seismic Safety Council (BSSC) and the American Society of Civil Engineers (ASCE). The basis is explained in commentaries prepared by BSSC and ASCE and in the references.

Ground motion values contoured on these maps incorporate:

- a target risk of structural collapse equal to 1% in 50 years based upon a generic structural fragility
- a factor of 1.1 and 1.3 for 0.2 and 1.0 sec, respectively, to adjust from a geometric mean to the maximum response regardless of direction
- deterministic upper limits imposed near large, active faults, which are taken as 1.8 times the estimated median response to the characteristic earthquake for the fault (1.8 is used to represent the 94th percentile response), but not less than 150 and 60% g for 0.2 and 1.0 sec, respectively.

As such, the values are different from those on the uniform-hazard 2003 USGS National Seismic Hazard Maps for Puerto Rico and the U.S. Virgin Islands posted at <http://earthquake.usgs.gov/hazmaps>.

Larger, more detailed versions of these maps are not provided because it is recommended that the corresponding USGS web tool (<http://earthquake.usgs.gov/designmaps>) or <http://content.seisintstate.org> be used to determine the mapped value for a specified location.



REFERENCES

Building Seismic Safety Council, 2009, NEHRP Recommended Seismic Provisions for New Buildings and Other Structures; FEMA P-750/2009 Edition, Federal Emergency Management Agency, Washington, DC.

Huang, Yin-Nan, Whittaker, A.S., and Luco, Nicolas, 2008, Maximum spectral demands in the near-fault region, Earthquake Spectra, Volume 24, Issue 1, pp. 319-341.

Luco, Nicolas, Ellingwood, B.R., Hamburger, R.O., Hooper, J.D., Kimball, J.K., and Kircher, C.A., 2007, Risk-targeted versus Current Seismic Design Maps for the Conterminous United States, Structural Engineers Association of California 2007 Convention Proceedings, pp. 163-175.

Mueller, C.S., Frankel, A.D., Petersen, M.D., and Leyendecker, E.V., 2003, Documentation for the 2003 USGS Seismic Hazard Maps for Puerto Rico and the U.S. Virgin Islands; U.S. Geological Survey Open-File Report 03-379.

Figure 22-6 Maximum Considered Earth Ground Motion (MCE_R) for Puerto Rico, Culebra, Vieques, St. Thomas, St. John and St. Croix

FIGURE 1613.5(6) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION (MCE_R) FOR PUERTO RICO, CULEBRA, VIEQUES, ST. THOMAS, ST. JOHN AND ST. CROIX OF 0.2 AND 1 SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

Committee Reason: This proposal incorporates the latest USGS ground motion maps. The modification updates the map titles and provides reformatted versions of the maps with no technical changes. It also separates areas outside the conterminous United States, on individual maps.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment 1:

Steven Winkel, FAIA, PE, and Kelly Cobeen, PE, SE, representing the Federal Emergency Management Agency/Building Seismic Safety Council Code Resource Support Committee (FEMA/BSSC CRSC) and James Rossberg, PE, representing the American Society of Civil Engineers (ASCE), request Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1613.2 Definitions. The following words and terms shall, for the purposes of this section, have the meanings shown herein.

RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R). ~~GROUND MOTION (MCE_R).~~ **RESPONSE ACCELERATIONS** The most severe earthquake effects considered by this code, determined for the orientation that results in the largest maximum response to horizontal ground motions and, with adjustment for targeted risk.

(No changes to definitions not shown)

FIGURE 1613.5(1)

RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS (MCE_R) FOR THE CONTERMINOUS UNITED STATES OF 0.2 SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

FIGURE 1613.5(2)

RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS (MCE_R) FOR THE CONTERMINOUS UNITED STATES OF 1 SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

FIGURE 1613.5(3)

RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS (MCE_R) FOR HAWAII OF 0.2 AND 1 SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

FIGURE 1613.5(4)

RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS (MCE_R) FOR ALASKA OF 0.2 SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

FIGURE 1613.5(5)

RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS (MCE_R) FOR ALASKA OF 1.0 SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

FIGURE 1613.5(6)

RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE (MCE_R) GROUND MOTION RESPONSE ACCELERATIONS (MCE_R) FOR PUERTO RICO, CULEBRA, VIEQUES, ST. THOMAS, ST. JOHN AND ST. CROIX OF 0.2 AND 1 SECOND SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

(Portions of proposal not shown remain unchanged)

Commenter's Reason: In the original submittal, the reason stated that this code change would be modified if required to correlate with modifications to the maps being adopted into ASCE 7. This modification changes the map titles and definition to coordinate with ASCE 7 wording. This change is editorial; there is no technical change.

Public Comment 2:

James Bela, Oregon Earthquake Awareness, requests Disapproval.

Commenter's Reason: The Commenter asks DISAPPROVAL, because the proposals [Part I- IBC STRUCTURAL; PART II- IRC B/E] were:

- (a) neither adequately nor correctly justified by the committee, but were let-in without a proper checking of their content and credentials, *simply* because they were "the latest USGS ground motion maps."
- (b) inclusive of *changes* in Mapped Spectral Response Acceleration contour values from previous code editions that were *not* specifically made clear (along with the impact of those changes on seismic design parameters).
- (c) inclusive of changes resulting from applying Next Generation Attenuation (NGA) relationships that reduced design spectral response acceleration by up to 25% in some earthquake prone regions of the country, despite a clear lack of consensus for this change from the USGS Ground Motion Mapping Workshop participants.
- (d) inclusive of a so-called "target risk of structural collapse equal to 1% in 50 years based upon a generic structural fragility" that is arbitrary, ambiguous and a major departure from all previous codes. This is particularly "dangerous" to public safety (because the specified target risk is

the same probability as considered for the May 2008 Wenchuan Earthquake in China—where that earthquake *disproved* all the assumptions underlying the formulation of the USGS Seismic Hazard Maps (and killed more than 80,000 people); and

- (e) introduces a new term MCE_R as a “MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION” – that is anything but an earthquake ground motion! This will introduce unnecessary confusion and complication into the code--in particular, problems in relating to earlier code editions for continuity.

The USGS Seismic Hazard Maps have been, and continue to be, too *unstable* as a basis for seismic design. We are introducing “yo-yo” tectonics into the seismic design codes, such that there is no longer *continuity* in our system of what to do to keep the earthquakes from attacking you! Engineers cannot develop experience and hone valuable judgement skills, if the code continues to be a moving target.

For the wholesale changes proposed here (where seismic hazard has become whatever input parameters people can come up with . . . from NGA, GPS strain monitoring, new faults and so-called slip rates, Risk mumbo-jumbo, generic fragility curves); the new spectral response acceleration maps should have first been produced in the same manner as the previous code editions - then a comparison of results could have been made with proposed applications of the new map. This is necessary for a reality check as to the reasonableness of adopting something “new” for “newness” sake, where it might weaken or reduce long accepted minimum standards for protecting the public safety.

These new maps should have provided a city-by-city comparison of old map design values with the “latest” to be considered values. Otherwise, there is no telling how the public safety is being impacted. Documentation of the discussions, votes taken, and specific explanations and justifications for these changes should have been provided to the committee and to the public. A code change proponent must be held accountable to justify these changes (and also clearly define these same) to the committee from which approval is sought. Giving a weblink to a telephone book of information is unacceptable for a public process.

These “latest” maps are unsafe at any parameter, because they are not anchored to a stable earthquake design methodology. The only stable basis for earthquake design provisions that truly protect public safety is to link them to a consideration of the maximum potential earthquake size (or Magnitude) from a specific fault. For example, earthquake design within 15 miles of a M 6 active fault, and within 25 miles of a M 7 or greater active faults should incorporate design provisions against that eventuality, regardless of complex methodologies of applied mathematics (to calculate earthquake probabilities and risk). These methods entice our code provisions to sink ever lower, because they too often perceive the likelihood of an earthquake to be too rare to be taken seriously.

Final Action: AS AM AMPC_____ D

S97-09/10-PART II

IRC Figure R301.2(2)

Proposed Change as Submitted

Proponent: Steven Winkel, FAIA, PE, Kelly Cobeen, PE, SE, and J. Daniel Dolan, PhD, PE, Building Seismic Safety Council (BSSC) of the National Institute of Building Sciences, representing the Federal Emergency Management Agency/BSSC Code Resource Support Committee

PART II – IRC BUILDING/ENERGY

Delete Figure R301.2(2) and substitute as follows:

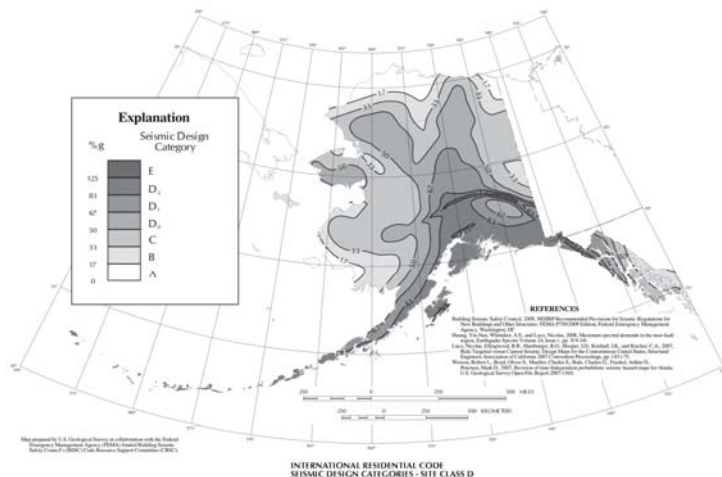
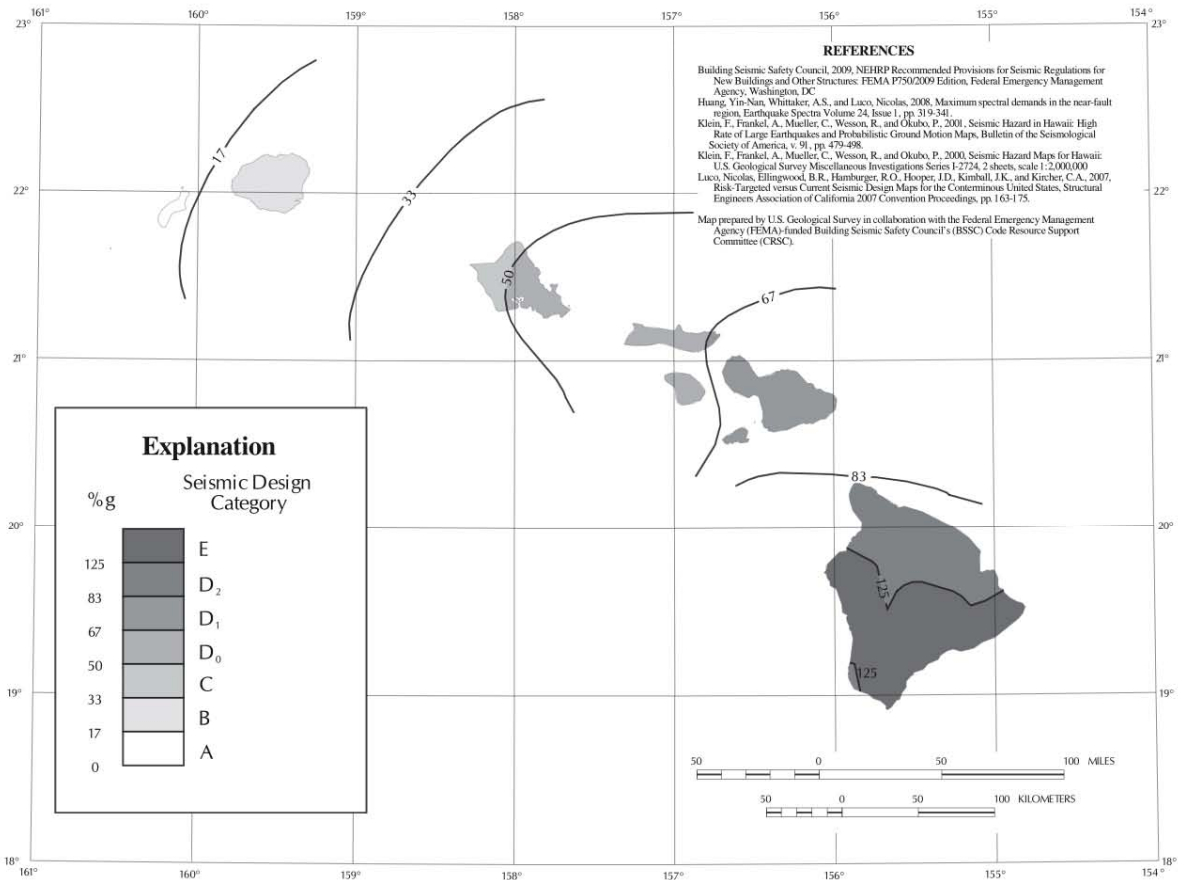


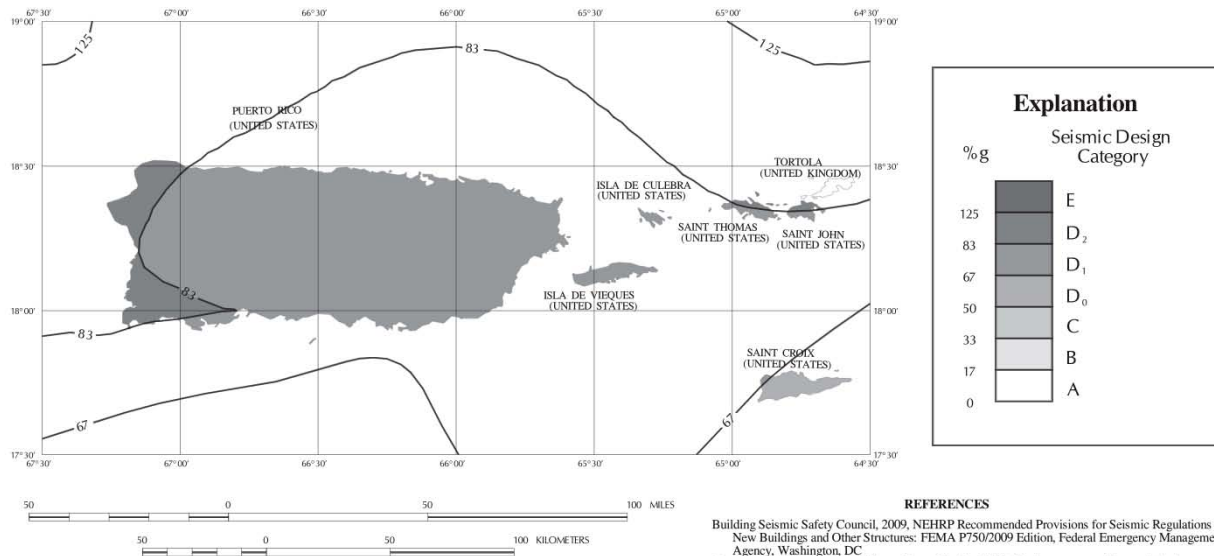
FIGURE R301.2(2)
SEISMIC DESIGN CATEGORIES -- SITE CLASS D

FIGURE R301.2(2) -- continued

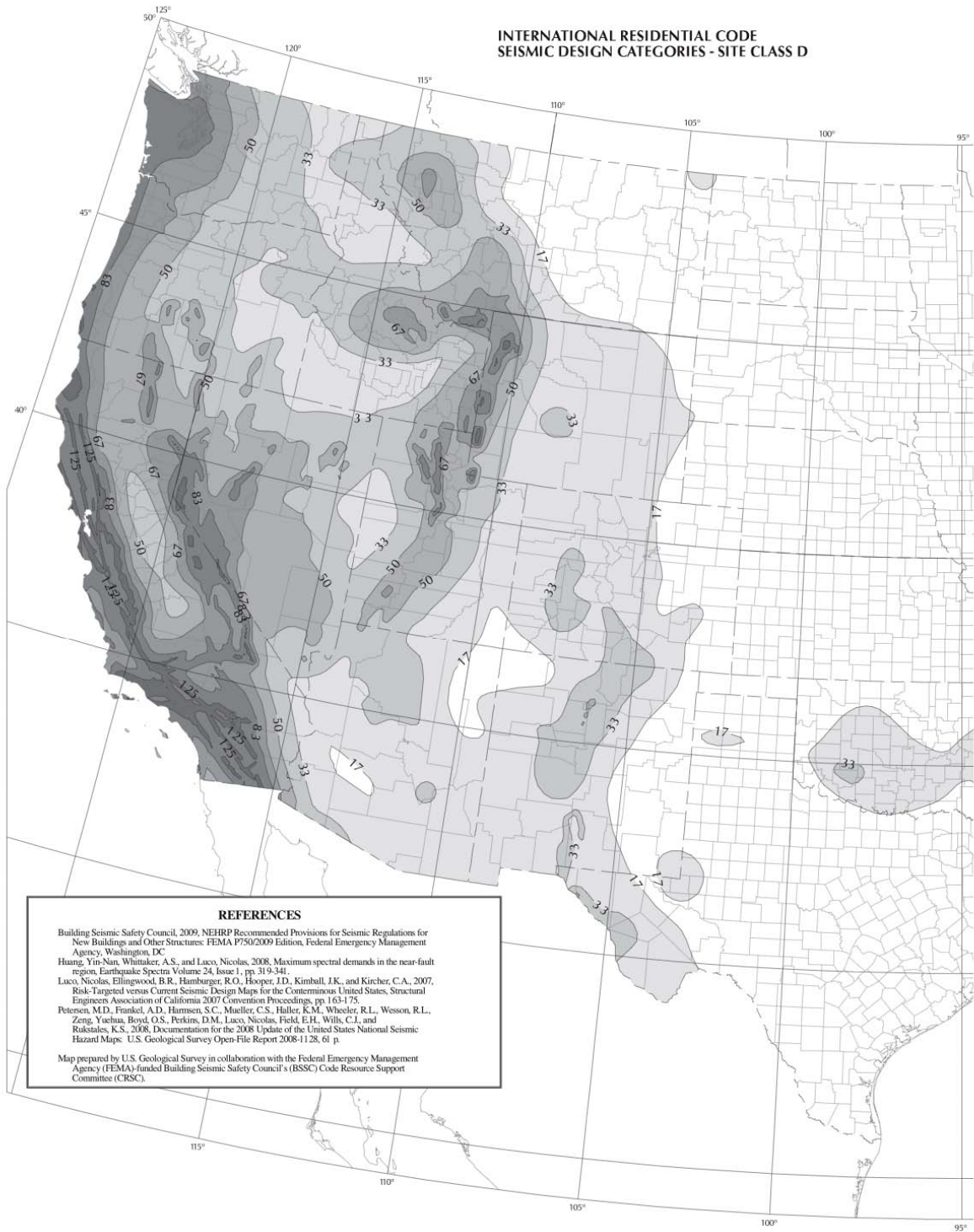


**INTERNATIONAL RESIDENTIAL CODE
SEISMIC DESIGN CATEGORIES - SITE CLASS D**

SEISMIC DESIGN CATEGORIES -- SITE CLASS D



**FIGURE R301.2(2) -- continued
SEISMIC DESIGN CATEGORIES -- SITE CLASS D**



**FIGURE R301.2(2) -- continued
SEISMIC DESIGN CATEGORIES -- SITE CLASS D**

INTERNATIONAL RESIDENTIAL CODE
SEISMIC DESIGN CATEGORIES - SITE CLASS D

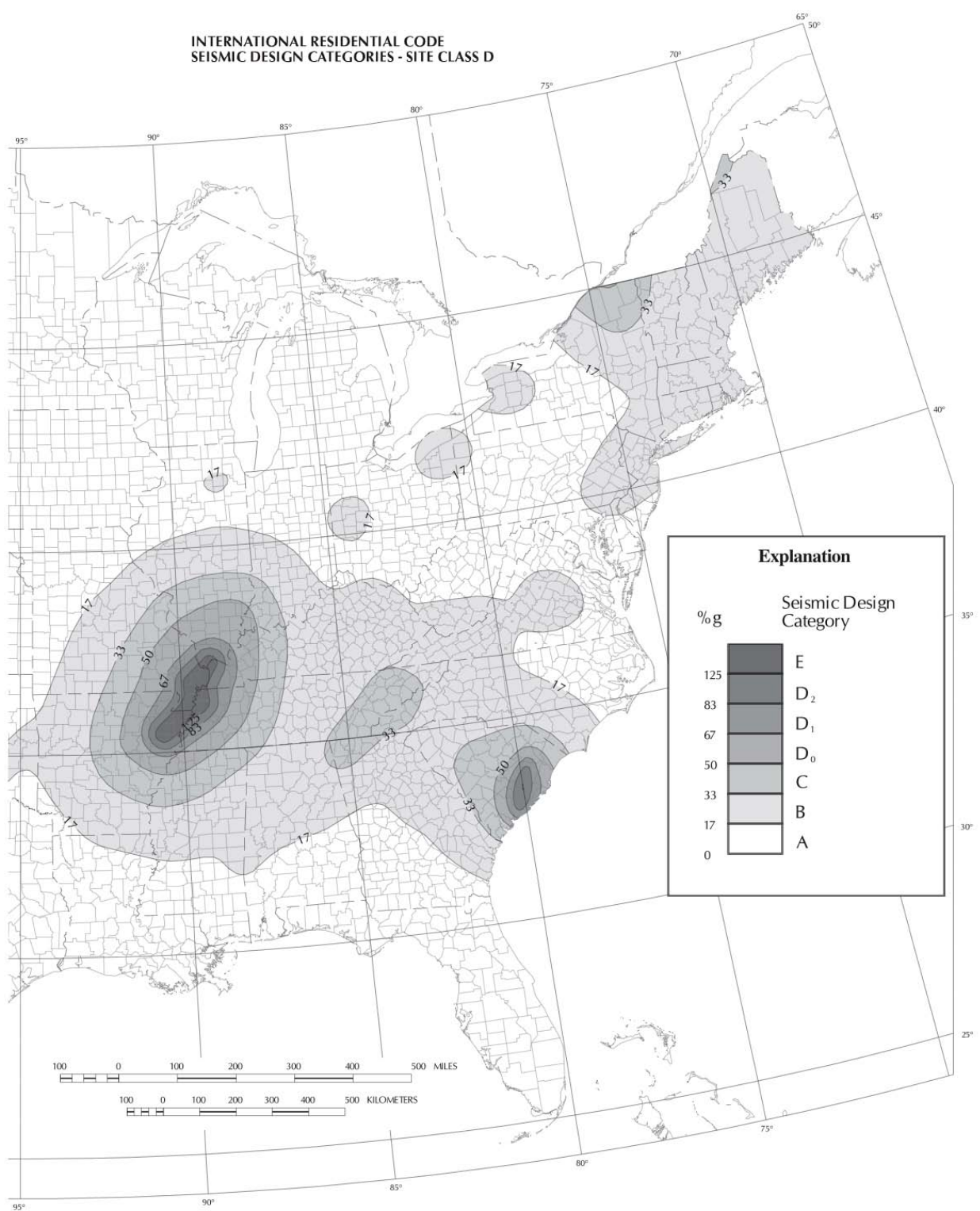


FIGURE R301.2(2) -- continued
SEISMIC DESIGN CATEGORIES -- SITE CLASS D

Reason: This proposal reflects new seismic hazard data developed by the U.S. Geological Survey (USGS) as part of its National Seismic Hazard Mapping Project and related technical changes developed by the Building Seismic Safety Council's (BSSC) Seismic Design Procedures Reassessment Group (SDPRG) as part of its work for the Federal Emergency Management Agency (FEMA).

The USGS and the FEMA-funded SDPRG worked together to update the seismic design maps and procedures for the 2009 edition of the NEHRP (National Earthquake Hazards Reduction Program) Recommended Seismic Provisions for New Building and Other Structures. The products of this collaboration are new design maps that appear in the 2009 *Provisions* and a similar version that is proposed for inclusion in ASCE 7-10. Although the terminology used in the *Provisions* is slightly different from that proposed for ASCE 7-10, the substance of the mapping changes is the same for both. The new design maps are based on USGS updates to their seismic hazard data and ground motion attenuation formulas as well as the SDPRG's use of risk-targeted ground motions, maximum direction ground motions, and near-source 84th percentile ground motions.

Code updates to the seismic maps and seismic resistant design requirements normally are drawn from ASCE 7 (*Minimum Design Loads for Buildings and Other Structures*) which is, in turn, based on the *NEHRP Recommended Provisions*. This proposal reflects material developed under

the 2009 *NEHRP Recommended Provisions* as presented in the current draft proposal for ASCE 7-10. The ICC code change submittal schedule makes it necessary to submit this working version with the understanding that it will be updated to the extent possible to reflect any modifications made by ASCE 7. Note that the maps included in this proposal are based on the maps proposed for inclusion in the IBC. If this code change is not moved forward, the IRC will retain superseded seismic hazard mapping information, thereby potentially being in conflict with the IBC.

These new IRC maps are different from earlier versions in that the division between Seismic Design Categories D2 and E has been changed from 118% g to 125% g. The 125% g contour would have been used in earlier maps but the mapping technology then available for drawing the IRC maps did not permit this to be done. The result of this change and the improved seismic hazard data generated by the USGS over the past 10 years is that the geographic region affected by the Seismic Design Category E designation is smaller. This occurs primarily in the region around Charleston, South Carolina, but is also evident in Seismic Design Category E regions in other parts of the United States. As noted above, maps developed on the same basis have been proposed for the IBC which will allow engineers to design components of the building that are outside of the scope of the IRC with compatible seismic loads.

Cost Impact: This proposal will not increase the cost of construction and will reduce the cost in some regions.

ICCFILENAME: WINKEL-COBEEN-S2-1613.5.1

Public Hearing Results

**PART II- IRC B/E
Committee Action:**

Approved as Submitted

Committee Reason: This change brings the latest and improved Seismic Maps into the code. This will correlate the maps with the IBC and ASCE 7-10. One benefit of the new map is that some Seismic Design Category E regions will be smaller in area. This will result in some previous Seismic Design Category E structures to now be Seismic Design Category D structures.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment:

James Bela, Oregon Earthquake Awareness, requests Disapproval.

Commenter's Reason: See S97-09/10-Part I

Final Action: AS AM AMPC____ D

S108-09/10 1613.8 (New), Appendix L (New)

Proposed Change as Submitted

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

Add new text as follows:

1613.8 Earthquake-recording instrumentations. For earthquake-recording instrumentations, see Appendix L.

APPENDIX L

EARTHQUAKE RECORDING INSTRUMENTATIONS

SECTION L101 GENERAL

L101.1 General. Every building located where the 1-second spectral response acceleration, S_1 , in accordance with Section 1613.5 is greater than 0.40 that either 1) exceeds six stories above grade plane with an aggregate floor area

of 60,000 square feet (5574 m²) or more, or 2) exceeds 10 stories above grade plane regardless of floor area, shall be provided with not less than three approved recording accelerographs.

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

L 101.2 Location. As a minimum, instruments shall be located at the lowest level, midheight, 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.

L 101.3 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.

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 from Section 1652 and Appendix Chapter 16, Division II of the 1997 UBC. When the IBC was created, this section was apparently inadvertently not included. The code change proposal is intended to correct this oversight. The proposed change only covers instrumentation in newly constructed buildings. This proposal was submitted in the last cycle as a mandatory requirement in Chapter 1613. The Structural Committee suggested it be resubmitted as a non-mandatory Appendix during this cycle.

Cost Impact: Because this is an optional Appendix, this change will only have a cost impact in Jurisdictions in which it is adopted. In Jurisdictions where it is adopted, the cost impact will depend on whether similar ordinances are already in place. If ordinances are already in place, the cost impact will be negligible. For jurisdictions that adopt where ordinances are not in place, the cost impact, would be very small (less than 0.1% of the cost of the new construction) and only apply to very few structures in the high areas of seismic activity.

ICCFILENAME: Bachman-S1-1613.8-App L

Public Hearing Results

Committee Action:

Approved as Modified

Modify the proposal as follows:

~~**1613.8 Earthquake Recording Instrumentations.** For earthquake recording instrumentations, see Appendix L.~~

L101.1 General. Every ~~structure building~~ located where the 1-second spectral response acceleration, S_1 , in accordance with Section 1613.5 is greater than 0.40 that either 1) exceeds six stories ~~in height above grade plane~~ with an aggregate floor area of 60,000 square feet (5574 m²) or more, or 2) exceeds ~~ten 40~~ stories ~~in height above grade plane~~ regardless of floor area, shall be ~~equipped~~ provided with not less than three approved recording accelerographs. ~~The accelerographs shall be interconnected for common start and common timing.~~

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

L 101.2 Location. As a minimum, instruments shall be located at the lowest level, mid-height, and near the top of the ~~structure 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" in one inch block letters shall be posted in a conspicuous location.

L 101.3 Maintenance. Maintenance and service of the instrumentation shall be provided by the owner of the ~~structure building~~, ~~subject to the approval of the building official~~. Data produced by the instrument shall be made available to the building official on request.

Maintenance and service of the instruments shall be performed annually by an approved testing agency. The owner shall file with the building official a written report from an approved testing agency certifying that each instrument has been serviced and is in proper working condition. This report shall be submitted when the instruments are installed and annually thereafter. Each instrument shall have affixed to it an externally visible tag specifying the date of the last maintenance or service and the printed name and address of the testing agency.

(Portions of the proposal not shown are unchanged.)

Committee Reason: An appendix chapter on earthquake recording instrumentation is an important addition to the IBC for those jurisdictions that have typically adopted such provisions. The data collected is valuable in understanding how earthquakes affect structures. The modification removes an unnecessary reference to the appendix from Chapter 16. "Building" has been appropriately changed to the more general term, "structure". The reference to the building official's approval was removed from the section on maintenance since this would be difficult to enforce after a certificate of occupancy is issued. Other changes are consistent with similar requirements in the LA City Building Code.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment:

James Bela, Oregon Earthquake Awareness, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1613.8 Earthquake Recording Instrumentation. For earthquake-recording instrumentations, see Appendix L.

L101.1 General. Every ~~structure building~~ located where the 1-second spectral response acceleration, S_1 , in accordance with Section 1613.5 is greater than 0.40, ~~or lies within 15 miles distance of an active fault with a maximum potential earthquake M 6 or above, or lies within 25 miles distance of an active fault with a maximum potential earthquake M 7 or above; and that either 1) exceeds six stories in height with an aggregate floor area of 60,000 square feet (5574 m²) or more, or 2) exceeds ten stories in height 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.~~

L 101.2 Location. As a minimum, instruments shall be located at the lowest level, mid-height, and near the top of the ~~structure 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" in one inch block letters shall be posted in a conspicuous location.

L 101.3 Maintenance. Maintenance and service of the instrumentation shall be provided by the owner of the ~~structure building~~, subject to the approval of the building official. Data produced by the instrument shall be made available to the building official on request.

Maintenance and service of the instruments shall be performed annually by an approved testing agency. The owner shall file with the building official a written report from an approved testing agency certifying that each instrument has been serviced and is in proper working condition. This report shall be submitted when the instruments are installed and annually thereafter. Each instrument shall have affixed to it an externally visible tag specifying the date of the last maintenance or service and the printed name and address of the testing agency.

(Portions of the proposal not shown are unchanged)

Commenter's Reason: The 1-second spectral response acceleration contours are interesting, but their locations are yo-yoing around with each new addition of the maps; so that are not reliable over time. An earthquake will occur on a fault, and it is the proximity of a building to an earthquake source that determines its actual experience to ground shaking in a real earthquake. This additional charging language fills this hole in locations, particularly in the western U.S. where there are active faults; but the sum total of all contributing faults is not enough to give 1-second contours of 0.40g.

The term building is as used in the city of Los Angeles strong motion accelerograph language. We have building officials, building codes, building permits, building maintenance, Building Owners and Managers Associations . . . so everyone is pretty clear what a "building" actually is. Maybe, for example, an airplane hangar is more of a structure, than it is a building?

Final Action: AS AM AMPC _____ D

S111-09/10
1702.1

Proposed Change as Submitted

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

Revise as follows:

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

SPECIAL INSPECTION, CONTINUOUS. The ~~full-time~~ observation of construction or work requiring special inspection by an approved special inspector who is continuously present in the area when and where the construction or work is being performed.

SPECIAL INSPECTION, PERIODIC. The ~~part-time or intermittent~~ observation of construction or work requiring special inspection by an approved special inspector who is intermittently present in the area when and where the construction or work has been or is being performed ~~and at the completion of the work.~~

Reason: The purpose for this proposal is to adjust the definitions for "continuous special inspection" and "periodic special inspection" for consistency with the requirements for special inspection elsewhere in Chapter 17. These requirements typically specify special inspections as either continuous or periodic. The only means in the IBC for determining what is required of a special inspector to perform continuous or periodic special inspection is their respective definitions in Section 1702.1. The definitions should be such that the special inspector is able to arrive at the site in time to observe the construction or work sufficiently to enable a determination of whether the construction complies with applicable requirements in the building code and its reference standards and is in accordance with the approved construction documents.

The definitions need to account for two primary aspects of special inspection: extent and frequency. Frequency can be seen as the number of times a special inspector inspects; extent can be seen as the degree to which a special inspector inspects. Neither can be comprehensively accounted for in a definition and this proposal does not attempt to do so. However, adjustments to the definitions are proposed to improve their correlation with the extent and frequency assumed for the special inspections where continuous or periodic special inspection is specified. In both definitions, "construction" is added before "work" for consistency with the same phrase in Section 110.1 on inspections by the building official. Also in both definitions, "when" is added before "where" to indicate that the special inspector is expected to be in the area while the work is being performed, not before or after the work is being performed, which is possible with the current definitions.

In the definition of periodic special inspection, "has been" is deleted so that the definition is silent on whether performing special inspections after the construction or work is completed constitutes periodic special inspection. It is conceivable that certain special inspections are possible after completion of the construction or work but this should be agreed upon by all affected parties, including, but not limited to, the owner or owner's representative, contractor, special inspector and the building official. Retaining "has been" in the definition, however, implies that special inspection after the construction or work is completed always constitutes periodic special inspection and there are certain special inspections identified as periodic elsewhere in Chapter 17 for which such inspection may not be sufficient.

Also in the definition of periodic special inspection, "at the completion of work" is deleted. Where periodic special inspection is warranted, whether the special inspector is present "at the completion of work" is irrelevant. An intermittent presence permits time gaps between actions or observations by the special inspector, which includes a period of time between the last action or observation by the special inspector and the completion of the work. Where this is not considered to be a sufficient presence by the special inspector, periodic special inspection is not warranted.

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

ICCFILENAME: Brazil-S41-1702.1

Public Hearing Results

Committee Action:

Disapproved

Committee Reason: There was concern over striking "at the completion of the work" from the definition of periodic special inspection. The proposed revisions should be reconciled with S115 – 09/10

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment:

Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing self; D. Kirk Harman, P.E., The Harman Group, representing the National Council of Structural Engineers Associations (NCSEA) Code Advisory Committee, Quality Assurance and Special Inspection Subcommittee; Jonathan Siu, P.E., S.E., City of Seattle, Department of Planning and Development, representing self; John Silva, P.E., S.E., Hilti North America, representing self, request Approval as Modified by this Public Comment.

Modify the proposal as follows:

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

SPECIAL INSPECTION. Inspection as herein required of the materials, installation, fabrication, erection or placement of components and connections of construction requiring special the expertise of an approved special inspector in order to ensure compliance with this code and the approved construction documents and referenced standards (see Section 1704).

SPECIAL INSPECTION, CONTINUOUS Continuous special inspection. The observation of construction or work requiring Special inspection by an approved the special inspector who is continuously present in the area when and where the construction or work to be inspected is being performed.

SPECIAL INSPECTION, PERIODIC Periodic special inspection. The observation of construction or work requiring Special inspection by an approved the special inspector who is intermittently present in the area when and where the construction or work to be inspected has been or is being performed.

Commenter's Reason: The original proposal modified the definitions of "special inspection, continuous" and "special inspection, periodic" but, based on comments at the code development hearings, it was recognized that some of the objectives in submitting the proposal would be better served by also considering the definition of "special inspection," which is being done in this public comment.

The proposed revisions to these definitions recognize that a definition is inherently nonmandatory and can be nothing more than an explanation of the meaning for a word or phrase. One purpose for the revisions in this public comment is so that these definitions provide this explanation.

The proposed revisions also recognize what these definitions can not do and that is to explain the meaning of "inspection." The IBC does not contain a definition of "inspection" deferring to dictionaries for an explanation. Therefore, the definition of "special inspection" should explain what it means for an inspection to be "special" and should not also explain the meaning of "inspection." Also, the definitions of continuous special inspection and periodic special inspection should explain what it means for a special inspection to be "continuous" or "periodic" and should not also explain the meaning of "special inspection."

In the definition of “special inspection,” inspection of the “materials, installation, fabrication, erection or placement of components and connections” is replaced by inspection of “construction” because they are related to the meaning of “inspection,” not to what it means for an inspection to be “special.” A list of this sort is also inappropriate because it is not comprehensive and implies that actions other than those listed are not special inspections.

In the definition of “special inspection,” “special” is deleted before “expertise” because specifying expertise is sufficient to denote the role of the special inspector in the definition of special inspection: provide expertise. If “special” were to remain, it would imply that a special inspector with the expertise to ensure compliance would not be conducting a special inspection unless his or her expertise is “special” and would raise the question of what is necessary for an individual’s expertise to be special in order to qualify as special expertise.

In the definition of “special inspection,” “this code” is added because an essential part of special inspection by a special inspector is compliance with the building code as well as the approved construction documents. There are requirements in the building code relevant to the performance of the special inspector’s duties that may not appear in the approved construction documents.

In the definition of “special inspection,” “and referenced standards (see Section 1704)” is deleted because specifying them is redundant and can lead to conflicts with the building code. According to IBC Section 102.4, referenced standards are a mandatory “part or the requirements of this code to the prescribed extent of each such reference. Where differences occur between provisions of this code and referenced codes and standards, the provisions of this code shall apply.” Specifying “referenced standards” but not “this code” conflicts with Section 102.4 in that the referenced standards apply rather than the building code where there are conflicts between them and the building code, and the referenced standards apply to their full extent rather than to their prescribed extent.

Adding “approved special inspector” to the definition of “special inspection,” and deleting “approved” from the definitions of continuous special inspection and periodic special inspection is seen as editorial because whether a special inspector is approved is not related to the distinction between “continuous” and “periodic.”

The definitions of “special inspection, continuous” and “special inspection, periodic” are adjusted to be sub-definitions of “special inspection” because the purpose in defining them should be to explain what it means for a special inspection to be “continuous” or “periodic.”

In the sub-definitions of “continuous special inspection” and “periodic special inspection,” deleting “the observation of construction or work” is seen as editorial because it is redundant given the adjustment of the current definitions to be sub-definitions that explain what it means for a special inspection to be “continuous” or “periodic” (see above).

In the sub-definitions, the addition in the original proposal of “construction or” before “work” is removed and “to be inspected” is added after “work” in this public comment to establish the distinction between “inspection of construction” in the definition of special inspection and “work to be inspected” by the special inspector in the sub-definitions. Thus, “special inspection” is a process involving the actions of a special inspector and the continuous special inspection” or “periodic special inspection” are the actions of a special inspector. With the addition of “to be inspected,” the deletion in the original proposal of “and at the completion of work” in the sub-definition of “periodic special inspection,” is now seen as editorial because it is redundant given the obvious assumption that a special inspector will complete the work to be inspected. This is also being done for consistency with similar terminology in Section 1704.1.

In the sub-definitions, deleting “in the area” is seen as editorial because it is redundant given the text explaining that the special inspector is present “where the work to be inspected is being performed.” Obviously, the special inspector is “in the area” when he or she is at “the work to be inspected.”

The deletion in the original proposal of “has been” before “or is being performed” is restored in the sub-definition of “periodic special inspection” to distinguish between periodic special inspection that could be delayed until after material installation but before the materials are covered (e.g., reinforcing steel after installation but before concrete placement), and continuous special inspection that typically can’t be delayed until after material installation (e.g., concrete placement). Also, the addition in the original proposal of “when and” before “where” is removed from the sub-definition of “periodic special inspection” but is retained in the sub-definition of “continuous special inspection” so that “has been” does not conflict with “intermittently present” (avoids “intermittently present when...the work...has been...performed”).

Finally, the original proposal and this public comment were submitted, in part, because of the apparent belief by some individuals that the current definition of continuous special inspection intends inspection by a special inspector who is continuously present the entire time construction requiring special inspection is occurring even to the extent of expecting a special inspector be continuously present to observe the actions of each worker involved in the construction (i.e., 100 welders would require 100 special inspectors). This is not the intent of the definition nor is it the purpose for continuous special inspection, which is intended to provide a level of quality assurance sufficient to ensure that public safety in the built environment is achieved. For any construction project, the extent of special inspection and the frequency of special inspections can not be reliably determined except through an agreement between the owner, contractor, special inspector and the building official before construction begins on the scope of work for the special inspector. Chapter 17 of the International Building Code does not specify extent or frequency except through the definitions of continuous and periodic special inspection and the revisions to the definitions in this public comment will not change that.

Final Action: AS AM AMPC___ D

S113-09/10 1702.1

Proposed Change as Submitted

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

Add new definition as follows:

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

SPECIAL INSPECTOR. An individual qualified in accordance with Section 1704.1 of this code, employed or retained by the *approved agency* and assigned to execute the *special inspections* or tests required by the statement of special inspections.

Reason: The term Special Inspector is used many times throughout the chapter but is currently not defined. Approved Agency is defined as the entity that provides inspection but the inspections are actually accomplished by the "special inspector". Both should be defined.

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

ICCFILENAME: Harman-S1-1702.1

Public Hearing Results

Committee Action:

Disapproved

Committee Reason: The proposed definition is not needed since Section 1704.1 currently contains this information.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment:

Philip Brazil, P.E., S.E., Reid Middleton, Inc., representing self; D. Kirk Harman, P.E., The Harman Group, representing the National Council of Structural Engineers Associations (NCSEA) Code Advisory Committee, Quality Assurance and Special Inspection Subcommittee; Jonathan Siu, P.E., S.E., City of Seattle, Department of Planning and Development, representing self; John Silva, P.E., S.E., Hilti North America, representing self, request Approval as Modified by this Public Comment.

Modify the proposal as follows:

SPECIAL INSPECTOR. ~~An individual A qualified in accordance with Section 1704.1 of this code, person employed or retained by the an approved agency and assigned to execute the special inspections or tests required by the statement of special inspections approved by the building official as having the competence necessary to inspect a particular type of construction requiring special inspection.~~

1704.1 General. Where application is made for construction as described in this section, the owner or the *registered design professional in responsible charge* acting as the owner's agent shall employ one or more *approved agencies* to perform inspections during construction on the types of work listed under Section 1704. These inspections are in addition to the inspections specified in Section 110. ~~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 act as the special inspector for the work designed by them, provided those personnel meet the qualification requirements of this section to the satisfaction of the building official they qualify as special inspectors.~~ The special inspector shall provide written documentation to the building official demonstrating his or her 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~~ construction 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 Group U occupancies that are accessory to a residential occupancy including, but not limited to, those listed in Section 312.1.

Commenter's Reason: This public comment addresses the concern of the Committee that the definition is not needed because Section 1704.1 currently contains the information in the proposed definition. The text from Section 1704.1 that is replaced by the definition is deleted. The definition is also revised to be consistent with the text in Section 1704.1 that it replaces. In addition, "work" is changed to "construction" in Exception #1 of Section 1704.1 for consistency with the public comment on Proposal S111-09/10, which distinguishes between "inspection of construction" in the definition of special inspection and "work to be inspected" by the special inspector in the sub-definitions of "continuous special inspection" and "periodic special inspection."

Note that the first paragraph of Section 1704.1 currently requires the owner, or the registered design professional in responsible charge acting as the owner's agent, to employ one or more approved agencies, not special inspectors, to perform the necessary special inspections; and the second paragraph of Section 1704.1 requires the special inspector to "demonstrate competence, to the satisfaction of the building official, for the inspection of the particular type of construction or operation requiring special inspection." This proposal defines a "special inspector" as "employed or retained by an approved agency" for consistency with the first paragraph of Section 1704.1 and also as "approved by the building official as having the competence necessary to inspect a particular type of construction requiring special inspection" for consistency with the second paragraph of Section 1704.1.

Final Action: AS AM AMPC D

S118-09/10

1704.1

Proposed Change as Submitted

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

Revise as follows:

1704.1 General. Where application is made for construction as described in this section, the owner or the *registered design professional in responsible charge* acting as the owner's agent shall employ one or more *approved agencies* to perform inspections during construction on the types of work listed under Section 1704. These inspections are in addition to the inspections identified in Section 110.

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 act as the special inspector for the work designed by them, provided those personnel meet the qualification requirements of this section to the satisfaction of the *building official*. 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 Group U occupancies 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 portions of structures designed and constructed in accordance with the conventional light-frame construction provisions of Section 2308.

Reason: The purpose of this proposal is to add an exemption from third-party special inspections for portions of wood-frame dwellings or other simple wood-frame structures constructed under prescriptive provisions within the International Building Code (IBC). Without this exception, a building official may require a builder to contract with a third-party inspector, with the expense passed on to the homeowner.

A change made to the IBC during the 2006-07 Code Development Cycle (S31-06/07) struck the exemption for Residential R-3 structures, and now subjects one- and two-family dwellings and townhouses designed under the IBC to the requirements for special inspections. These inspections are in addition to the standard inspections performed by the building department. Also, other structures classified as R-3 occupancies (group homes, day care) will be subject to these special inspections for all elements of their construction. As justification for the original code change, the proponent claimed R 3 structures often contain complicated roof truss systems, structural steel framing, reinforced masonry and other complex elements or unusual construction materials and methods requiring the qualifications and experience of a special inspector.

But, IBC Section 1704.1.1 exempts the registered design professional from needing to prepare, and the permit applicant from needing to submit, a statement of special inspections for structures designed and constructed per Section 2308. This clearly implies that structures built under Section 2308 do not need special inspections for any element, including the wood wall framing, roof and floor trusses, concrete or masonry foundations, and any miscellaneous masonry or steel framing inside the structure. In a structure designed to the conventional construction provisions, these elements are not likely to be as complex as those in a fully-engineered structure.

Building departments are more than capable of reviewing and inspecting these simple structures. In the case of items such as trusses and miscellaneous steel framing that may occur in a structure otherwise designed using conventional construction provisions, shop drawings will be submitted to the building official for their review and use in inspections. The building department does not need a special inspector to do their work for them in reviewing and inspecting these structures and elements.

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

ICCFILENAME: Ehrlich-S1-1704 1

Public Hearing Results

Committee Action:

Disapproved

Committee Reason: The wording of the proposed exception in Section 1704.1 is potentially confusing, specifically the reference to "portions of structures". Furthermore, the reference solely to section 2308 would be too narrow since it would not include other types of light-frame construction.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment:

Gary J. Ehrlich, PE, National Association of Home Builders; Bonnie Manley, PE, AISI, representing Steel Framing Alliance; Larry Wainright, Qualtim, Inc., representing Structural Building Components Association (SBCA); Philip Brazil, PE, SE, Reid Middleton, Inc., representing self, request Approval as Modified by this Public Comment.

Modify the proposal as follows:

1704.1 General. Where application is made for construction as described in this section, the owner or the *registered design professional in responsible charge* acting as the owner's agent shall employ one or more *approved agencies* to perform inspections during construction on the types of work listed under Section 1704. These inspections are in addition to the inspections identified in Section 110.

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 act as the special inspector for the work designed by them, provided those personnel meet the qualification requirements of this section to the satisfaction of the *building official*. 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 Group U occupancies 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 portions of structures designed and constructed in accordance with the cold formed steel light-frame construction provisions of Section 2210.7 or the conventional light-frame construction provisions of Section 2308.

1704.1.1 Statement of special inspections. The applicant shall submit a statement of *special inspections* prepared by the *registered design professional in responsible charge* in accordance with Section 107.1 as a condition for issuance. This statement shall be in accordance with Section 1705.

Exceptions:

1. A statement of *special inspections* is not required for portions of structures designed and constructed in accordance with the cold formed steel light-frame construction provisions of Section 2210.7 or the conventional light frame construction provisions of Section 2308.
2. The statement of *special inspections* is permitted to be prepared by a qualified person *approved* by the *building official* for construction not designed by a *registered design professional*.

Commenter's Reason: At the Public Hearing in Baltimore, a floor modification was offered by the Steel Framing Alliance to add a reference to Section 2210.7 for construction of cold-formed steel light-frame structures using the prescriptive provisions of AISI S230. The modification was ruled out-of-order, leading to disapproval of the proposal. The IBC Structural Committee strongly indicated their desire to have the proposed Exception #4 to Section 1704.1 incorporate the floor modification.

In addition to providing the desired reference in Exception #4 of Section 1704.1, this public comment also makes the corresponding change in Exception #1 of Section 1704.1.1 to exempt cold-formed steel light frame construction from the requirement for a statement of special inspections. Furthermore, Exception #1 is amended to apply only to those portions of structures constructed using prescriptive methods. This clarifies that special inspections would still be required for those portions not designed and constructed using prescriptive method, e.g. deep foundations, helical piers, or structural steel.

Final Action:

AS

AM

AMPC_____

D

S121-09/10

1704.3-1704.3.3.3, Table 1704.3

Proposed Change as Submitted

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

1. Revise as follows:

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

Exceptions:

4. *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, and grade and mill test reports for the main stress-carrying elements are capable of being determined. Mill test reports shall be identifiable to the main stress-carrying elements when required by the approved construction documents.
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 members.~~
 - 2.5. ~~Welding of stairs and railing systems.~~

1704.3.1 Structural steel. Special inspection for structural steel shall be in accordance with the quality assurance inspection requirements of AISC 360.

1704.3.2 Steel construction other than structural steel. Special inspection for steel construction other than structural steel shall be in accordance with Table 1704.3 and this section.

~~1704.3.1~~ **1704.3.2.1 Welding.** Welding inspection and welding inspector qualification shall be in accordance with this section.

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

~~1704.3.1.2~~ **1704.3.2.1.1 Cold-formed steel.** Welding inspection and welding inspector qualification for cold-formed steel floor and roof decks shall be in accordance with AWS D1.3.

~~1704.3.1.3~~ **1704.3.2.1.2 Reinforcing steel.** Welding inspection and welding inspector qualification for reinforcing steel shall be in accordance with AWS D1.4 and ACI 318.

2. Delete without substitution:

~~1704.3.2 Details.~~ The special inspector shall perform an inspection of the steel frame to verify compliance with the details shown on the *approved construction documents*, such as bracing, stiffening, member locations and proper application of joint details at each connection.

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

1704.3.3.1 General. While the work is in progress, the special inspector shall determine that the requirements for bolts, nuts, washers and paint; bolted parts and installation and tightening in such standards are met. For bolts requiring pretensioning, the special inspector shall observe the preinstallation testing and calibration procedures when such procedures are required by the installation method or by project plans or specifications; determine that all plies of connected materials have been drawn together and properly snugged and monitor the installation of bolts to verify that the selected procedure for installation is properly used to tighten bolts. For joints required to be tightened only to the snug-tight condition, the special inspector need only verify that the connected materials have been drawn together and properly snugged.

1704.3.3.2 Periodic monitoring. Monitoring of bolt installation for pretensioning is permitted to be performed on a periodic basis when using the turn-of-nut method with matchmarking techniques, the direct tension indicator method or the alternate design fastener (twist-off bolt) method. Joints designated as snug-tight need be inspected only on a periodic basis.

1704.3.3.3 Continuous monitoring. Monitoring of bolt installation for pretensioning using the calibrated-wrench method or the turn-of-nut method without matchmarking shall be performed on a continuous basis.

3. Revise as follows:

1704.3.4 1704.3.2.2 Cold-formed steel trusses spanning 60 feet or greater. Where a cold-formed steel truss clear span is 60 feet (18 288 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 truss submittal package.

TABLE 1704.3

REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL

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	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. Snug-tight joints.	—	X	AISC 360, Section M2.5	1704.3.3
b. Pretensioned and slip-critical joints using turn-of-nut with matchmarking, twist-off bolt, or direct tension indicator methods of installation.	—	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 and cold-formed steel deck:				
a. For structural steel, identification markings to conform to AISC 360.	—	X	AISC 360, Section M5.5	
a.b. For other steel, identification markings to conform to ASTM standards specified in the approved construction documents.	—	X	Applicable ASTM material standards	
b.e. Manufacturers' certified test reports.	—	X		
4. Material verification of weld filler materials:				
a. Identification markings to conform to AWS specification in the approved	—	X	AISC 360, Section A3.5 and Applicable AWS A5	—

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a	IBC REFERENCE
construction documents.			documents	
b. Manufacturer's certificate of compliance required.	—	X	—	—
25. Inspection of welding:				
a. Structural steel and cold-formed steel deck:				
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 $\geq 5/16"$	X	—		
4) Plug and slot welds	X	—		
5) Single pass fillet welds $\leq 5/16"$	—	X		
16) Floor and roof deck welds.	—	X	AWS D1.3	
b. Reinforcing steel:				
1) Verification of weldability of reinforcing steel other than ASTM A 706.	—	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	—		
4) Other reinforcing steel.	—	X		
6. Inspection of steel frame joint details for compliance with approved construction documents:				
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: The 2010 edition of ANSI/AISC 360, *Specification for Structural Steel Buildings*, incorporates a new Chapter N, which addresses comprehensive quality control and quality assurance requirements for all structural steel construction. These requirements are similar in nature to those that were incorporated into the 2005 edition of AISC 341, Appendix Q. Those AISC 341 requirements are currently referenced in the 2009 edition of the IBC, Sections 1707 and 1708 for special inspection requirements in high-seismic applications. AISC 360-10, Chapter N provisions provide the foundation for the quality control and quality assurance requirements for general structural steel construction, with AISC 341-10, Chapter I (previously contained in AISC 341-05, Appendix Q) extending specific requirements to high-seismic applications.

AISC 360, Chapter N covers quality control requirements on the part of the structural steel fabricator and erector, as well as quality assurance requirements on the part of the owners inspecting and or testing agents. While AISC 360 addresses the total quality aspects of the structural steel project, the inspection requirements of the Quality Assurance Inspector can be equated to those specified for the Special Inspector under IBC Chapter 17.

The present Section 1704.3 addresses all forms of steel construction. The majority of the requirements in this section and Table 1704.3 pertain to structural steel construction. However, there are a few items which refer to cold-formed steel construction and rebar welding, which are not covered by AISC 360. The current special inspection requirements for structural steel as covered in Section 1704.3 and Table 1704.3 are recommended for deletion by this proposal; and, instead, a direct reference is made to the more detailed requirements of AISC 360, Chapter N. Requirements for special inspection of other forms of steel construction are left in a separate section of Section 1704.3.2, and in a reduced Table 1704.3, *Steel Construction Other than Structural Steel*.

Specifically, the topics currently in IBC Section 1704.3 are covered in AISC 360, Chapter N as follows:

Section 1704.3, Exception 2: The structural steel items are covered in AISC 360, Section N5.5. As for the cold formed steel exception applicable to roof and floor deck, it really is not correct and is recommended for deletion. Shop welding is typically used for a multi-skin closed cell deck, which would be a violation of the AWS D1.3 requirement that arc spot is only valid for deck to underlying structural members (D1.3, Clause 1.5.4). Multi-skin deck within itself appears to fall outside of the code itself and requires direct qualification by the manufacturer of their processes, potentially through testing rather than calculations. In reality, cold formed steel deck is sufficiently covered in Section 1704.3.2.1.1, Table 1704.3, and the reference to AWS D1.3.

Section 1704.3.2: AISC 360, Section N5.8.

Section 1704.3.3: AISC 360, Section N5.7(3)

Additionally, the topics currently in IBC Table 1704.3 are covered as in AISC 360, Chapter N as follows:

Table 1704.3, Item 1: AISC 360, Section N5.7 and Table N5.7-1.

Table 1704.3, Item 2: AISC 360, Section N5.7

Table 1704.3, Item 3a: AISC 360, Section N3.2 requires that the MTRs, as well as numerous other documents be made available for EOR review.

Table 1704.3, Item 4: AISC 360, Section N5.5 and Table N5.5-1

Table 1704.3, Item 5: AISC 360, Section N5.5

Table 1704.3, Item 6: AISC 360, Section N5.8

Also, Section 1704.3, Exception 1 is retained and modified to clarify the requirements. Often in practice, the "representative mill test reports" are supplied as described in the AISC Code of Standard practice. The added sentence on mill test reports allows for traceability when required by the construction documents, and defers to AISC 360 in other cases.

Please note, public review drafts of the 2010 AISC documents can be found on the AISC website (www.aisc.org). The public review period for AISC 360-10 is currently scheduled for 8/14/09 through 9/28/09 and the public review period for AISC 341-10 is currently scheduled for 9/11/09 through 10/26/09. It is anticipated that the 2010 editions of both AISC 360 and AISC 341 will be technically complete by the end of October 2009, with ANSI approval in March 2010 and publication in August 2010.

Cost Impact: There is no anticipated impact on the cost of construction.

ICCFILENAME: Manley-S13-1704.3

Public Hearing Results

Committee Action:

Approved as Submitted

Committee Reason: This proposal makes use of the more comprehensive inspection requirements for structural steel by referencing AISC 360 quality assurance inspections. Replacing the IBC provisions with this reference is similar to the reference to AISC 341 for steel seismic systems.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment 1:

Steven Winkel, FAIA, PE, and Kelly Cobeen, PE, SE, representing the Federal Emergency Management Agency/Building Seismic Safety Council Code Resource Support Committee (FEMA/BSSC CRSC) and Bonnie Manley, PE, representing the American Institute of Steel Construction (AISC), request Approval as Modified by this Public Comment.

Modify the proposal as follows:

1704.3 Steel construction. The *special inspections* for steel elements of buildings and structures shall be as required in this Section.

Exception: *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 and grade for the main stress-carrying elements are capable of being determined. Mill test reports shall be identifiable to the main stress-carrying elements when required by the approved construction documents.

1704.3.1 Structural steel. Special inspection for structural steel shall be in accordance with the quality assurance inspection requirements of AISC 360.

1704.3.2 Steel Construction other than structural steel. Special inspection for steel construction other than structural steel shall be in accordance with Table 1704.3 and this section.

1704.3.2.1 Welding. Welding inspection and welding inspector qualification shall be in accordance with this section.

1704.3.2.1.1 Cold-formed steel. Welding inspection and welding inspector qualification for cold-formed steel floor and roof decks shall be in accordance with AWS D1.3.

1704.3.2.1.2 Reinforcing steel. Welding inspection and welding inspector qualification for reinforcing steel shall be in accordance with AWS D1.4 and ACI 318.

1704.3.2.1.3 Other steel construction. Hot-rolled steel construction, other than structural steel covered in Section 1704.3.1, that has been designated in the statement of special inspections by the registered design professional in responsible charge as requiring special inspection, shall be subject to the welding inspection requirements of AWS D1.1. The welding shall be inspected on a periodic basis. As a minimum, such construction shall include H-piles and stair and railing systems.

1704.3.2.2 Cold-formed steel trusses spanning 60 feet or greater. Where a cold-formed steel truss clear span is 60 feet (18 288 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* truss submittal package.

**TABLE 1704.3
REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL**

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a	IBC REFERENCE
1. Material verification cold-formed steel deck:				
a. Identification markings to conform to ASTM standards specified in the approved construction documents.	—	X	Applicable ASTM material standards	
b. Manufacturers' certified test reports.	—	X		
2. Inspection of welding:				
a. Cold-formed steel deck:				
1) Floor and roof deck welds.	—	X	AWS D1.3	
b. Reinforcing steel:				
1) Verification of weldability of reinforcing steel other than ASTM A 706.	—	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	—		
4) Other reinforcing steel.	—	X		

For SI: 1 inch = 25.4 mm.

a. Where applicable, see also Section 1707.1, special inspection for seismic resistance

Commenter's Reason: Proposal S121-09/10, as originally written, inadvertently deleted a narrow sub-set of hot-rolled structural steel items that are intended to be covered by the special inspection requirements of IBC Chapter 17 but that fall outside the defined scope of AISC 360. Specifically, the scope of AISC 360 states the following in Section A1:

The *Specification for Structural Steel Buildings*, hereafter referred to as the Specification, shall apply to the design of the *structural steel* system, where the steel elements are defined in the AISC *Code of Standard Practice for Steel Buildings and Bridges*, Section 2.1.

In addition, the AISC 360 glossary defines structural steel as follows:

Structural steel. Steel elements as defined in Section 2.1 of the AISC *Code of Standard Practice for Steel Buildings and Bridges*.

In contrast, the 2009 IBC does not specifically define "structural steel." Rather, Chapter 22, *Steel*, includes definitions for *Cold Formed Steel Construction*, *Steel Joist*, and *Structural Steel Member*. The definition of *Structural Steel Member* is as follows:

STEEL MEMBER, STRUCTURAL. Any steel structural member of a building or structure consisting of a rolled steel structural shape other than cold-formed steel, or steel joist members.

And, IBC-09, Section 2205, *Structural Steel*, references AISC 360 for the "...design, fabrication and erection of structural steel for buildings and structures..." (2205.1).

However, IBC-09, Section 1704.3, Exception 2 includes welding special inspection requirements for steel construction other than steel deck and reinforcing steel that is not purely defined as "structural steel," such as stair and railing systems. The deletion of this exception by Proposal S121-09/10 removed these necessary special inspection requirements.

This public comment reinstates the welding special inspection for such elements, when identified in the statement of special inspection by the design professional in responsible charge. In practice, structural engineers often use the provisions of AISC 360 to design steel members using hot-rolled steel shapes, plates, and bars that resist loads and forces, even when such members do not meet AISC's definition of structural steel. Rather than continue to use AISC 360 outside of its intended scope, this comment references AWS D1.1 and specifies "periodic" frequency for the welding inspection, which is consistent with the provisions inadvertently deleted by S121. The minimum list of steel elements includes both H-piles and stair and railing systems. The inclusion of steel H-piles is consistent with the requirements of Table 1704.8, *Required Verification and Inspection of Driven Deep Foundation Elements*, Item 5.

Public Comment 2:

Alan Robinson, SE and Art Dell, PE, representing Structural Engineers Association of California, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1704.3 Steel construction. The *special inspections* for steel elements of buildings and structures shall be as required in this Section.

Exception: *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 and grade for the main stress-carrying elements are capable of being determined. Mill test reports shall be identifiable to the main stress-carrying elements when required by the approved construction documents.

1704.3.1 Structural steel. Special inspection for structural steel shall be in accordance with the quality assurance inspection requirements of AISC 360.

Exception: The special inspection for complete and partial penetration groove welds, multipass fillet welds, and fillet welds greater than 5/16 inch (7.9 mm) shall be continuous special inspection as defined in this code.

1704.3.2 Steel Construction other than structural steel. Special inspection for steel construction other than structural steel shall be in accordance with Table 1704.3 and this section.

1704.3.2.1 Welding. Welding inspection and welding inspector qualification shall be in accordance with this section.

1704.3.2.1.1 Cold-formed steel. Welding inspection and welding inspector qualification for cold-formed steel floor and roof decks shall be in accordance with AWS D1.3.

1704.3.2.1.2 Reinforcing steel. Welding inspection and welding inspector qualification for reinforcing steel shall be in accordance with AWS D1.4 and ACI 318.

1704.3.2.2 Cold-formed steel trusses spanning 60 feet or greater. Where a cold-formed steel truss clear span is 60 feet (18 288 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* truss submittal package.

**TABLE 1704.3
REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION OTHER THAN STRUCTURAL STEEL**

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a	IBC REFERENCE
1. Material verification cold-formed steel deck:				
a. Identification markings to conform to ASTM standards specified in the approved construction documents.	—	X	Applicable ASTM material standards	
b. Manufacturers' certified test reports.	—	X		
2. Inspection of welding:				
a. Cold-formed steel deck:				
1) Floor and roof deck welds.	—	X	AWS D1.3	
b. Reinforcing steel:				
1) Verification of weldability of reinforcing steel other than ASTM A 706.	—	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	—		
4) Other reinforcing steel.	—	X		

Commenter's Reason: As approved at the Code Development Hearing, S121-09/10 will result in a reduction of the frequency of welding special inspection.

The frequency of special inspections has always been defined in the International Building Code (IBC) by the use of the terms "continuous" and "periodic". Continuous special inspection is invoked for work that cannot be verified when complete, such as concrete placement and multipass welding. Periodic inspection is invoked for work that can be verified adequately by initial inspection of materials and procedures, some in-process inspection, and final inspection of the completed work.

Continuous special inspection has been required by Table 1704.3 for complete and partial penetration groove welds, multipass fillet welds, fillet welds > 5/16", and plug and slot welds.

Proposal S121-09/10 has removed the special inspections for welding and bolting of structural steel from IBC Chapter 17 in favor of the quality assurance provisions of the new Chapter N in AISC 360-10.

Chapter N of AISC 360-10 does not use "continuous" or "periodic" to describe the frequency of welding inspection. Rather, the terms **Observe** and **Perform** are applied to the welding inspection tasks identified in three tables (for Before, During, and After welding).

Observe and Perform are defined as follows (AISC 360-10, N5.4 Inspection of Welding):

O – Observe these items on a random basis. Operations need not be delayed pending these inspections.

P – Perform these tasks for each welded joint or member.

The Observe level of inspection is applied to all of the Before and During Welding tasks, with the exception of verifying that WPSs are available, and that manufacturer certifications for welding consumables are available. Inspection tasks such as verifying fit-up of groove welds and fillet welds (before welding), verifying that the WPS is followed and that interpass cleaning and quality are adequate (during welding), would thus only be performed on a "random basis".

This would allow welding on any particular joint to be performed without any in-process inspection, once the welding inspector has verified the materials, procedures, and welder skill. In fact, there is no requirement that the welding inspector be on site while welding is performed.

This is not appropriate for complete and partial penetration groove welds, multipass fillet welds, and fillet welds > 5/16" which currently require continuous inspection.

This public comment corrects this deficiency by adding an exception to the language of 1704.3.1 which invokes the quality assurance inspection requirements of AISC 360.

Public Comment 3:

Homer Maiel, PE, CBO, City of San Jose, CA, representing ICC Tri-Chapter (Peninsula, East Bay, Monterey Bay Chapters), requests Disapproval.

Commenter's Reason: As approved at the Code Development Hearing, S121-09/10 permits a significant reduction of the frequency and thoroughness of inspections that must be conducted for multi-pass welds by the special inspector. This reduction was not disclosed in the proponent's reason statement nor was it discussed by the IBC Structural Committee that voted to approve the proposal.

Continuous special inspection has been required by Table 1704.3 since the inception of the IBC for complete and partial penetration groove welds, multipass fillet welds, and single pass fillet welds greater than 5/16". Plug and slot welds were added to that list in the 2009 edition.

Proposal S121-09/10 removes most of Table 1704.3 and replaces that information by referencing the quality assurance (QA) provisions of the new Chapter N in AISC 360-10.

Chapter N of AISC 360-10 does not use the traditional code terminology of “continuous” or “periodic” to describe the frequency of welding inspection. Rather, the terms Observe (O), and Perform (P) are applied to the welding inspection tasks identified in three separate tables that address the inspection tasks to be performed Before, During, and After welding.

Observe and Perform are defined in AISC 360-10, Section N5.4 Inspection of Welding as follows:

- O – Observe these items on a random basis. Operations need not be delayed pending these inspections.
- P – Perform these tasks for each welded joint or member.

The Observe level of inspection is applied to all of the “Before” and “During” welding inspection tasks, with the exception of verifying that welding procedure specifications (WPSs) are available, and that manufacturer certifications for welding consumables are available. Inspection tasks such as verifying fit-up of groove welds and fillet welds (before welding), and verifying that the WPS is followed and that interpass cleaning and quality (during welding) are adequate, would thus only be performed on a “random basis”. AISC 360 Commentary indicates that the inspector would not need to be on site when welding is performed, provided he or she has verified the materials, processes, and the welder’s skill level. This is clearly very different than current IBC requirements that specify continuous inspection yet the reason statement provided for S121 did not acknowledge this significant change nor did it provide any insight into why such a reduction might be warranted.

The Tri-Chapter requests that the approval given to S121-09/10 be rescinded so that welding special inspection is not reduced, until such time that this reduction can be fully justified and be openly discussed by the members of ICC that enforce the current provisions of the code.

Final Action: AS AM AMPC D

S124-09/10

1704.6, Table 1704.6 (New), 1706.2, 1707.3, 1704.3.5 (New), Table 1704.3, 1706.3, 1707.4

Proposed Change as Submitted

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

1. 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 prefabricated and site built wood construction and assemblies including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs shall be as required by this section and Table 1704.6.

Exceptions:

1. Special inspection of wood construction for buildings and structures in Occupancy Category I shall not be required.
2. Special inspection of wood construction for buildings and structures in Occupancy Category II that are 3 or less stories in height shall not be required.

2. Add new Table as follows:

**TABLE 1704.6
REQUIRED VERIFICATION AND INSPECTION OF WOOD CONSTRUCTION**

<u>VERIFICATION AND INSPECTION</u>	<u>CONTINUOUS</u>	<u>PERIODIC</u>
<u>1. Verify that grade stamp on framing lumber, plywood and OSB panels conforms to the construction documents.</u>		<u>X</u>
<u>2. Verify that wood connections including nail quantity, size and spacing; bolt size and location anchor bolt size, spacing and location; tie down size location and configuration; beam hangers and framing anchors conform to the approved construction documents.</u>		<u>X</u>
<u>3. Inspect details of wood framing including framing layout, member sizes, blocking, bridging and bearing lengths.</u>		<u>X</u>
<u>4. Inspect diaphragms and shear walls to verify that wood structural panel sheathing is of the grade and thickness indicated on the approved construction documents and the nominal size of framing</u>		<u>X</u>

members at adjoining panel edges, the nail or staple diameter and length, are as indicated on the approved construction documents.		
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3. Revise as follows:

1706.2 Structural wood. Continuous special inspection is required during field gluing operations of elements of the main wind-force-resisting system. ~~Periodic special inspection is required for nailing, bolting, anchoring and other fastening of components within the main wind force-resisting system, including wood shear walls, wood diaphragms, drag struts, braces and hold-downs.~~

Exception: ~~Special inspection is not required for wood shearwalls, 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.~~

1707.3 Structural wood. Continuous special inspection is required during field gluing operations of elements of the seismic-force-resisting system. ~~Periodic special inspection is required for nailing, bolting, anchoring and other fastening of components within the seismic force-resisting system, including wood shear walls, wood diaphragms, drag struts, braces, shear panels and hold-downs.~~

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

4. Add new text as follows:

1704.3.5 Cold-formed steel light-frame construction. Special Inspections for prefabricated and site built cold-formed steel light-frame construction and assemblies including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs shall be as required by this section and Table 1704.3.

Exceptions:

1. Special inspection of cold-formed steel light-frame construction for buildings and structures in Occupancy Category I shall not be required.
2. Special inspection of cold-formed steel light-frame construction for buildings and structures in Occupancy Category II that are 3 or less stories in height shall not be required.

5. Revise as follows:

**TABLE 1704.3
REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION**

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a	IBC REFERENCE
3. Material verification of structural steel, cold-formed steel light-frame construction and cold-formed steel deck:				
a. For structural steel, identification markings to conform to AISC 360	–	X	AISC 360, Section M5.5	
b. For other steel, identification markings to conform to ASTM standards specified in the approved construction documents.	–	X	Applicable ASTM material standards	
c. Manufacturer's certified test reports.	–	X		
5. Inspection of welding:				
a. Structural steel, cold-formed steel light-frame construction and cold-formed steel deck:				
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	–		
5) Single-pass fillet welds, 5/16"	–	X		

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a	IBC REFERENCE
6) Floor and roof deck welds.	–	X	AWS D1.3	
7) <u>Cold-formed steel light-frame construction welds</u>		<u>X</u>	<u>AWS D1.3</u>	
6. Inspection of steel frame joint details for compliance with approved construction documents:				
a. Details such as bracing, <u>drag struts</u> and stiffening.	–	X	–	1704.3.2
b. Member locations.	–	X		
c. Application of joint details at each connection.	–	X		
d. <u>Mechanical connections for cold-formed steel light-frame construction including screws, powder actuated fasteners, bolts, anchor bolts, tie downs, anchors and other fastening components</u>		<u>X</u>	<u>Applicable ASTM material standards</u>	

(Portions of table not shown remain unchanged)

6. Delete without substitution:

~~**1706.3 Cold-formed steel 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 shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.~~

~~**Exception:** *Special inspection* is not required for cold-formed steel light frame shear walls, braces, diaphragms, collectors (drag struts) and hold-downs 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.).~~

~~**1707.4 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 shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.~~

~~**Exception:** *Special inspection* is not required for cold-formed steel light frame shear walls, braces, diaphragms, collectors (drag struts) and hold-downs 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) o.c.~~

Reason: NCSEA believes that light frame construction in wood and cold formed steel have become more commonly used for load bearing applications of significant height and in regions with moderate and high seismic and wind concerns. These types of construction should be subject to Special Inspections in a similar manner and to a comparable extent as other systems such as concrete, structural steel and masonry. The Code is vague in the requirements for these systems resulting in confusion as to what special inspections and to what extent special inspection is required. This proposal clarifies requirements to be consistent across both systems and to improve the consistency of special inspections across all the major structural materials.

The emphasis of the existing special inspection requirements for wood framed construction is on shop inspection of fabricated wood assemblies 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. Deletion of the exception under 1707.3 coordinates with this change.

Exceptions are provided to limit the applicability of these provisions to exclude single and two family dwellings, small commercial, agricultural and buildings of lesser occupancies.

Sections 1706.2, 1706.3, 1707.3 and 1707.4 are revised because the provisions deleted from these sections are now covered in the new or revised tables. The exceptions are deleted to be consistent with the proposal.

This proposal contains provisions addressing both wood frame and cold-formed steel light-frame construction together. This is an effort to address both systems in one change therefore avoiding any perception of one system having an advantage over the other regarding special inspection.

There will be some increase in construction cost due to the increased special inspection that will take place. However, the improved field quality assurance will improve safety and reduced field errors resulting in a savings in construction cost and schedule. The improved public safety far outweighs any minor increase there may be in construction cost.

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

ICCFILENAME: Harman-S2-1704

Public Hearing Results

Committee Action:

Disapproved

Committee Reason: Based on the historical performance of light-frame construction of wood and cold-formed steel, the proposed changes in special inspections were too substantial to make without better substantiation by the proponent. There was nothing in the way of case studies, calculation or rational analysis offered to the committee. Additionally the proponent’s rather extensive floor modification would indicate that this proposal needs work before it can be approved. Clarification of inspection for prefabricated structural assemblies and components may be necessary but these need to be clearer so that it can be implemented both with building inspectors and third party inspectors. Since the proposal is getting into new territory, it would be preferable to treat wood and cold-formed steel separately so they can be discussed and voted on individually.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment:

D. Kirk Harman, PE, SE, SECB , The Harman Group, Inc, representing The National Council of Structural Engineers Associations (NCSEA) Code Advisory Committee, Quality Assurance and Special Inspection Subcommittee requests Approval as Modified by this Public Comment.

Replace the proposal 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 prefabricated and site built wood construction and assemblies including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs shall be as required by this section and Table 1704.6.

Exceptions:

1. Special inspection of wood construction for buildings and structures in Occupancy Category I shall not be required.
2. Special inspection of wood construction for buildings and structures in Occupancy Category II that are 3 or less stories in height above grade plane and that are not included in Sections 1706 or 1707, shall not be required.

**TABLE 1704.6
REQUIRED VERIFICATION AND INSPECTION OF WOOD CONSTRUCTION**

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC
1. <u>Verify that grade stamp on framing lumber, plywood and OSB panels conforms to the construction documents.</u>		X
2. <u>Verify that wood connections including nail quantity, size and spacing; bolt size and location anchor bolt size, spacing and location; tie down size location and configuration; beam hangers and framing anchors conform to the approved construction documents.</u>		X
3. <u>Inspect details of wood framing including framing layout, member sizes, blocking, bridging and bearing lengths.</u>		X
4. <u>Inspect diaphragms and shear walls to verify that wood structural panel sheathing is of the grade and thickness indicated on the approved construction documents and the nominal size of framing members at adjoining panel edges, the nail or staple diameter and length, are as indicated on the approved construction documents.</u>		X

1706.2 Structural wood. Continuous special inspection is required during field gluing operations of elements of the main wind-force-resisting system. Periodic special inspection is required for nailing, bolting, anchoring and other fastening of components within the main wind-force-resisting system, including wood shear walls, wood diaphragms, drag struts, braces and hold-downs.

Exception: For buildings and structures in Occupancy Category I, or in Occupancy Category II and 3 or less stories in height above grade plane, special inspection is not required for wood shearwalls, 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.

1707.3 Structural wood. Continuous special inspection is required during field gluing operations of elements of the seismic-force-resisting system. Periodic special inspection is required for nailing, bolting, anchoring and other fastening of components within the seismic-force-resisting system, including wood shear walls, wood diaphragms, drag struts, braces, shear panels and hold-downs.

Exception: For buildings and structures in Occupancy Category I, or in Occupancy Category II and 3 or less stories in height above grade plane, special inspection is not required for wood shearwalls, shear panels and diaphragms, including nailing, bolting, anchoring and other fastening to other components of the seismic-force-resisting system, where the fastener spacing of the sheathing is more than 4 inches (102 mm) on center (o.c.).

1704.3.5 Cold-formed steel light-frame construction. Special Inspections for prefabricated and site built cold-formed steel light-frame construction and assemblies including shear walls, braces, diaphragms, collectors (drag struts) and hold-downs shall be as required by this section and Table 1704.3.

Exceptions:

1. Special inspection of cold-formed steel light-frame construction for buildings and structures in Occupancy Category I shall not be required.
2. Special inspection of cold-formed steel light-frame construction for buildings and structures in Occupancy Category II that are 3 or less stories in height above grade plane and that are not included in Sections 1706 or 1707, shall not be required.

**TABLE 1704.3
REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION**

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD ^a	IBC REFERENCE
3. Material verification of structural steel, cold-formed steel light-frame construction and cold-formed steel deck:				
a. For structural steel, identification markings to conform to AISC 360	-	X	AISC 360, Section M5.5	
b. For cold-formed steel light-frame construction, identification markings to conform to AISI S200 as specified in the approved construction documents.	-	<u>X</u>	<u>AISI S200</u> <u>Section A5.4</u>	
c. For other steel, identification markings to conform to ASTM standards specified in the approved construction documents.	-	X	Applicable ASTM material standards	
d. Manufacturers certified test reports.	-	X		
5. Inspection of welding:				
a. Structural steel, cold-formed steel light-frame construction and cold-formed steel deck:				
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	-		
5) Single-pass fillet welds 5/16"	-	X		
6) Floor and roof deck welds.	-	X	AWS D1.3	
7) Cold-formed steel light-frame construction welds		<u>X</u>	<u>AWS D1.3</u>	
6. Inspection of steel frame joint details for compliance with approved construction documents:				
a. Details such as bracing, drag struts and stiffening.	-	X	-	1704.3.2
b. Member locations.	-	X		
c. Application of joint details at each connection.	-	X		
d. Mechanical connections for cold-formed steel light-frame construction including screws, powder actuated fasteners, bolts, anchor bolts, tie downs, anchors and other fastening components		<u>X</u>	<u>AISI S200</u> <u>Section D</u>	

(Portions of table not shown are unchanged).

1706.3 Cold-formed steel 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 shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

Exception: For buildings and structures in Occupancy Category I, or in Occupancy Category II and 3 or less stories in height above grade plane, special inspection is not required for cold-formed steel light-frame shear walls, braces, diaphragms, collectors (drag struts) and hold-downs 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.).

1707.4 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 shear walls, braces, diaphragms, collectors (drag struts) and hold-downs.

Exception: For buildings and structures in Occupancy Category I, or in Occupancy Category II and 3 or less stories in height above grade plane, special inspection is not required for cold-formed steel light-frame shear walls, braces, diaphragms, collectors (drag struts) and hold-downs 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) o.c.

Commenter's Reason: The above revision to the proposal addresses comments received from various interested parties. This version maintains all Special Inspections for light frame systems in high wind and high seismic situations, (Sections 1706 and 1707). The purpose of the previous floor modification was to address these areas. The above version now reflects these changes.

The above revision also adds reference to AISI S200 for inspection procedures. This document is currently referenced in the IBC for design of cold formed steel light frame construction.

The proposal seeks to capture the buildings constructed with light frame systems that are over 3 stories in height. For this reason buildings in Occupancy Category I, or in Occupancy Category II and 3 or less stories in height above grade plane are excluded from the requirements. This will exclude nearly all single family and two family dwellings, most low-rise multi-family residential buildings and light frame low-rise commercial buildings. Therefore, the vast majority of buildings constructed with light frame wood or cold formed steel systems will not be included in the proposed Special Inspection requirements. It is only those taller buildings, 4 stories or more in height, which would be subject to the proposed Special Inspections.

Each comment by the committee contained in the Public Hearing Results is addressed below individually:

Based on the historical performance of light-frame construction of wood and cold-formed steel, the proposed changes in special inspections were too substantial to make without better substantiation by the proponent.

To date light frame construction has been used predominantly in buildings 3 stories or less in height. These buildings would remain exempt under this proposal; therefore, the historical performance of these buildings has been recognized. It is only recently that light frame construction has been pushed to heights of nine stories and more. These taller buildings are a small number of the total buildings constructed each year with light frame systems therefore the proposed changes are not substantial across all light frame buildings. The proposed Special Inspections are limited to the small number of light frame structures that are at risk under the current IBC. NCSEA represents over 11,000 structural engineers throughout the United States. The structural engineers in our Member Organizations have voiced concern for the safety of these taller buildings as these systems are pushed to the limits of their capacity while not subject to the same level of Special Inspections as other structural systems used in the same height buildings. The only reason that there has not been a failure is that engineers designing these taller buildings have specified a higher level of inspection than the IBC requires. However, this is not a mandated inspection and the situation presents a very high risk to the public.

There was nothing in the way of case studies, calculation or rational analysis offered to the committee.

If the ICC waited for buildings to collapse before making changes to the Code, seismic design requirements would be twenty years out of date. This proposal is based on the observations of numerous structural engineers working in the field and observing these structures on a daily basis. The risk is real. The day that one of these structures collapses due to lack of inspection will be one day too late for the lives lost.

Additionally the proponent's rather extensive floor modification would indicate that this proposal needs work before it can be approved.

The floor modification has been incorporated into the revised proposal represented in this public comment.

Clarification of inspection for prefabricated structural assemblies and components may be necessary but these need to be clearer so that it can be implemented both with building inspectors and third party inspectors.

The proposal addresses both prefabricated assemblies and components as well as field constructed structural systems. The requirements are set forth in the same format as the current requirements for other systems.

Since the proposal is getting into new territory, it would be preferable to treat wood and cold-formed steel separately so they can be discussed and voted on individually.

To avoid any argument that one system (wood or cold formed steel) would have a competitive disadvantage relative to the other as a result of improved inspection of one and not the other, the two materials are covered in one proposal. Both systems of light frame-frame construction are being used in taller buildings previously envisioned by the current Special Inspection requirements.

This is not "new territory" for Special Inspection. Special inspections have been required for decades for all other buildings over three stories in height constructed using structural steel, concrete and masonry. Until recently light frame construction was rarely used for buildings over 3 stories so Special Inspection for these systems were never addressed. These systems are now "getting into new territory" and should be treated in the same manner as any other structural system employed in buildings over 3 stories.

One additional argument against providing Special Inspections for light frame construction is that the Inspections of Chapter 1 of the IBC are sufficient and no special expertise is required for these inspections.

If a four story building is constructed with structural steel, it is subject to the rigorous Special Inspections of Chapter 17. If the same building is constructed using wood frame or cold formed steel, there is no requirement for Special Inspections and the only inspections are those performed by the Building Official. This places an unreasonable burden on the Building Official to inspect at a much higher level than is anticipated by the provisions of Chapter 1. The AWS welding inspection requirements for cold formed steel are well outside the responsibility of the Building Official yet the Code has no requirements for Special Inspection.

If an eight story masonry bearing wall building is constructed with precast concrete floor units, it is subject to the rigorous Special Inspections of Chapter 17. If that same building is constructed using cold formed steel bearing walls and precast concrete floor units, there is no requirement for Special Inspections of the bearing wall system and the only inspections are those performed by the Building Official. This places an even more unreasonable burden on the Building Official to inspect at a much higher level than is anticipated by the provisions of Chapter 1. There is a serious risk to public safety when an eight story building can be constructed and the only inspections are those by a Building Official, in the same manner as a single family home. These are buildings such as senior living facilities, student housing, apartments, and hotels.

This proposal seeks to plug a serious hole in the Special Inspections requirements of the IBC. It seeks to apply the same level of Special Inspection to all structural systems used in buildings over three stories in height.

Final Action: AS AM AMPC D

S127-09/10
1704.15 (New), Chapter 35

Proposed Change as Submitted

Proponent: William E. Koffel, Koffel Associates, Inc., representing Firestop Contractors International Association

1. Add new text as follows:

1704.15 Fire-resistant penetrations and joints. In buildings assigned an Occupancy Category of III or IV in accordance with Section 1604.5, special inspections for through penetrations, membrane penetration firestops, fire resistant joint systems, and perimeter fire barrier systems of the types specified in Sections 713.3.1.2, 713.4.1.2, 714.3 and 714.4 shall be in accordance with Sections 1704.15.1 or 1704.15.2.

1704.15.1 Penetration firestops. Inspections of penetration firestop systems of the types specified in Sections 713.3.1.2 and 713.4.1.2 shall be conducted by an approved inspection agency in accordance with ASTM E 2174.

1704.15.2 Fire-resistant joint systems. Inspection of fire resistant joint systems of the types specified in Sections 714.3 and 714.4 shall be conducted by an approved inspection agency in accordance with ASTM E 2393.

2. Add standards to Chapter 35 as follows:

ASTM International

E 2174-09 Standard Practice for On-Site Inspection of Installed Fire Stops

E 2393-09 Standard Practice for On-Site Inspection of Installed Fire Resistive Joint Systems and Perimeter Fire Barrier

Reason: Through penetration and joint firestop systems are critical to maintaining the fire resistance rating of fire resistance rated construction, including fire barriers, smoke barriers, and fire resistance rated horizontal assemblies. Every construction trade has very unique requirements that are specific to that trade, with technical knowledge built through cumulative continued work in the trade. Firestopping is no different. The concept has been proposed in the past and some felt the scope was too broad. Therefore, the scope of the proposed requirement has been limited to those buildings that represent a substantial hazard to human life in the event of a system failure or that are considered to be essential facilities in accordance with Table 1604.5.

In order to meet the requirements of a listed firestop system from the UL Fire Resistance, Intertek, FM Approvals or other testing laboratory directories, a 'zero tolerance' systems installation protocol is needed, or a system can be violated and rendered ineffective. The violation can be as small as a minor annular space size variance, joint width exceeding system requirements, penetrating item size or type not as listed. There are no typical 'construction tolerances' allowed in firestopping for fire and life safety.

Firestop Systems must be selected from the listing directories, then applied in the correct manner, in the right place. With endless variations to penetrating items, hole sizes and shapes, plus the classified systems to restore the fire ratings, firestop systems selection looks easy to the untrained eye.

The UL Fire Resistance Directories have over 8,500 listed firestop systems, each with variations that multiplies possible systems for a building exponentially. Systems selection is not a 'generic process'. Systems selection is an exacting exercise by skilled contractors who submit appropriate systems for approval, then communicate these systems to the educated firestop – containment workers they employ...which becomes the inspection document for a qualified inspector of firestop systems to leading documents such as International Accreditation Services Accreditation Criteria, AC 291, section 6.11, Firestop Systems.

Should a penetration or joint condition in the field vary from the system design listing from the directories, the firestop system may not perform as intended, opening risk to the structure, and the occupants on the other side of the fire. Structurally, the floor, floor-ceiling or wall assemblies are not tested with unprotected holes with penetrating items or joints allowing fire attack to take place from both sides at once. They are tested with fire attack from one side, with all openings and penetrating items and joints firestopped.

On construction projects, there are three ways firestopping is installed currently. First, the 'he or she who pokes the hole fills it with firestopping' takes place, about 1/3 the time. A specialty firestop contractor installs for about another 1/3 of installations. The final 1/3 is a combination of specialty firestop contractors and the 'he or she who pokes the hole fills it' method. In other words, about 1/2 of the installations are installed by companies who most likely do not understand firestop systems selection nor the zero tolerance installation protocol. And, with the 20+ trades who potentially touch firestopping, many who perform the work as a 'sideline', the potential for a mistake increases exponentially when inexperienced companies install firestopping. However, firestopping is a complex operation, just like any other trade. Mastering more than one trade by attending a 30 minute to 16 hour class is nearly impossible for workers of any trade background.

In simple terms, inadequate firestopping makes the fire resistance rated floor or wall assembly become swiss cheese like, and not representative of testing. The risks of inadequate firestopping are apparent due to the many trades who install firestopping as a sideline...who just don't get the 'zero tolerance' systems oriented approach needed to get firestopping done right. Inspection to ASTM E 2174 and ASTM E 2393 brings a needed check to this important discipline, whether a FCIA Member specialty firestop contractor is installing or not.

Cost Impact: This will increase cost of construction when a contractor installing firestopping does not understand the zero tolerance protocol for firestopping. It will not increase the cost of construction when a contractor knowledgeable in the zero tolerance protocol for firestopping is used.

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM E2174-09 and ASTM E2393-09, 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 September 24, 2009.

ICCFILENAME: KOFFELL-S3-1704.15 NEW

Public Hearing Results

This code change was contained in the errata posted on the ICC website. Please go to <http://www.iccsafe.org/cs/codes/Pages/09-10ProposedChanges.aspx>.

Note: The following analysis was not in the Code Change monograph but was published on the ICC website at <http://www.iccsafe.org/cs/codes/Documents/2009-10cycle/ProposedChanges/Standards-Analysis.pdf> :

Analysis: Review of proposed new standards ASTM E 2174 and ASTM E 2393 indicated that, in the opinion of ICC Staff, the standards comply with ICC standards criteria.

Committee Action:

Approved as Modified

Modify the proposal as follows:

1704.15 Fire-resistant penetrations and joints. In buildings assigned an Occupancy Category of III or IV in accordance with Section 1604.5, special inspections for through penetrations, membrane penetration firestops, fire resistant joint systems, and perimeter fire barrier systems ~~of the types specified in tested and listed in accordance with~~ Sections 713.3.1.2, 713.4.1.2, 714.3 and 714.4 shall be in accordance with Sections 1704.15.1 or 1704.15.2.

1704.15.1 Penetration firestops. Inspections of penetration firestop systems ~~of the types specified in tested and listed in accordance with~~ Sections 713.3.1.2 and 713.4.1.2 shall be conducted by an approved inspection agency in accordance with ASTM E 2174.

1704.15.2 Fire-resistant joint systems. Inspection of fire resistant joint systems ~~of the types specified in tested and listed in accordance with~~ Sections 714.3 and 714.4 shall be conducted by an approved inspection agency in accordance with ASTM E 2393.

(Portions of the proposal not shown remain unchanged)

Committee Reason: The committee agreed that these installations were critical and that special inspections should be required for these installations in buildings assigned an Occupancy Category of III or IV. The modification more appropriately identifies the systems as those that are tested and listed.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment 1:

Dave Frable, U.S. General Services Administration, requests Disapproval.

Commenter's Reason: We do not concur with the action of the Fire Safety Code Committee to require the need for special inspections to be conducted by an approved inspection agency in accordance with ASTM E 2174 for all penetration firestop systems and ASTM E 2393 for all fire-resistant joint systems in all Occupancy Category Class III and IV buildings.

Typically, special inspections are only required for complex engineering systems or systems that are unusual in nature. It is our opinion that penetration firestop systems and fire resistant joint systems typically occur on every new construction project and are currently adequately addressed in Sections 110.3.6 and 110.3.8. Therefore, the need to require special inspections in all Occupancy Category Class III and IV buildings is not warranted.

In addition, the proponent has failed to provide any technical data or life loss data to substantiate that if the penetration firestop systems and fire resistant joint systems fail within any of the Occupancy Category Class III and IV buildings, a substantial hazard to human life would occur (e.g., an office building having an occupancy greater than 5000, a fire station, a police station, etc.).

It should also be pointed out that ASTM E 2174 and ASTM E 2393 do not provide any minimum qualification requirements for an "inspection agency" but does have qualification requirements for the inspector. For example, one acceptable qualification includes having a minimum of two years experience in construction field inspections and have education, credentials, and experience acceptable to the authorizing authority (i.e., architect, engineer, building owner). Therefore, we feel that these requirements should be in the project specification and not the Code.

Lastly, we also disagree with the proponent that the requirement for special inspections will not increase construction costs.

Public Comment 2:

Steve Orlowski, National Association of Home Builders (NAHB), requests Disapproval.

Commenter's Reason: The authority having jurisdiction, under the auspice of section 1704.15 as found in the current 2009 IBC, already has the authority to require certain methods and materials to be certified by a third party. The proponent has provide no technical justification nor any historical data showing that AHJ or their inspectors are unqualified to conduct these inspection or have failed in their duties due to a loss of any magnitude. In the proponent's written testimony, there are numerous examples referencing the misapplication of products by unqualified installers, yet the proposed change takes away the ability for the AHJ to perform inspections on these products unless they are an approved inspection agency in accordance with the two ASTM standards, as referenced. NAHB requests the final assembly to reject this proposal given that the AHJ already has the option to request special inspections be conducted if they are unfamiliar with products and that this needlessly increases the number of special inspection required by the code.

Final Action: AS AM AMPC _____ D

S128-09/10

1704.15 (New), Chapter 35

Proposed Change as Submitted

Proponent: William E. Koffel, Koffel Associates, Inc., representing Firestop Contractors International Association

1. Add new text as follows:

1704.15 Fire-resistant penetrations and joints. In buildings having occupied floors located more than 75 feet (22860 mm) above the lowest level of fire department vehicle access, special inspections for through penetrations, membrane penetration firestops, fire resistant joint systems, and perimeter fire barrier systems of the types specified in Sections 713.3.1.2, 713.4.1.2, 714.3 and 714.4 shall be in accordance with Sections 1704.15.1 or 1704.15.2.

1704.15.1 Penetration firestops. Inspections of penetration firestop systems of the types specified in Sections 713.3.1.2 and 713.4.1.2 shall be conducted by an approved inspection agency in accordance with ASTM E 2174.

1704.15.2 Fire-resistant joint systems. Inspection of fire resistant joint systems of the types specified in Sections 714.3 and 714.4 shall be conducted by an approved inspection agency in accordance with ASTM E 2393.

2. Add standards to Chapter 35 as follows:

ASTM International

E 2174-09 Standard Practice for On-Site Inspection of Installed Fire Stops

E 2393-09 Standard Practice for On-Site Inspection of Installed Fire Resistive Joint Systems and Perimeter Fire Barrier

Reason: Through penetration and joint firestop systems are critical to maintaining the fire resistance rating of fire resistance rated construction, including fire barriers, smoke barriers, and fire resistance rated horizontal assemblies. Every construction trade has very unique requirements that are specific to that trade, with technical knowledge built through cumulative continued work in the trade. Firestopping is no different. The concept has been proposed in the past and some felt the scope was too broad. Therefore, the scope of the proposed requirement has been limited to high-rise buildings.

In order to meet the requirements of a listed firestop system from the UL Fire Resistance, Intertek, FM Approvals or other testing laboratory directories, a 'zero tolerance' systems installation protocol is needed, or a system can be violated and rendered ineffective. The violation can be as small as a minor annular space size variance, joint width exceeding system requirements, penetrating item size or type not as listed. There are no typical 'construction tolerances' allowed in firestopping for fire and life safety.

Firestop Systems must be selected from the listing directories, then applied in the correct manner, in the right place. With endless variations to penetrating items, hole sizes and shapes, plus the classified systems to restore the fire ratings, firestop systems selection looks easy to the untrained eye.

The UL Fire Resistance Directories have over 8,500 listed firestop systems, each with variations that multiplies possible systems for a building exponentially. Systems selection is not a 'generic process'. Systems selection is an exacting exercise by skilled contractors who submit appropriate systems for approval, then communicate these systems to the educated firestop – containment workers they employ...which becomes the inspection document for a qualified inspector of firestop systems to leading documents such as International Accreditation Services Accreditation Criteria, AC 291, section 6.11, Firestop Systems.

Should a penetration or joint condition in the field vary from the system design listing from the directories, the firestop system may not perform as intended, opening risk to the structure, and the occupants on the other side of the fire. Structurally, the floor, floor-ceiling or wall assemblies are not tested with unprotected holes with penetrating items or joints allowing fire attack to take place from both sides at once. They are tested with fire attack from one side, with all openings and penetrating items and joints firestopped.

On construction projects, there are three ways firestopping is installed currently. First, the 'he or she who pokes the hole fills it with firestopping' takes place, about 1/3 the time. A specialty firestop contractor installs for about another 1/3 of installations. The final 1/3 is a combination of specialty firestop contractors and the 'he or she who pokes the hole fills it' method. In other words, about 1/2 of the installations are installed by companies who most likely do not understand firestop systems selection nor the zero tolerance installation protocol. And, with the 20+ trades who potentially touch firestopping, many who perform the work as a 'sideline', the potential for a mistake increases exponentially when inexperienced companies install firestopping. However, firestopping is a complex operation, just like any other trade. Mastering more than one trade by attending a 30 minute to 16 hour class is nearly impossible for workers of any trade background.

In simple terms, inadequate firestopping makes the fire resistance rated floor or wall assembly become swiss cheese like, and not representative of testing. The risks of inadequate firestopping are apparent due to the many trades who install firestopping as a sideline...who just don't get the 'zero tolerance' systems oriented approach needed to get firestopping done right. Inspection to ASTM E 2174 and ASTM E 2393 brings a needed check to this important discipline, whether a FCIA Member specialty firestop contractor is installing or not.

Cost Impact: This will increase cost of construction when a contractor installing firestopping does not understand the zero tolerance protocol for firestopping. It will not increase the cost of construction when a contractor knowledgeable in the zero tolerance protocol for firestopping is used.

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM E2174-09 and ASTM E2393-09, 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 September 24, 2009.

ICCFILENAME: KOFFELL-S4-1704.15 NEW

Public Hearing Results

This code change was contained in the errata posted on the ICC website. Please go to <http://www.iccsafe.org/cs/codes/Pages/09-10ProposedChanges.aspx>.

Note: The following analysis was not in the Code Change monograph but was published on the ICC website at <http://www.iccsafe.org/cs/codes/Documents/2009-10cycle/ProposedChanges/Standards-Analysis.pdf> :

Analysis: Review of proposed new standards ASTM E 2174 and ASTM E 2393 indicated that, in the opinion of ICC Staff, the standards comply with ICC standards criteria.

Committee Action:

Approved as Modified

Modify the proposal as follows:

1704.15 Fire-resistant penetrations and joints. In buildings having occupied floors located more than 75 feet (22860 mm) above the lowest level of fire department vehicle access, special inspections for through penetrations, membrane penetration firestops, fire resistant joint systems, and perimeter fire barrier systems ~~of the types specified in~~ tested and listed in accordance with Sections 713.3.1.2, 713.4.1.2, 714.3 and 714.4 shall be in accordance with Sections 1704.15.1 or 1704.15.2.

1704.15.1 Penetration firestops. Inspections of penetration firestop systems ~~of the types specified in~~ tested and listed in accordance with Sections 713.3.1.2 and 713.4.1.2 shall be conducted by an approved inspection agency in accordance with ASTM E 2174.

1704.15.2 Fire-resistant joint systems. Inspection of fire resistant joint systems ~~of the types specified in~~ tested and listed in accordance with Sections 714.3 and 714.4 shall be conducted by an approved inspection agency in accordance with ASTM E 2393.

(Portions of the proposal not shown remain unchanged)

Committee Reason: The committee agreed that these installations were critical and that special inspections should be required for these installations in buildings having occupied floors located more than 75 feet above the lowest level of fire department vehicle access. The modification more appropriately identifies the systems as those that are tested and listed.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment 1:

Dave Frable, U.S. General Services Administration, requests Disapproval.

Commenter's Reason: We do not concur with the action of the Fire Safety Code Committee to require the need for special inspections to be conducted by an approved inspection agency in accordance with ASTM E 2174 for all penetration firestop systems and ASTM E 2393 for all fire-resistant joint systems in all high-rise buildings.

Typically, special inspections are only required for complex engineering systems or systems that are unusual in nature. It is our opinion that penetration firestop systems and fire resistant joint systems typically occur on every new construction project and are currently adequately addressed in Sections 110.3.6 and 110.3.8. Therefore, the need to require special inspections in all high-buildings is not warranted.

In addition, the proponent has failed to provide any technical data or life loss data to substantiate that if the penetration firestop systems and fire resistant joint systems fail within any high-building, a substantial hazard to human life would occur (e.g., a fully sprinklered six story office building, etc.).

It should also be pointed out that ASTM E 2174 and ASTM E 2393 do not provide any minimum qualification requirements for an "inspection agency" but does have qualification requirements for the inspector. For example, one acceptable qualification includes having a minimum of two years experience in construction field inspections and have education, credentials, and experience acceptable to the authorizing authority (i.e., architect, engineer, building owner). Therefore, we feel that these requirements should be in the project specification and not the Code.

Lastly, we also disagree with the proponent that the requirement for special inspections will not increase construction costs.

Public Comment 2:

Steve Orlowski, National Association of Home Builders, requests Disapproval.

Commenter's Reason: The authority having jurisdiction, under the auspice of section 1704.15 as found in the current 2009 IBC, already has the authority to require certain methods and materials to be certified by a third party. The proponent has provide no technical justification nor any historical data showing that AHJ or their inspectors are unqualified to conduct these inspection or have failed in their duties due to a loss of any magnitude. In the proponent's written testimony, there are numerous examples referencing the misapplication of products by unqualified installers, yet the proposed change takes away the ability for the AHJ to perform inspections on these products unless they are an approved inspection agency in accordance with the two ASTM standards, as referenced. NAHB requests the final assembly to reject this proposal given that the AHJ already has the option to request special inspections be conducted if they are unfamiliar with products and that this needlessly increases the number of special inspection required by the code.

Final Action: AS AM AMPC____ D

S141-09/10
1715.5.1

Proposed Change as Submitted

Proponent: William E. Koffel, Koffel Associates, Inc., representing Glazing Industry Code Committee

Revise as follows:

1715.5.1 Exterior windows and doors. Exterior windows and sliding doors shall be tested and labeled as conforming to AAMA/WDMA/CSA 101/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/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 1715.5.2. Products installed in buildings of Group R not more than three stories above grade plane that are 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 restrict the application of 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 3/4 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. Although the deflection exemption remains in the IRC for residential buildings and as proposed in the IBC for low-rise residential, 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.

ICCFILENAME: KOFFELL-S1-1714.5.1

Public Hearing Results

Committee Action:

Disapproved

Committee Reason: Disapproved for same reasoning as S140 – 09/10.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment:

Thomas S. Zaremba, Roetzel & Andress, representing Glazing Industry Code Committee (GICC), a committee of the Glass Association of North America (GANA) and Thomas D. Culp, Ph.D, Birch Point Consulting LLC, representing Aluminum Extruders Council, request Approval as Submitted.

Commenter's Reason: The adoption of this proposal will 1- strengthen the structural integrity of exterior window and door assemblies, 2- decrease the likelihood they will break, and, thus, 3- make them safer when subjected to design loads.

Chapter 24 of the IBC and ASTM E1300 both establish maximum deflection limits for glazing in order to ensure that the glazing is firmly supported and does not break when subjected to design loads. However, Section 1715.5.1 provides an exemption from deflection limits for products labeled to the AAMA/WDMA/CSA 101/I.S.2/A440 standard. This exemption applies to all product types and to all occupancies and is, therefore, far too broad. This proposal would correct the overbreadth of Section 1715.5.1 by removing it from most occupancies, thus, restoring an appropriate safety margin of less than an 8 in 1000 probability of glass breakage under design loads.

In disapproving the proposal, the committee questioned why the proposal retained the exemption from deflection limits for low-rise Group R occupancies (ie., those that are not more than three stories above grade) but eliminated it as to other occupancies. The reason for retaining the exemption for low-rise Group R occupancies is, simply, that an exemption for the lighter products used in low-rise residential construction may well be appropriate. However, just because it is appropriate for the very light construction used in those applications, does not make it appropriate to exempt all other occupancies from the deflection limits mandated by Chapter 24 and ASTM E1300. Moreover, retaining the exemption for low-rise Group R occupancies will maintain consistency with the IRC, where detached one- and two-family dwellings and townhouses with a separate means of egress are currently exempt from deflection limit requirements.

Final Action Agenda voters are urged to vote against the standing motion to disapprove this proposal in order to vote in favor of a motion to approve S141 As Submitted.

Final Action: AS AM AMPC_____ D

S143-09/10

1715.4.2, Chapter 35

Proposed Change as Submitted

Proponent: John Woestman, The Kellen Company, representing the Door Safety Council (DSC)

1. Add new text as follows:

1715.5.2 Exterior windows and door assemblies not provided for in Section 1715.5.1. Exterior window and door assemblies shall be tested in accordance with ASTM E 330. Structural performance of exterior side-hinged door assemblies shall be determined in accordance with either ASTM E330 or ANSI A250.13. 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.

2. Add new standard to Chapter 35 as follows:

ANSI

ANSI A250.13-08 Testing and Rating of Severe Windstorm Resistant Components for Swinging Door Assemblies

Reason: This proposal helps resolve performance and code compliance issues when exterior side-hinged door openings are comprised of components from multiple sources and include interchangeable elements (ie; doors, frames, hinging and latching hardware, etc.).

Through the ANSI standards development process, stake-holders, comprising most major manufacturing associations, testing and certification organizations, specifiers, code officials and end users, developed a national standard for a component-based approach to testing for windstorm resistance of swinging door openings. The test procedures used in this standard represent the most severe requirements found in the windstorm resistance standards referenced in today's building codes. These procedures are designed to isolate the loads, conditions and critical performance requirements that a particular component is subjected to in full assembly tests and duplicate these specific conditions. Using a combination of worst-case scenario design and safety factors, this standard is designed to provide a component rating that relates directly to the component's ability to withstand the conditions that occur in full assembly tests.

This proposed change allows an alternative method to demonstrate structural performance for side-hinged door openings by requiring components to be tested per ANSI A250.13-2008. A250.13 contains language that prescribes how components are to be selected to create complete swinging door openings expected to perform equivalently to those tested to ASTM E 330. ANSI A250.13 has additional requirements that are more stringent than those in the current 1714.5.2, including testing for a minimum of 30 seconds at a load equal to 1.5 times the design pressure. Currently 1714.5.2 requires testing for 10 seconds at a load equal to 1.5 times the design pressure.

Prior to releasing the current revision, validation tests were performed at three design-load levels, using the A250.13 test protocol to establish performance ratings. The study confirmed that at the same design-load level, openings comprised of such components will perform in the same manner as those in assembly based test protocols. The validation tests also showed that where an element was identified as the weakest in an opening during component testing, it would perform similarly when tested as part of an assembly at the same design-load.

Building designers will use the performance based criteria of ANSI A250.13 to select appropriate components to construct swinging door openings by conducting the presently required opening-by-opening design analysis, verify code compliance, and submit the results through the normal plans review process. Code authorities will therefore need only to verify the design load calculations and compliance analysis are correct and that ANSI A250.13 compliant products are utilized and installed in accordance with the manufacturer's instructions during construction.

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 A250 13-08 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 September 24, 2009.

ICCFILENAME: WOESTMAN-S2-1714.5.2

Public Hearing Results

Note: The following analysis was not in the Code Change monograph but was published on the ICC website at <http://www.iccsafe.org/cs/codes/Documents/2009-10cycle/ProposedChanges/Standards-Analysis.pdf> :

Analysis: Review of proposed new standard ANSI A250.13 indicated that, in the opinion of ICC Staff, the standard complies with ICC standards criteria.

Committee Action:

Disapproved

Committee Reason: There are concerns on the applicability of the proposed referenced standard to this portion of the IBC. There is also a question of who takes responsibility for the entire door assembly, when only the individual parts are tested by the standard.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment 1:

John Woestman, The Kellen Company, representing Builders Hardware Manufacturers Association (BHMA), Bud Bulley, representing the National Association of Architectural Metal Manufacturers (NAAMM) and representing the Hollow Metal Manufacturer's Association (HMMA); Jeff Wherry, representing the Steel Door Institute (SDI); Jerry Heppes, representing the Door and Hardware Institute (DHI), requests Approval as Submitted.

Commenter's Reason: This proposal helps resolve performance and code compliance issues when exterior side-hinged door openings are comprised of components from multiple sources and include interchangeable elements (ie; doors, frames, hinging and latching hardware, etc.).

Through the ANSI standards development process, stake-holders, comprising most major manufacturing associations, testing and certification organizations, specifiers, code officials and end users, developed a national standard for a component-based approach to testing for windstorm resistance of swinging door openings.

The test procedures used in this standard represent the most severe requirements found in the windstorm resistance standards referenced in today's building codes. These procedures are designed to isolate the loads, conditions and critical performance requirements that a particular component is subjected to in full assembly tests and duplicate these specific conditions. Using a combination of worst-case scenario design and engineering safety factors, this standard is designed to provide a component rating that relates directly to the component's ability to withstand the conditions that occur in full assembly tests. The developers of the standard recognized the complexity and variables of door components interacting as an assembly and incorporated stringent test criteria and engineering safety factors in the standard to address these variables.

This proposed change to the IBC allows an alternative method to demonstrate structural performance for side-hinged door openings by requiring components to be tested per ANSI A250.13-2008. A250.13 prescribes how components are to be selected to create complete swinging door assemblies expected to perform equivalently (or better) to those tested to ASTM E 330. ANSI A250.13 has additional requirements that are more stringent than those in the current 1715.5.2, including testing for a minimum of 30 seconds at a load equal to 1.5 times the design pressure. Currently 1715.5.2 requires testing for 10 seconds at a load equal to 1.5 times the design pressure.

Prior to releasing the current revision, validation tests were performed at three design-load levels, using the A250.13 test protocol to establish performance ratings. The study confirmed that at the same design-load level, door assemblies comprised of such components will perform the same (or better) as those in assembly-based test protocols (ASTM E330). The validation tests also showed that where a door component was identified as the weakest in a door assembly during component testing, it would perform similarly when tested as part of an entire door assembly at the same design-load.

Building designers will use the performance based criteria of ANSI A250.13 to select appropriate components to construct swinging door assemblies by conducting the presently required opening-by-opening design analysis, verify code compliance, and submit the results through the normal plans review process. Code authorities will therefore need only to verify the design load calculations and compliance analysis are correct and that ANSI A250.13 compliant products are utilized and installed in accordance with the manufacturer's instructions during construction.

Addressing the committee's question of "who takes responsibility for the entire door assembly, when only individual parts are tested by the standard", Section 1703.4 of the IBC addresses this question by requiring specific and sufficient information be provided to the building official. With the introduction to the IBC of A250.13 for door component selection for side-hinged doors, similar to door component selection for fire-rated doors, each component of the side-hinged door would be selected to meet or exceed the code-required design criteria of the opening.

The committee's other concern of the applicability of ANSI A250.13 to this section of the code, this standard was developed explicitly as an optional alternative to ASTM E330 testing (as required by the code). Door assemblies designed and assembled for hurricane-prone areas generally exceed the design pressure requirements required outside hurricane-prone areas. As a result, doors assembled of components tested and evaluated to A250.13 (i.e. meeting the required design pressure of a specific project in a hurricane-prone area) could be used in a building where the design pressure is considerably lower.

This proposal will not increase the cost of construction – it should decrease the testing costs of complying with the building code resulting in a slight decrease of the cost of door components.

Public Comment 2:

Larry J. Tanner, P.E., Texas Tech University, representing Wind Science & Engineering Research Center, Debris Impact Test Laboratory, requests Disapproval

Commenter's Reason: The ANSI A250 Standard, along with the ASTM 1886/1996 standards, were developed to prevent the proliferation of envelope perforations and the resulting inundation of rainwater from hurricane events. Evidence from hurricane investigations has revealed that indeed buildings designed to these standards performed better than buildings without said protection. However, it should be understood and specifically included in technical specifications by the manufacturers and advertisements to the consumers, that such products are intended only for non-catastrophic property protection from rainwater inundation and not for the protection of building occupants (Life Safety). I was a coauthor of both FEMA 320 and FEMA 361 which utilize Tornado and Hurricane Saferoom Design Wind Speed Maps. Never were the above referenced ANSI and ASTM standards considered suitable for FEMA 320 Saferooms or FEMA 361 Community Shelters. Specifics to the proposed changes to the ANSI A250.13-2008 Standard:

1. From a quantitative standpoint the "stiffness theory" appears reasonable; however laboratory tests have proven otherwise. Texas Tech University has been the leading "storm debris impact researcher" for over 35 years. Tests on door assemblies have proven that success or failure from wind pressures and debris impacts is unique to the door (or window unit) and the hardware components installed. A heavily constructed door absorbs little energy and directs most of the energy to the attaching components and has proven to fail components that previously passed on other less massive doors. Lighter constructed doors can bend excessively and either pull out locking bolts or cause bolting bending and ultimate failure. Doors passing the impact tests must have a unique set of hardware that matches the door performance, thus doors are rated as a complete assembly, inclusive of frame, door(s), hinging, and locking hardware. Window lites in doors compromise the strength of the door and present another set of unique circumstances which require the unit to be rated as a complete assembly. Window unit performance is unique to the opening size, frame type, and the glazing. The elasticity of the glazing is a function of size and type. Based on size, some glazing is so elastic that it bounces out of the frame. Smaller is not always better; some glazing will destroy the framing system and be pushed out. Thus, the only way to predict window behavior under impact is by full scale testing in the laboratory in "as specified and installed" condition.

2. Though these Standards were developed for "envelope" protection to reduce rainwater intrusion, these components that are now rated as "Hurricane Tested" are now being used in hardened "Hurricane Shelters" which are intended to protect lives. This is the result of misleading specification sheets, and uninformed dealers and consumers.

3. The "component rating" system does not consider the size of doors or glazed openings; the stiffness of doors with various sizes of lites, nor the quantity of hinges or latches required per size to carry the loads.

4. The Standard requires the component to be rated by ultimate load, but there is no guidance regarding the "assembly rating" based upon mixture of components with various ratings.

5. The Standard does not require engineering review or oversight.

6. According to the Test Procedure stated in Section 5.2.2, the impact energy should be 350 foot-pounds. However, for hinges, Section 6.1.1.2 and Latching Hardware, Section 7.1.1.3, the impact kinetic energy has been reduced to 125 foot-pounds. I understand the "stiffness" theory of the test fixture and product configuration, but this assumes that every laboratory will have the same fixtures and the same laboratory conditions.

7. The wind speed range has changed from 110-150 mph for the 2003 Standard to 110-170 mph for the 2008 Standard, but the test loads and impact criteria has not changed.

8. The impact location 6" above the floor on Figure 4, page 6 is unrealistic. In researching most all of the severe tornadoes and hurricanes since 1989, I have never seen an impact lower than 2.5 feet on a vertical surface.

9. Although there is not the opportunity at this time, to prevent the misuse of these products as describe above, I would suggest that the title of the standard be changed to: *Testing and Rating of Windstorm Resistant Components for Swinging Doors for Non-Life Safety Uses.*

Final Action: AS AM AMPC_____ D

**S144-09/10-PART I
1715.6 (New), 202**

Proposed Change as Submitted

Proponent: Julie Ruth, PE, JRuth Code Consulting, representing American Architectural Manufacturers Association

PART I – IBC STRUCTURAL

1. Revise as follows:

SECTION 202

SKYLIGHT, UNIT. A factory-assembled, glazed fenestration unit, containing one panel of glazing material that allows for natural lighting through an opening in the roof assembly while preserving the weather-resistant barrier of the roof. Unit skylights include, but are not limited to, tubular daylighting devices (TDDs).

2. Add new text as follows:

1715.6 Skylights and sloped glazing. Unit skylights shall comply with the requirements of Section 2405. All other skylights and sloped glazing shall comply with the requirements of Chapter 24.

(ReNUMBER subsequent sections)

Reason: This proposal clarifies that tubular daylighting devices (TDDs) are unit skylights and therefore subject to the testing and labeling requirements of Section 2405 for these devices. It also points the code user to the appropriate location in the IBC for the structural requirements for unit skylights, TDDs and all other types of sloped glazing.

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

ICCFILENAME: RUTH-G1-202

Public Hearing Results

PART I- IBC STRUCTURAL

Committee Action:

Disapproved

Committee Reason: There was concern with the proposed Section 1715.6 being located in the section on testing.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment 1:

Julie Ruth, JRuth Code Consulting, representing American Architectural Manufacturers Association, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

PART I – IBC STRUCTURAL

SECTION 202

SKYLIGHT, UNIT. A factory-assembled, glazed fenestration unit, containing one panel of glazing material that allows for natural lighting through an opening in the roof assembly while preserving the weather-resistant barrier of the roof. Unit skylights include, but are not limited to, tubular daylighting devices (TDDs).

~~**1715.6 Skylights and sloped glazing.** Unit skylights shall comply with the requirements of Section 2405. All other skylights and sloped glazing shall comply with the requirements of Chapter 24.~~

Commenter's Reason: The intent of S144 was to clarify within both the IBC and IRC that tubular daylighting devices are unit skylights, and therefore shall be tested and labeled in accordance with AAMA/WDMA/CSA 101/I.S.2/A440, as required for unit skylights in IBC Section 2405 and IRC Section R308.6.9.

S144, Part I also included a pointer from Section 1715 of the IBC, where AAMA/WDMA/CSA 101/I.S.2/A440 is referenced for windows and sliding doors, to Section 2405, which contains the reference to the same standard for unit skylights. The IBC Structural Committee did not agree with the addition of this pointer in Section 1715 of the IBC, and therefore they disapproved S144, Part I.

S144, Part II, which only added tubular daylight devices to the definition of unit skylights, and did not contain a pointer to another code section for reference to the applicable standard, was approved by the IRC Building and Energy Committee.

This Public Comment removes the proposed pointer, and simply seeks the addition of tubular daylighting devices to the definition of unit skylights in the IBC. Its approval would be consistent with the action taken on S144, Part II.

Public Comment 2:

Gary J. Ehrlich, PE, National Association of Home Builders, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

PART I – IBC STRUCTURAL

SECTION 202

SKYLIGHT, UNIT. A factory-assembled, glazed fenestration unit, containing one panel of glazing material that allows for natural lighting through an opening in the roof assembly while preserving the weather-resistant barrier of the roof. ~~Unit skylights include, but are not limited to, tubular daylighting devices (TDDs).~~

TUBULAR DAYLIGHTING DEVICE (TDD). A non-operable fenestration unit primarily designed to transmit daylight from a roof surface to an interior ceiling via a tubular conduit. The basic unit consists of an exterior glazed weathering surface, a light-transmitting tube with a reflective interior surface, and an interior-sealing device such as a translucent ceiling panel. The unit may be factory assembled, or field-assembled from a manufactured kit.

1715.6 Skylights and sloped glazing. Unit skylights and tubular daylighting devices (TDDs) shall comply with the requirements of Section 2405. All other skylights and sloped glazing shall comply with the requirements of Chapter 24.

Commenter's Reason: The purpose of this public comment is to amend the proposed requirements for tubular daylighting devices. A tubular daylighting device (TDD) is typically field-assembled from a manufactured kit, unlike a unit skylight which is typically shipped as a factory-assembled unit. If the current unit skylight definition is applied to TDDs, some code users will expect that TDDs be entirely assembled in the factory. Also, the dome of a TDD is not necessarily constructed out of a single panel of glazing material. As such, a separate definition from that of a unit skylight is needed. The proposed definition is adapted from the definition in AAMA/WDMA A440. A reference to TDDs is added to Section 1715.6.

Final Action: AS AM AMPC_____ D

S144-09/10-PART II IRC R308.6.1

Proposed Change as Submitted

Proponent: Julie Ruth, PE, JRuth Code Consulting, representing American Architectural Manufacturers Association

PART II – IRC BUILDING/ENERGY

Revise as follows:

R308.6.1 Definitions.

UNIT SKYLIGHT SKYLIGHT, UNIT. A factory assembled, glazed fenestration unit, containing one panel of glazing material, that allows for natural daylighting through an opening in the roof assembly while preserving the weather-resistant barrier of the roof. Unit skylights include, but are not limited to, tubular daylighting devices (TDDs).

Reason: This proposal clarifies that tubular daylighting devices (TDDs) are unit skylights and therefore subject to the testing and labeling requirements of the IRC for same.

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

ICCFILENAME: RUTH-G1-202

Public Hearing Results

**PART II- IRC B/E
Committee Action:**

Approved as Submitted

Committee Reason: This change clarifies that a tubular daylighting devices (TDDs) is a unit skylight. The TDD was added to the energy conservation part of the code.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment:

Gary J. Ehrlich, PE., National Association of Home Builders, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

PART II – IRC BUILDING/ENERGY

R308.6.1 Definitions.

SKYLIGHTS AND SLOPED GLAZING. Glass or other transparent or translucent glazing material installed at a slope of 15 degrees (0.26 rad) or more from vertical. Glazing materials in skylights, including unit skylights, tubular daylighting devices, solariums, sunrooms, roofs and sloped walls are included in this definition.

SKYLIGHT, UNIT. A factory assembled, glazed fenestration unit, containing one panel of glazing material, that allows for natural daylighting through an opening in the roof assembly while preserving the weather resistant barrier of the roof. ~~Unit skylights include, but are not limited to, tubular daylighting devices (TDDs).~~

TUBULAR DAYLIGHTING DEVICE (TDD). A non-operable fenestration unit primarily designed to transmit daylight from a roof surface to an interior ceiling via a tubular conduit. The basic unit consists of an exterior glazed weathering surface, a light-transmitting tube with a reflective interior surface, and an interior-sealing device such as a translucent ceiling panel. The unit may be factory assembled, or field-assembled from a manufactured kit.

R308.6.9 Testing and labeling. Unit skylights and tubular daylighting devices shall be tested by an *approved* independent laboratory, and bear a *label* identifying manufacturer, performance *grade* rating and *approved* inspection agency to indicate compliance with the requirements of AAMA/WDMA/CSA 101/I.S.2/A440.

Commenter's Reason: The purpose of this public comment is to amend the proposed requirements for tubular daylighting devices. A tubular daylighting device (TDD) is typically field-assembled from a manufactured kit, unlike a unit skylight which is typically shipped as a factory-assembled unit. If the current unit skylight definition is applied to TDDs, some code users will expect that TDDs be entirely assembled in the factory. Also, the dome of a TDD is not necessarily constructed out of a single panel of glazing material. As such, a separate definition from that of a unit skylight is needed. The proposed definition is adapted from the definition in AAMA/WDMA A440. A reference to TDDs is added to Sections R308.6.1 and R308.6.9.

Final Action: AS AM AMPC _____ D

S149-09/10

1803.5.12

Proposed Change as Submitted

Proponent: Ali M. Fattah, City of San Diego, representing SD Area Chapter ICC Code Committee

Revise as follows:

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:

1. The determination of lateral earth pressures on foundation walls and retaining walls supporting more than 12 feet (3.66 m) of backfill height, due to earthquake motions.
2. 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 1613.5.4.
3. 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.
4. 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.

Reason: The proposed code change deletes a current requirement. The current requirement is onerous on small structures and light framed structures as well as for retaining walls. The California Building Code has had an amendment that was added in the 1990's that addresses this issue and limits the requirement to retaining walls higher than 12 ft. The amendment only applies to hospitals projects, school projects and State owned buildings (See Section 1806A.1 General, <http://www.bsc.ca.gov/default.htm>).

Evidence from recent earthquakes and recent experimental research results, including work recently completed at the University of California, Berkeley, CA (Al Atik and Sitar, 2008) have demonstrated that the retaining walls structures would have to move in order to develop the failure wedge postulated in the so-called Mononobe and Okabe method. This method was developed by Okabe (1926) and Mononobe & Matsuo (1929) as an extension of Coulomb's static earth pressure theory to include the inertial forces due to the horizontal and vertical back-fill accelerations. The M-O method was developed for dry cohesionless backfill retained by a gravity wall and is mainly based on the following assumptions (Seed & Whitman 1970):

1. The wall yields sufficiently to produce minimum active pressure and the soil is assumed to satisfy the Mohr-Coulomb failure criterion;
2. When the minimum active pressure is attained, a soil wedge behind the wall is at the point of incipient failure, and the maximum shear strength is mobilized along the potential sliding surface; and
3. The soil wedge behaves as a rigid body, and accelerations are constant throughout the mass.

However, this condition can only occur when the wall has already failed due to other causes and the current body of field evidence does not provide any evidence of existence of this proposed mechanism of failure.

Retaining wall backfill is what imposes the inertial forces and is controlled backfill, usually not cohesionless and is compacted.

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

ICCFILENAME:Fattah-S2-1803.5.1.2

Public Hearing Results

Committee Action:

Approved as Submitted

Committee Reason: This code change relieves the geo-technical requirement for determination of lateral earth pressure on small structures as well as retaining walls that support backfill no more than 12 feet in height. It is the height of the backfill that imposes the inertial force. This is based on a California Building Code requirement that recognizes earthquake is not controlling loading on these structures.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment 1:

Ali M. Fattah, PE., City of San Diego, Development Services Department, representing San Diego Area Chapter of ICC, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1803.5.12 Seismic Design Category D, E or 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:

1. The determination of lateral earth pressures on foundation walls, and retaining walls supporting more than ~~42~~ 6 feet of backfill height, due to earthquake motions.
2. 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 1613.5.4.
3. 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.
4. 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

Commenter's Reason: This public comment requests that the voting membership support the IBC Structural Committee's assessment of the need for this code change and to support a modification to the height threshold that may allay the concerns of other interested parties that may not be in support of the Committee's action.

The proposed code change was concurrently submitted for adoption into the 2010 California Building Code and the Structural Design - Lateral Forces Committee Code Advisory Committee recommended that the California Building Standard Commission adopt a similar code change that was limited to 6 ft height and not 12 ft as was proposed in the IBC.

The public comment reduces the height exemption to 6 ft to satisfy the concerns of certain stakeholders that disagree with the IBC Structural Committee decision. The SEAOC Code Committee saw the merits of the code change as proposed to both bodies and took a neutral stance. The proponent was not able to attend the meeting but was informed that NCSEA offered support to the IBC proposal.

The Division of the State Architect and the Office of Statewide Health Planning and Development repealed the 12 ft height exemption that has been published in editions of the California Building Code prior to the 2010 edition and decided to defer to Section 11.8 of ASCE 7-05. The proponent acknowledges that the code change will result in a conflict with Section 11.8 of ASCE 7-05 however the building code governs over the referenced standard. Had the requirements been struck from the Building Code as was the case for most of the seismic requirements in Chapter 16 the proponent would have sought to change the ASCE 7-05.

The proponent was not able to attend the Structural Committee meeting to explain that the justification for the code change was not only based on the CBC. The justification included a research study on the issue performed at the University of California as well as post earthquake reconnaissance. The committee action report seems to imply that the basis of approval was because a similar provision existed in the CBC.

The proposed code change is necessary for uniform enforcement and to avoid non-enforcement of the requirement on many miscellaneous structures such as swimming pools, minor earth retaining structures, some of which many not require a building permit. As structured, the code requirement in both ASCE 7-05 and the IBC places an enormous burden on an applicant to investigate a site for a structure that will be located on native undisturbed ground, that does not include fill materials or expansive soils and where ASCE 7-05 and the IBC provide adequate lateral earth design parameters that negate the need for a report. We urge the voting memberships support.

Public Comment 2:

Steven Winkel, FAIA, PE; Susan Dowty, SE; John Gillengerten, SE, representing the Federal Emergency Management Agency/Building Seismic Safety Council Code Resource Support Committee (FEMA/BSSC CRSC), request Disapproval.

Commenter's Reason: The committee reason for support of this proposal, that it "...is based on a California Building Code (CBC) requirement that recognizes earthquake is not controlling on these structures" is not valid. This CBC requirement was repealed in 2009 in favor of the requirements in ASCE 7-05 Section 11.8.3. This ASCE 7 provision requires that determination of lateral pressures on basement and retaining walls due to earthquake motions for structures Seismic Design Categories D, E, and F be included in a geotechnical investigations. Also, it should be noted that ASCE 7 Section 15.6.1, Earth-Retaining Structures, references Section 11.8.3 for determining lateral earth pressures due to earthquake ground motions.

Final Action: AS AM AMPC_____ D

S164-09/10 **1908.1.2**

Proposed Change as Submitted

Proponent: Alan Robinson, SE, representing Structural Engineers Association of California

Revise as follows:

1908.1.2 ACI 318, Section 21.1.1. Modify ACI 318 Sections 21.1.1.3 and 21.1.1.7 to read as follows:

21.1.1.3 – Structures assigned to Seismic Design Category A shall satisfy requirements of Chapters 1 to 19 and 22; Chapter 21 does not apply. Structures assigned to Seismic Design Category B, C, D, E or F also shall satisfy 21.1.1.4 through 21.1.1.8, as applicable. Except for structural elements of plain concrete complying with Section 1908.1.8 of the International Building Code, structural elements of plain concrete are prohibited in structures assigned to Seismic Design Category C, D, E or F.

21.1.1.7 – Structural systems designated as part of the seismic-force-resisting system shall be restricted to those *permitted by ASCE 7*. Except for *Seismic Design Category A*, for which Chapter 21 does not apply, the following provisions shall be satisfied for each structural system designated as part of the seismic-force-resisting system, regardless of the *Seismic Design Category*:

- (a) Ordinary moment frames shall satisfy 21.2.
- (b) Ordinary reinforced concrete structural walls *and ordinary precast structural walls* need not satisfy any provisions in Chapter 21.
- (c) Intermediate moment frames shall satisfy 21.3.
- (d) Intermediate precast *structural walls* shall satisfy 21.4.
- (e) Special moment frames shall satisfy 21.5 through 21.8.
- (f) Special structural walls shall satisfy 21.9.
- (g) Special structural walls constructed using precast concrete shall satisfy 21.10.
- (h) *In Seismic Design Category D, E or F, concrete tilt-up wall panels that exceed the limitations of intermediate precast structural wall system shall satisfy 21.9 in addition to 21.4.2 and 21.4.3.*

All special moment frames and special structural walls shall also satisfy 21.1.3 through 21.1.7.

Reason: Concrete tilt-up wall panels is an alternative forming system of site-cast concrete wall panels which are tilted or lifted in place. They do not qualify for special precast structural wall system, which must meet the PRESSS test protocol or ACI ITG-5.2. Unlike earlier construction of box-like industrial buildings, current practice in commercial buildings constructed using tilt-up panel wall system commonly consists of large window and door openings in consecutive panels. Wall panels varying up to three stories high with openings in consecutive panels tend to resemble wall frame, which is not currently recognized under any of the defined seismic-force resisting systems other than consideration as one of the precast structural wall systems. While special boundary elements are probably not required by calculation if there are a number of panels in one shear line, spandrel panels often should be investigated for requirements of coupling beams.

Large tilt-up buildings with flexible diaphragm also may include isolated interior structural wall panels, either cast-in-place or precast, which are designed to resist high required shear strength demand. These isolated structural wall panels must be investigated for special boundary elements. Based on the current code language, intermediate precast structural wall are exempt from requirements of ACI 318-08 section 21.9 and thus design for boundary element, coupling beam and ductile detailing will be absent. This proposal does not affect the selection of seismic response R-factor given in ASCE 7 Table 12.2-1. This proposal gives requirement under which design and detailing need to conform to special structural wall system

provisions in ACI-318 section 21.9. This proposal further enhances minimum life safety building performance under earthquake forces in SDC D, E or F.

Cost Impact: The code change proposal will not increase the cost of construction for typical tilt-up buildings in higher SDC.

ICCFILENAME: ROBINSON-S1-1908.1.2

Public Hearing Results

Committee Action:

Disapproved

Committee Reason: The current requirements on intermediate precast structural wall systems are clear, making this proposal unnecessary.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment:

Alan Robinson, SE., Structural Engineers Association of California, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1908.1.2 ACI 318, Section 21.1.1. Modify ACI 318 Sections 21.1.1.3 and 21.1.1.7 to read as follows:

21.1.1.3 – Structures assigned to Seismic Design Category A shall satisfy requirements of Chapters 1 to 19 and 22; Chapter 21 does not apply. Structures assigned to Seismic Design Category B, C, D, E or F also shall satisfy 21.1.1.4 through 21.1.1.8, as applicable. Except for structural elements of plain concrete complying with Section 1908.1.8 of the International Building Code, structural elements of plain concrete are prohibited in structures assigned to Seismic Design Category C, D, E or F.

21.1.1.7 – Structural systems designated as part of the seismic-force-resisting system shall be restricted to those *permitted by ASCE 7*. Except for *Seismic Design Category A*, for which Chapter 21 does not apply, the following provisions shall be satisfied for each structural system designated as part of the seismic-force-resisting system, regardless of the *Seismic Design Category*:

- (a) Ordinary moment frames shall satisfy 21.2.
- (b) Ordinary reinforced concrete structural walls *and ordinary precast structural walls* need not satisfy any provisions in Chapter 21.
- (c) Intermediate moment frames shall satisfy 21.3.
- (d) Intermediate precast *structural* walls shall satisfy 21.4.
- (e) Special moment frames shall satisfy 21.5 through 21.8.
- (f) Special structural walls shall satisfy 21.9.
- (g) Special structural walls constructed using precast concrete shall satisfy 21.10.
- ~~(h) In Seismic Design Category D, E or F, concrete tilt-up wall panels that exceed the limitations of intermediate precast structural wall system shall satisfy 21.9 in addition to 21.4.2 and 21.4.3.~~

All special moment frames and special structural walls shall also satisfy 21.1.3 through 21.1.7. In Seismic Design Category D, E or F, concrete tilt-up wall panels classified as intermediate precast structural wall system shall satisfy 21.9 in addition to 21.4.2 and 21.4.3.

Commenter's Reason: After further research in ACI 318, it was noted that by virtue of ACI 318 Sec. 21.1.1.7(d), intermediate precast structural walls designed under Sec. 21.4, material requirements intended under provisions 21.1.4, 21.1.5, 21.1.6, and 21.1.7 would be excluded for structures assigned to SDC D, E or F. Since the deliberation at the code development hearing, we have had further discussions with ACI 318-H in their meeting in New Orleans. It was a consensus that clarification of ACI 318 chapter 21 is needed to ensure that structural walls designed under ASCE 7 using the intermediate wall panel category would conform to ductility requirements comparable to special structural wall; and conformance to the long standing practice of ACI 318 to impose special requirements for high seismic design regions. This public comment gives explicit requirement under which design and detailing need to conform to special structural wall system provision in ACI-318 section 21.9, which covers both cast-in-place as well as precast. This public comment further gives building officials the tools to enforce minimum life safety building performance under earthquake forces in SDC D, E or F.

Final Action: AS AM AMPC____ D

S166-09/10
1908.1.8

Proposed Change as Submitted

Proponent: Homer Maiel, PE, CBO, City of San Jose, representing ICC Tri-Chapter (Peninsula, East Bay, Monterey Bay)

Revise as follows:

1908.1.8 ACI 318, Section 22.10. Delete ACI 318, Section 22.10, and replace with the following:

22.10 – Plain concrete in structures assigned to Seismic Design Category C, D, E or F.

22.10.1 – Structures assigned to Seismic Design Category C, D, E or F shall not have elements of structural plain concrete, except as follows:

- (a) Structural plain concrete basement, foundation or other walls below the base are permitted in detached one- and two-family dwellings three stories or less in height constructed with stud-bearing walls. In dwellings assigned to Seismic Design Category D or E, the height of the wall shall not exceed 8 feet (2438 mm), the thickness shall not be less than 7 1/2 inches (190 mm), and the wall shall retain no more than 4 feet (1219 mm) of unbalanced fill. Walls shall have reinforcement in accordance with 22.6.6.5.
- (b) Isolated footings of plain concrete supporting pedestals or columns are permitted, provided the projection of the footing beyond the face of the supported member does not exceed the footing thickness.

Exception: In detached one- and two-family dwellings three stories or less in height, the projection of the footing beyond the face of the supported member is permitted to exceed the footing thickness.

- (c) Plain concrete footings supporting walls are permitted, provided the footings have at least two continuous longitudinal reinforcing bars. Bars shall not be smaller than No. 4 and shall have a total area of not less than 0.002 times the gross cross-sectional area of the footing. For footings that exceed 8 inches (203 mm) in thickness, a minimum of one bar shall be provided at the top and bottom of the footing. Continuity of reinforcement shall be provided at corners and intersections.

Exceptions:

- 1. In Group U occupancies ~~detached one- and two-family dwellings three stories or less in height and~~ constructed with stud-bearing walls, plain concrete footings without longitudinal reinforcement supporting walls are permitted.
- 2. In structures assigned to Seismic Design Categories D, E and F, ~~For~~ foundation systems consisting of a plain concrete footing and a plain concrete stemwall, a minimum of one No. 4 bar shall be provided at the top of the stemwall and at the bottom of the footing.
- 3. Where a slab on ground is cast monolithically with the footing, one No. 5 bar is permitted to be located at either the top of the slab or bottom of the footing.

Reason: If any occupancy warrants no reinforcing, it is a U occupancy. A three story dwelling in Seismic Design Category D, E or F should have at least 1 #4 bar at the top and bottom of the footing. Concrete cracks without reinforcing. A minimal amount of reinforcing will limit cracks during a seismic event.

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

ICCFILENAME: MAIEL-S5-1908.1.8

Public Hearing Results

Committee Action:

Disapproved

Committee Reason: There are concerns with revising the exemption to now apply to Group U. In addition these proposed changes would be inconsistent with the NEHRP Provisions.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment:

Homer Maiel, PE, CBO, City of San Jose, representing ICC Tri-Chapter (Peninsula, East Bay, Monterey Chapters), requests Approval as Modified by this Public Comment.

Replace the proposal as follows:

1908.1.8 ACI 318, Section 22.10. Delete ACI 318, Section 22.10, and replace with the following:

22.10 – Plain concrete in structures assigned to Seismic Design Category C, D, E or F.

22.10.1 – Structures assigned to Seismic Design Category C, D, E or F shall not have elements of structural plain concrete, except as follows:

- (a) Structural plain concrete basement, foundation or other walls below the base are permitted in detached one and two-family dwellings three stories or less in height constructed with stud-bearing walls. In dwellings assigned to Seismic Design Category D or E, the height of the wall shall not exceed 8 feet (2438 mm), the thickness shall not be less than 7 1/2 inches (190 mm), and the wall shall retain no more than 4 feet (1219 mm) of unbalanced fill. Walls shall have reinforcement in accordance with 22.6.6.5.
- (b) Isolated footings of plain concrete supporting pedestals or columns are permitted, provided the projection of the footing beyond the face of the supported member does not exceed the footing thickness.

Exception: In detached one- and two-family dwellings three stories or less in height, the projection of the footing beyond the face of the supported member is permitted to exceed the footing thickness.

- (c) Plain concrete footings supporting walls are permitted, provided the footings have at least two continuous longitudinal reinforcing bars. Bars shall not be smaller than No. 4 and shall have a total area of not less than 0.002 times the gross cross-sectional area of the footing. For footings that exceed 8 inches (203 mm) in thickness, a minimum of one bar shall be provided at the top and bottom of the footing. Continuity of reinforcement shall be provided at corners and intersections.

Exceptions:

1. In Seismic Design Categories A, B and C, detached one- and two-family dwellings three stories or less in height and constructed with stud-bearing walls, are permitted to have plain concrete footings without longitudinal reinforcement ~~supporting walls are permitted~~.
2. For foundation systems consisting of a plain concrete footing and a plain concrete stemwall, a minimum of one bar shall be provided at the top of the stemwall and at the bottom of the footing.
3. Where a slab on ground is cast monolithically with the footing, one No. 5 bar is permitted to be located at either the top of the slab or bottom of the footing.

Commenter's Reason: In seismic design categories D, E and F, the flexural demands placed upon footings of stud wall framed detached one- and two-family dwellings make the use of plain concrete footings devoid of any longitudinal reinforcing unacceptable. The footing is an integral part of the seismic force-resisting load path and deserves to be constructed in a manner consistent with the seismic-resisting braced walls or shear wall panels it is supporting. The current specific allowance for absence of any longitudinal reinforcing will also prevent any vertical reinforcing from being placed in the footing, since there is nothing to tie any vertical bar to; consequently the current provision is allowing totally unreinforced footings in dwellings up to three stories in height.

Since the mid-1990's wood light-frame prescriptive provisions (currently in IBC Section 2308.9.3.1) for alternative wall bracing using tie-downs have required that the foundation at these alternative panels utilize one No. 4 bar top and bottom. Also, more recent alternative wall bracing provisions (Section 2308.9.3.2) that use tie-downs similarly specify footings with one No. 4 bar top and bottom. In addition, since the 2003 IBC, provisions for tie-downs at braced walls of buildings having stone or masonry veneer have been specified (Sections 2308.11.2 and 2308.12.2), but without any mention of minimum foundation reinforcing. Each time a tie-down is installed, the footing should be capable of resisting the flexural demands induced by that connection, yet the current 1918.1(c) exception 1 ignores this need.

There are additional reasons that this provision should be revised. The 2006 IBC reduced the number of stories permitted when using conventional construction provisions to two stories in Seismic Design Category C (Section 2308.11.1) and to one story in Seismic Design Categories D and E (Section 2308.12.1), while section 1908.1.8 continues to allow plain concrete footings for stud bearing wall of one- and two-family dwellings up to three stories in height. This implies that plain concrete footings are permitted even in engineered one- and two-family dwelling construction. The IBC also explicitly deems the use of AF&PA Wood frame Construction Manual (WFCM) as permitted to substitute for the traditional 2308 bracing provisions, but in that document all walls providing lateral resistance are required to use various types of tie-downs.

To address the concern of the ICC Structural Committee in Baltimore regarding the original proposal's application of the exception 2 to U occupancies, that change has been removed in this amended public comment proposal. With regard to any inconsistency of this proposal with the NEHRP Provisions, it must be noted that the applicable NEHRP provision (Sec. 9.4.2.2 Exception 1) has not been updated since its publication in 2004 (FEMA 450-1/2003) while conventional construction limits on number of permitted story levels and the use of tie-downs have progressed in the IBC since that time, as noted above.

While we recognize there is a cost of installing this minimum reinforcing, we believe that most builders of dwellings in Seismic Design Categories D through F are already providing this level of reinforcing, and that the cost of repairing cracks caused to interior and exterior finishes not to mention the foundation itself would far exceed the cost of minimal reinforcement of footings during the original construction.

Final Action: AS AM AMPC _____ D

S167-09/10

1908.1.9

Proposed Change as Submitted

Proponent: Kevin Moore, PE, SE, SECB and Edwin Huston, PE, SE, SECB, representing National Council of Structural Engineers Associations

Revise as follows:

1908.1.9 ACI 318, Section D.3.3. Modify ACI 318, Sections D.3.3.1, D3.3.4 and D3.3.5, and add Section D.3.3.7 to read as follows:

D.3.3.1 – The provisions of Appendix D do not apply to the design of anchors in plastic hinge zones of concrete structures under earthquake forces or to anchors that meet the requirements of Section D.3.3.7.

D.3.3.4 – Anchors shall be designed to be governed by the steel strength of a ductile steel element as determined in accordance with D.5.1 and D.6.1, unless either D.3.3.5 or D.3.3.6 is satisfied.

Exceptions:

1. Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.4.
2. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.4.

D.3.3.5 – Instead of D.3.3.4, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a force level corresponding to anchor forces no greater than the design strength of anchors specified in D.3.3.3.

Exceptions:

1. Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.5.
2. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.5.

D.3.3.7 – For anchors installed in wood sill plates a maximum of 2 ½ inches (38 mm) in net thickness, the allowable lateral design values for shear in the cast-in-place anchor, parallel to the grain of the wood sill plate, are permitted to be determined in accordance with Section 2305 of the *International Building Code*, provided the anchor installation complies with all of the following:

1. Anchor nominal diameter is 5/8 inches (16 mm);
2. Anchors are embedded into concrete a minimum of 7 inches (178 mm);
3. Anchors are located a minimum of 2 ½ anchor diameters from the edge of the concrete parallel to the length of the wood sill plate; and
4. Anchors are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the wood sill plate

Reason: Current design provisions require calculation of the capacity of anchor bolt fastening wood sill plates to concrete foundations via methods promulgated in ACI 318, Appendix D. These methods result in significantly reduced capacities for this connection when compared to historical values and legacy code requirements. The state of knowledge regarding this connection is ambiguous and does not support such a large reduction for a common assembly.

Recent experimental testing and analysis indicates that actual capacities of the considered connection far exceed those historically used for design, supporting the use of wood dowel design values for the connection. The experimental data used to support this code change proposal indicates that concrete failure modes do not control the capacity of the connection, so the need to calculate the capacity of the bolt related to concrete strength for proper embedment and edge spacing is superfluous.

Reference: W. Andrew Fennell, et al. *Report on laboratory testing of anchor bolts connecting wood sill plates to concrete with minimum edge distances*. March 29, 2009.

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

ICCFILENAME: MOORE-HUSTON-S1-1908.1.9 D.3.3.1

Public Hearing Results

Committee Action:

Approved as Modified

Modify the proposal as follows:

1908.1.9 ACI 318, Section D.3.3. Modify ACI 318, Sections ~~D.3.3.4~~, D3.3.4 and D3.3.5, ~~and add Section D.3.3.7~~ to read as follows:

~~D.3.3.1 — The provisions of Appendix D do not apply to the design of anchors in plastic hinge zones of concrete structures under earthquake forces or to anchors that meet the requirements of Section D.3.3.7.~~

D.3.3.4 – Anchors shall be designed to be governed by the steel strength of a ductile steel element as determined in accordance with D.5.1 and D.6.1, unless either D.3.3.5 or D.3.3.6 is satisfied.

Exceptions:

- ~~1. Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.4.~~
1. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.4.
2. In light-frame wood structure bearing or non-bearing walls, for the design of anchors used to attach wood sill plates to foundations or foundation stem walls, it shall be permitted to take the allowable in-plane shear strength of the anchors in accordance with Section 2305.1.2 of the International Building Code.

D.3.3.5 – Instead of D.3.3.4, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a force level corresponding to anchor forces no greater than the design strength of anchors specified in D.3.3.3.

Exceptions:

1. Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.5.
2. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.5.

~~D.3.3.7 — For anchors installed in wood sill plates a maximum of 2 ½ inches (38 mm) in net thickness, the allowable lateral design values for shear in the cast-in-place anchor, parallel to the grain of the wood sill plate, are permitted to be determined in accordance with Section 2305 of the International Building Code, provided the anchor installation complies with all of the following:~~

2305.1.2 Sill plate anchor bolts. For sill plates of 2x or 3x nominal thickness, the allowable lateral design for shear parallel to the grain of sill plate anchor bolts is permitted to be determined using the lateral design value for a bolt attaching a wood sill plate to concrete, as specified in AF&PA NDS Table 11E, provided the anchor bolts comply with all of the following:

1. The maximum anchor nominal diameter is 5/8 inches (16 mm);
2. Anchors are embedded into concrete a minimum of 7 inches (178 mm);
3. Anchors are located a minimum of 2 ¼ anchor diameters 1-3/4 inches (45 mm) from the edge of the concrete parallel to the length of the wood sill plate; and
4. Anchors are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the wood sill plate.

Committee Reason: This proposal revises the determination of anchor bolt capacity under Appendix D of ACI 318, in recognition that both lab tests and field experience show that failure of the wood sill plate controls the capacity. In these instances there is no need for laborious concrete strength calculations. The modification removes an exception that is no longer needed with the updates in the next edition of the ASCE 7 Standard. It also reformats the proposal as new Exception 3 and places the sill plate anchor details in new Section 2305.1.2. This also combines and addresses issues raised by code changes S170- 09/10 and S209 – 09/10.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment 1:

Don Allen, Steel Framing Alliance, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1908.1.9 ACI 318, Section D.3.3. Modify ACI 318, Sections D3.3.4 and D3.3.5 to read as follows:

D.3.3.4 – Anchors shall be designed to be governed by the steel strength of a ductile steel element as determined in accordance with D.5.1 and D.6.1, unless either D.3.3.5 or D.3.3.6 is satisfied.

Exceptions:

1. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.4.
2. In light-frame wood structure bearing or non-bearing walls, for the design of anchors used to attach wood sill plates to foundations or foundation stem walls, it shall be permitted to take the allowable in-plane shear strength of the anchors in accordance with Section 2305.1.2 of the International Building Code.
3. In cold-formed steel light-frame construction, for the design of anchors used to attach cold-formed steel track to foundations or foundation stem walls, it shall be permitted to take the allowable in-plane shear strength of the anchors in accordance with Section 2210.8 of the International Building Code.

D.3.3.5 – Instead of D.3.3.4, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a force level corresponding to anchor forces no greater than the design strength of anchors specified in D.3.3.3.

Exceptions:

1. Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.5.
2. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.5.

Add new text as follows:

2210.8 Sill plate anchor bolts. For cold-formed steel light-frame tracks of 33 to 68 mil designation thickness, the allowable lateral design for shear parallel to the track with anchor bolts is permitted to be determined using the lateral design value for a bolt attaching a cold-formed steel light-frame track to concrete, as specified in AISI S100, Section E.3.3.1, provided the anchor bolts comply with all of the following:

1. The maximum anchor nominal diameter is 5/8 inches (16 mm);
2. Anchors are embedded into concrete a minimum of 7 inches (178 mm);
3. Anchors are located a minimum of 1-3/4 inches (45 mm) from the edge of the concrete parallel to the length of the track; and
4. Anchors are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the track.

(Portions of proposal not shown remain unchanged)

Commenter's Reason: ACI 318 Appendix D design provisions are required for calculation of the capacity of anchor bolt fastening cold-formed steel (CFS) bottom track sill plates to concrete foundations. These methods result in significantly reduced capacities for this connection when compared to historical values and legacy code requirements. The state of knowledge regarding this connection is ambiguous and does not support such a large reduction for a common assembly.

Recent experimental testing and analysis indicates that actual capacities of the track-to-concrete anchor bolt connection far exceed those historically used for design, supporting the use of AISI bolt-bearing design values for the connection. The experimental data used to support this code change proposal indicates that ductile steel failure rather than concrete failure modes control the capacity of the connection, so the need to calculate the capacity of the bolt related to concrete strength for proper embedment and edge spacing is superfluous.

Please note that testing results will be posted at www.steel.org by April 15, 2010.

Public Comment 2:

Bonnie Manley, American Iron and Steel Institute, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1908.1.9 ACI 318, Section D.3.3. Modify ACI 318, Sections D3.3.4 and D3.3.5 to read as follows:

D.3.3.4 – Anchors shall be designed to be governed by the steel strength of a ductile steel element as determined in accordance with D.5.1 and D.6.1, unless either D.3.3.5 or D.3.3.6 is satisfied.

Exceptions:

1. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.4.
2. In light-frame wood structure bearing or non-bearing walls, for the design of anchors used to attach wood sill plates to foundations or foundation stem walls, it shall be permitted to take the allowable in-plane shear strength of the anchors in accordance with Section 2305.1.2 of the International Building Code.
3. Section D.3.3.4 need not apply and the design shear strength in accordance with Section D.6.2.1(c) need not be computed for anchor bolts attaching cold-formed steel track of bearing or non-bearing walls of light-frame construction to foundations or foundation stem walls provided all of the following are satisfied:
 1. The maximum anchor nominal diameter is 5/8 inches (16 mm).
 2. Anchors are embedded into concrete a minimum of 7 inches (178 mm).
 3. Anchors are located a minimum of 1-3/4 inches (45 mm) from the edge of the concrete parallel to the length of the track.
 4. Anchors are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the track.
 5. The track is 33 to 68 mil designation thickness.

Allowable in-plane shear strength of exempt anchors, parallel to the edge of concrete shall be permitted to be determined in accordance with AISI S100 Section E3.3.1.

D.3.3.5 – Instead of D.3.3.4, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a force level corresponding to anchor forces no greater than the design strength of anchors specified in D.3.3.3.

Exceptions:

1. Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.5.
2. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.5.

(Portions of proposal not shown remain unchanged)

Commenter's Reason: ACI 318 Appendix D design provisions are required for calculation of the capacity of anchor bolt fastening cold-formed steel (CFS) bottom track sill plates to concrete foundations. These methods result in significantly reduced capacities for this connection when compared to historical values and legacy code requirements. The state of knowledge regarding this connection is ambiguous and does not support such a large reduction for a common assembly.

Recent experimental testing and analysis indicates that actual capacities of the track-to-concrete anchor bolt connection far exceed those historically used for design, supporting the use of AISI bolt-bearing design values for the connection. The experimental data used to support this code change proposal indicates that ductile steel failure rather than concrete failure modes control the capacity of the connection, so the need to calculate the capacity of the bolt related to concrete strength for proper embedment and edge spacing is superfluous.

Please note that testing results will be posted at www.steel.org by April 15, 2010.

Public Comment 3:

James E. Russell, Building Codes Consultant and Brad Douglas AF&PA and American Wood Council, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

1908.1.9 ACI 318, Section D.3.3. Modify ACI 318, Sections D3.3.4 and D3.3.5 to read as follows:

D.3.3.4 – Anchors shall be designed to be governed by the steel strength of a ductile steel element as determined in accordance with D.5.1 and D.6.1, unless either D.3.3.5 or D.3.3.6 is satisfied.

Exceptions:

1. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.4.
2. ~~In light-frame wood structure bearing or non-bearing walls, for the design of anchors used to attach wood sill plates to foundations or foundation stem walls, it shall be permitted to take the allowable in-plane shear strength of the anchors in accordance with Section 2305.1.2 of the International Building Code.~~
2. D.3.3.4 need not apply and the design shear strength in accordance with D.6.2.1(c) need not be computed for anchor bolts attaching wood sill plates of bearing or non-bearing walls of light-frame wood structures to foundations or foundation stem walls provided all of the following are satisfied:
 1. The allowable in-plane shear strength of the anchor is determined in accordance with AF&PA NDS Table 11E for lateral design values parallel to grain.
 2. The maximum anchor nominal diameter is 5/8 inches (16 mm).
 3. Anchor bolts are embedded into concrete a minimum of 7 inches (178 mm).
 4. Anchor bolts are located a minimum of 1-3/4 inches (45 mm) from the edge of the concrete parallel to the length of the wood sill plate.

5. Anchor bolts are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the wood sill plate.
6. The sill plate is 2-inch or 3-inch nominal thickness.

D.3.3.5 – Instead of D.3.3.4, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a force level corresponding to anchor forces no greater than the design strength of anchors specified in D.3.3.3.

Exceptions:

1. Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.5.
2. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.5.

~~**2305.1.2 Sill plate anchor bolts.** For sill plates of 2x or 3x nominal thickness, the allowable lateral design for shear parallel to the grain of sill plate anchor bolts is permitted to be determined using the lateral design value for a bolt attaching a wood sill plate to concrete, as specified in AF&PA NDS Table 11E, provide the anchor bolts comply with all of the following:~~

- ~~1. The maximum anchor nominal diameter is 5/8 inches (16 mm);~~
- ~~2. Anchors are embedded into concrete a minimum of 7 inches (178 mm);~~
- ~~3. Anchors are located a minimum of 1-3/4 inches (45 mm) from the edge of the concrete parallel to the length of the wood sill plate; and~~
- ~~4. Anchors are located a minimum of 15 anchor diameters from the edge of the concrete perpendicular to the length of the wood sill plate.~~

Commenter's Reason: The addition of 2305.1.2 in the "As Modified" approval of S167 as currently worded limits the use of NDS provisions for sill anchor bolts to applications with 2x or 3x sill plates, parallel to grain applications, and anchor diameters of 5/8" or less for all lateral and shear loads including those from wind and low seismic areas. Anchor bolt limitations were originally intended to define a specific range of sill plate anchor conditions for which relief from specific concrete anchor strength provisions for seismic design was warranted, based on results of cyclic testing. Concrete anchor strength provisions for which relief is provided are (i) ductility requirements of D.3.3.4, and (ii) required concrete breakout strength in shear parallel to the edge in accordance with D.6.2.1(c). To restore the original intent of the proposal, provisions of the proposed new section 2305.1.2, that specify exactly what size sill plates, anchor bolts and bolt locations in concrete were tested, are relocated by this public comment to the section from which they are referenced (e.g. 1908.1.9 Exception 2).

This revision is necessary to accomplish two purposes: First it avoids the unintended interpretation that new section 2305.1.2 limits the use of the NDS as it applies to anchor bolt installations, and second it clarifies that in lieu of computing the concrete breakout strength in shear parallel to the edge for the anchor, it is the intent to allow the allowable load parallel to the edge of the foundation to be based on AF&PA NDS Table 11E for a limited range of sill plate anchor conditions.

Final Action: AS AM AMPC_____ D

S168-09/10
1908.1.9

Proposed Change as Submitted

Proponent: Alan Robinson, SE, representing Structural Engineers Association of California

Revise as follows:

1908.1.9 ACI 318, Section D.3.3. Modify ACI 318, Sections D.3.3.4 and D.3.3.5 delete and replace D.3.3.6 and add D.3.3.7 to read as follows:

D.3.3.4 - Anchors shall be designed to be governed by the steel strength of a ductile steel element as determined in accordance with D.5.1 and D.6.1, unless either D.3.3.5 or D.3.3.6 is satisfied.

Exceptions:

1. Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.4.
2. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.4.
3. In light-frame wood construction, design of anchors in concrete shall be permitted to satisfy D.3.3.7.

D.3.3.5 - Instead of D.3.3.4, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a force level corresponding to anchor forces no greater than the design strength of anchors specified in D.3.3.3.

Exceptions:

1. Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.5.
2. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.5.

D.3.3.6 - As an alternative to D.3.3.4 and D.3.3.5, it shall be permitted to take the design strength of the anchors as 0.4 times the design strength determined in accordance with D.3.3.3.

D.3.3.7 – In light-frame wood structures, bearing or non-bearing walls, concrete anchors of sill plate to foundation or foundation stem wall need not satisfy D.3.3.5 and D.3.3.6 when the design strength of the anchors is determined in accordance with D.3.3.3.

Reason: Development of Appendix D was based primarily on tests of concrete anchor using steel plates with substantially larger edge distance than common practice in light-frame construction. There are insufficient tests of concrete anchors with wood sill plate at minimum side cover distance to justify the arbitrary assignment of 50 per cent reduction of the design strength stated in D.3.3.6. Additional limitation for anchorage of wood stud wall is removed from current ACI 318-08 D.3.3.6 (i.e. D.3.3.6 - As an alternative to D.3.3.4 and D.3.3.5, it shall be permitted to take the design strength of the anchors as 0.4 times the design strength determined in accordance with D.3.3.3. A new section D.3.3.7 is introduced under this proposal to further modify ACI code for concrete anchors used in light-framed wood construction.

In common construction practice of light-frame construction, bolts are centered on sill plates giving a side edge distance of $1\frac{3}{4}$ inches. Current code requirements under ACI 318 Appendix D lead to substantial reduction of design capacity based on breakout strength of a single anchor under Section D.6.2.1(c) or D.6.2.2. As an example, the design strength for 5/8 inch diameter anchor bolt strength under D.3.3.3 is 1116 lbs. Requirement under D.3.3.6 will further reduce the design strength to 558 lbs. for use in sill bolts. The ASD value would be 398 lbs. This is a substantial reduction from prior codes leading to impractical bolt spacing for most wood shear panel nail spacing range. A comparison between ACI Appendix D to IBC Table 1911.2, Allowable Service Loads on Embedded Bolts, shows the disparity of concrete anchor value.

The primary mode of failure of plywood sheathed panels attached to wood sill plate has been through nail slippage and yielding. This mode of failure together with bending of bolt offers the ductility and toughness of wood wall panels. Over-strength factor does not apply in the transfer of seismic forces from wood shear panel to concrete. Additional reduction factor is not warranted based on recent laboratory test conducted under SEAOC Seismology Committee purview. Result of the test is summarized below.

Attachment:

Excerpt from Report on laboratory testing of anchor bolts connecting wood sill plates to concrete with minimum edge distances

IBC-06 references ACI 318-05 Appendix D for the determination of anchor bolt capacity (in single-shear) when attaching wood sill plates to concrete foundations. Engineers have historically anticipated the controlling failure of this connection to occur between the anchor bolts and the wood sill plate. Under the IBC, the wind resistance values of anchor bolts are about the same as in historical practice. However, design capacities seismic forces based on break-out strength in shear determined in accordance with ACI 318-05 Appendix D are greatly reduced and less than the wood to concrete connection design capacity for small side edge distances. Many practicing engineers and building officials are mystified by the substantial reduction of anchor bolt capacities obtained from the application of Appendix D equations for wood framed construction in seismic design categories D, E and F. In the absence of available test data, members of SEAOC Seismology Committee undertook a study of typical anchor bolted connections to establish a basis for evaluating design capacities while better understanding the behavior of this traditional connection.

Test parameters and procedures were established. The testing consisted of typical anchor bolt connections found in wood framed shear walls using pressure treated wood 2x4, 3x4, 2x6 and 3x6 sill plates and 5/8 inch diameter by seven inches embedment anchor bolts with code prescribed washers. Side edge distance of 1-3/4 inches for 2x4 and 3x4 and 2-3/4 inches for 3x4 and 3x6 sill plates. This Testing Program was completed in December 2008 and the results can be downloaded on the SEAOC website: <http://www.SEAOC.org/bluebook>.

The load protocol adopted for the tests was a displacement-controlled load protocol. Peak loads from monotonic tests were used to establish the reference force, which was used to prescribe the load steps in the pseudo-cyclic testing. Monotonic tests were run at a sufficiently slow rate to pick up the internal flaws forming within the concrete by using impact-echo testing. The Pseudo-cyclic tests were based on the CUREE load protocol but with cycles added at low load levels. All tests were conducted without intentionally pre-cracked concrete.

Impact-echo method was used to detect concrete side break-out, if any, during the tests. When concrete deterioration was detected, the corresponding load and displacement were recorded for each specimen. It was observed that the first stage of deterioration is a series of cracks that form within the concrete propagated from the centerline of the anchor bolt and angling out towards the outer free face of the concrete. The cracks ultimately reach the outer face and became shallow spall shapes. It is important to note that the early stages of concrete deterioration are not always visually apparent. A strong correlation between the "peak" envelope values with the onset of concrete side break-out was, however, observed. Peak values were in the range of 7,200 lb to 8,500 lb.

All cyclic test data was analyzed in accordance with ASTM E 2126 Standard Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Walls for Buildings. Result of cycle test of specimen 296/ 302 is shown in Figure A-1. The positive and negative envelop curves for each specimens were combined to produce an average envelope curve used to establish peak load, displacement at peak load, ultimate load, and displacement at ultimate load and summarized in Figures A-2 and A-3.

Findings of the anchor bolt test program were as follows:

1. The results of the Anchor Bolt Testing Program has shown that wood components attached to concrete with minimum edge distances exhibited ductile behavior. The wood "yield" is the first material limit state.
2. The tests indicated that concrete cracks were not produced at service level loads. In the non-linear range of performance, delamination generally produced a decline in capacity corresponding to a wood displacement of about 0.60 inches, with the bolt experiencing considerable deformation.
3. Further excursion of the wood plate in some cases produced a complete concrete spall, however the bolt head remained intact and considerable residual strength was provided as the bolt remained in tension.
4. Cracking through the section did not occur at any point. For these reason, cracked section reduction appears overly conservative. It should also be noted that according to the available literature reductions are generally not required for shear anchorage applied perpendicular to a crack.
5. Test support design bolt values based on ACI 318 section D.3.3.3 using $0.75\Phi V_n$.

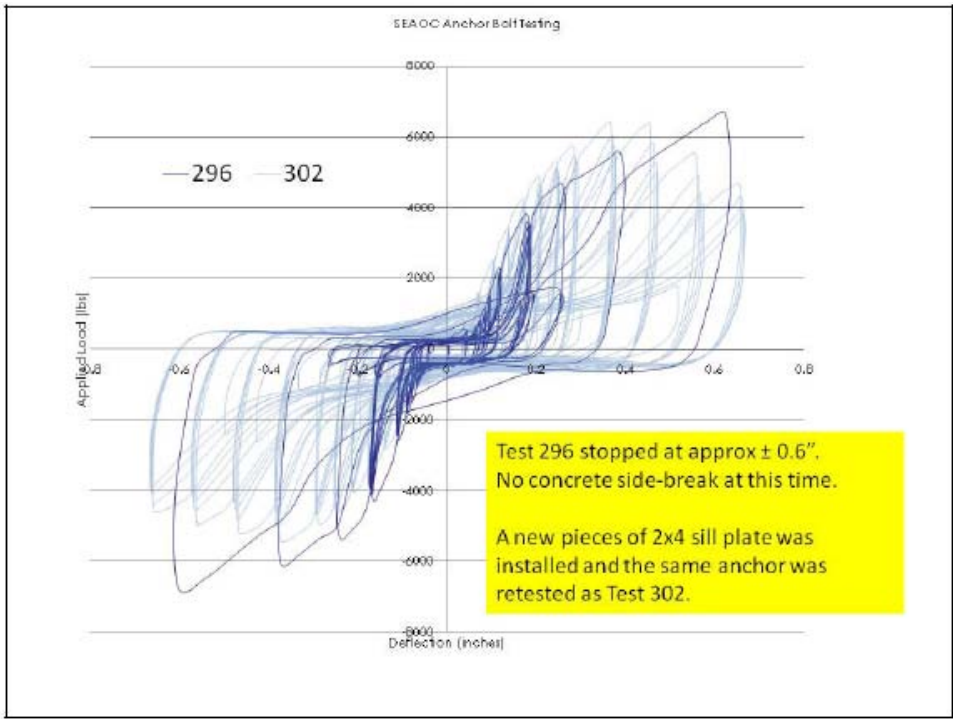


Figure A1 - Result of cycle test specimen 296 and 302

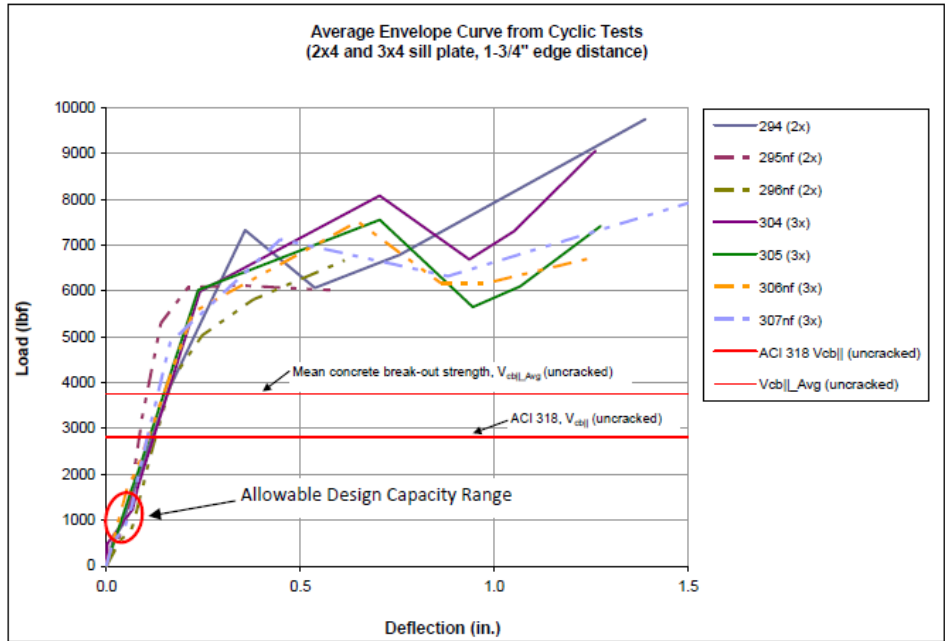


Fig. A2 – Average envelope curve of cyclic tests for concrete anchor of 2x4 and 3x4 sill plate

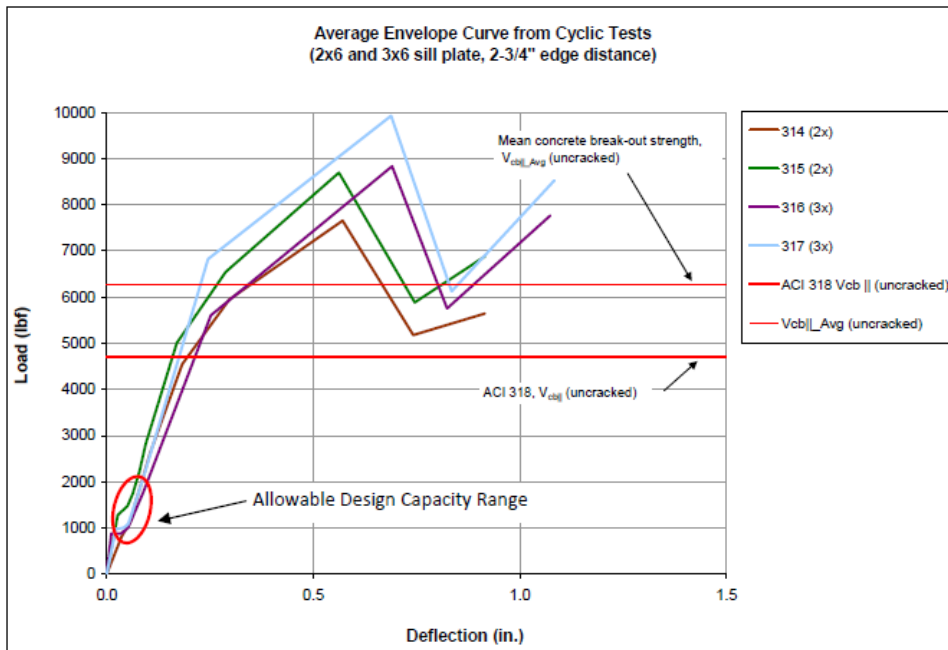


Fig. A3 – Average envelope curve of cyclic tests for concrete anchor of 2x6 and 3x6 sill plate

Ref. Fennel, Mochizuki, Moore (2008) "Report on laboratory testing of anchor bolts connection wood sill plates to concrete with minimum edge distances" SEAONC, San Francisco, CA

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

ICCFILENAME: ROBINSON-S3-1908.1.9

Public Hearing Results

Committee Action:

Disapproved

Committee Reason: With the liberalization of concrete anchorage approved in S167-09/10 a significant portion of problems posed in light-frame construction has been addressed. There is concern about the proposed extrapolation of data from testing that is ongoing. When dealing with an edge distance of only a little over an inch and considering typical construction tolerances, some anchor bolts could be installed awfully close to the edge of the concrete. Approval could possibly conflict with some portions of S167-09/10. The proponent is encouraged to provide better justification in the public comment phase.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment:

Alan Robinson, SE, representing Structural Engineers Association of California, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

1908.1.9 ACI 318, Section D.3.3. Modify ACI 318, Sections D.3.3.4 and D.3.3.5 delete and replace D.3.3.6 and add D.3.3.7 to read as follows:

D.3.3.4 - Anchors shall be designed to be governed by the steel strength of a ductile steel element as determined in accordance with D.5.1 and D.6.1, unless either D.3.3.5 or D.3.3.6 is satisfied.

Exceptions:

1. Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.4.

2. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.4.
3. In light-frame ~~wood~~ construction, design of anchors in concrete shall be permitted to satisfy D.3.3.7.

D.3.3.5 - Instead of D.3.3.4, the attachment that the anchor is connecting to the structure shall be designed so that the attachment will undergo ductile yielding at a force level corresponding to anchor forces no greater than the design strength of anchors specified in D.3.3.3.

Exceptions:

1. Anchors in concrete designed to support nonstructural components in accordance with ASCE 7 Section 13.4.2 need not satisfy Section D.3.3.5.
2. Anchors designed to resist wall out-of-plane forces with design strengths equal to or greater than the force determined in accordance with ASCE 7 Equation 12.11-1 or 12.14-10 need not satisfy Section D.3.3.5.

D.3.3.6 - As an alternative to D.3.3.4 and D.3.3.5, it shall be permitted to take the design strength of the anchors as 0.4 times the design strength determined in accordance with D.3.3.3.

D.3.3.7 – In light-frame ~~wood structures construction~~, bearing or non-bearing walls, shear strength of concrete anchors less than or equal to 1 inch [25 mm] in diameter of sill plate or track to foundation or foundation stem wall need not satisfy ~~D.3.3.5 and~~ D.3.3.6 when the design strength of the anchors is determined in accordance with ~~D.3.3.3~~ D.6.2.1(c).

Commenter's Reason: As presented in a companion change proposal S167, which was approved in the Code Development Hearing, based on results of Anchor Bolt Testing Program. The results of the bolt test, summarized under the original submittal of S168, showed the wood components attached to concrete exhibited ductile behavior. The wood "yield" being the first material limit state demonstrated that the reduction factor in ACI 318-08 Section D.3.3.6 need not be satisfied for bolts anchored to footings provided minimum concrete cover under ACI 318 sec. 7.7 is met. ACI 318 is currently considering changes to section 3.3 with various arbitrary reduction factors removed. ACI 318-08 sec. D.6.2.1(c) is specific for the design of concrete anchors parallel to a free edge.

A series of tests are planned by AISI for spring 2010, following closely with the test protocols used in the SEAOC wood sill concrete anchor tests. It is expected that results of the tests will be available prior to the Final Action Hearing. It is anticipated that similar ductile behavior of light gauge metal track as for wood sill plate with yielding of the steel track occurred prior to spalling of concrete. This public comment is amended to apply to light frame construction.

Final Action: AS AM AMPC____ D

S179-09/10

2109.1.1, 2308.2, 2308.2.1

Proposed Change as Submitted

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

Revise as follows:

2109.1.1 Limitations. The use of empirical design of masonry shall be limited as noted in Section 5.1.2 of TMS 402/ACI 530/ASCE 5. Section 5.1.2.2 of TMS 402/ACI 530/ASCE 5 shall be modified as follows:

5.1.2.2 Wind – Empirical requirements shall not apply to the design or construction of masonry for buildings, parts of buildings, or other structures to be located in areas where the basic wind speed exceeds 130 mph (58 m/s) as given in ASCE 7.

The use of dry-stacked, surface-bonded masonry shall be prohibited in *Occupancy Category IV* structures. In buildings that exceed one or more of the limitations of Section 5.1.2 of TMS 402/ACI 530/ASCE 5, masonry shall be designed in accordance with the engineered design provisions of Section 2101.2.1, 2101.2.2 or 2101.2.3 or the foundation wall provisions of Section 1807.1.5.

2308.2 Limitations. Buildings are permitted to be constructed in accordance with the provisions of *conventional light-frame construction*, subject to the following limitations, and to further limitations of Sections 2308.11 and 2308.12.

1. Buildings shall be limited to a maximum of three *stories above grade plane*. For the purposes of this section, for buildings in *Seismic Design Category D* or *E* as determined in Section 1613, cripple stud walls shall be considered to be a *story*.

Exception: Solid blocked cripplewalls not exceeding 14 inches (356 mm) in height need not be considered a *story*.

2. Maximum floor-to-floor height shall not exceed 11 feet, 7 inches (3531 mm). Bearing wall height shall not exceed a stud height of 10 feet (3048 mm).
3. Loads as determined in Chapter 16 shall not exceed the following:
 - 3.1. Average dead loads shall not exceed 15 psf (718 N/m²) for combined roof and ceiling, exterior walls, floors and partitions.

Exceptions:

1. Subject to the limitations of Sections 2308.11.2 and 2308.12.2, stone or masonry veneer up to the lesser of 5 inches (127 mm) thick or 50 psf (2395 N/m²) and installed in accordance with Chapter 14 is permitted to a height of 30 feet (9144 mm) above a noncombustible foundation, with an additional 8 feet (2438 mm) permitted for gable ends.
2. Concrete or masonry fireplaces, heaters and chimneys shall be permitted in accordance with the provisions of this code.
- 3.2. Live loads shall not exceed 40 psf (1916 N/m²) for floors.
- 3.3. Ground snow loads shall not exceed 50 psf (2395 N/m²).
4. Wind speeds shall not exceed 130 ~~400~~ miles per hour (mph) (44 ~~58~~ m/s) (3-second gust).

Exception: Wind speeds shall not exceed 140 ~~140~~ mph (48.4 ~~63~~ m/s) (3-second gust) for buildings in Exposure Category B that are not located in a *hurricane-prone region*.

5. Roof trusses and rafters shall not span more than 40 feet (12 192 mm) between points of vertical support.
6. The use of the provisions for *conventional light-frame construction* in this section shall not be permitted for *Occupancy Category IV* buildings assigned to *Seismic Design Category B, C, D, E or F*, as determined in Section 1613.
7. *Conventional light-frame construction* is limited in irregular structures in *Seismic Design Category D or E*, as specified in Section 2308.12.6.

2308.2.1 Basic wind speed greater than 400 130 mph (3-second gust). Where the basic wind speed exceeds 130 ~~400~~ mph (58 m/s) (3-second gust), the provisions of either AF&PAWFCM, or the ICC 600 are permitted to be used. Wind speeds in Figure 1609A, 1609B, and 1609C shall be converted in accordance with Section 1609.3.1 for use with AF&PA WFCM or ICC 600.

Reason: The purpose of this code change is to correlate the prescriptive limits for empirical design of masonry and conventional wood frame construction with other proposals that are updating the wind speed maps in the IBC and the IRC. The wind speed maps in ASCE 7 are being updated to ultimate wind speeds as opposed to the ASD level wind speed maps that currently exist in ASCE 7 and in the IBC and IRC. See IBC code change for information on why the wind speed maps are being updated. While a way to convert the ultimate wind speeds to ASD level wind speeds is proposed in the IBC, the converted wind speeds do not match, from a geographic standpoint, the limitations the code previously imposed. Since the empirical provisions and conventional methods for wood frame construction, typically can't be calculated to equate to the lower level wind speeds at the current limit, including the fact that these provisions are missing some of the key wind resistant construction design methods (e.g. gable end wall bracing, bond beam reinforcement, vertical wall reinforcement, etc.), the proposed limitations will roughly, maintain the current limitations on empirical and conventional methods that currently exist in terms of geographic location on the wind speed map. While some areas of the country will see a reduction in areas where empirical design of masonry or conventional construction would be allowed, other areas will see an increase in areas where these methods would be allowed.

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

ICCFILENAME: Stafford-S1-2109

Public Hearing Results

Committee Action:

Disapproved

Committee Reason: The proposed correlation of wind speed triggers with the updated provisions approved in code change S84-09/10 need to be consistent with the wind terminology.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment:

T. Eric Stafford, representing Institute for Business and Home Safety, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

2308.2 Limitations. Buildings are permitted to be constructed in accordance with the provisions of *conventional light-frame construction*, subject to the following limitations, and to further limitations of Sections 2308.11 and 2308.12.

1. Buildings shall be limited to a maximum of three *stories above grade plane*. For the purposes of this section, for buildings in *Seismic Design Category D or E* as determined in Section 1613, cripple stud walls shall be considered to be a *story*.

Exception: Solid blocked cripplewalls not exceeding 14 inches (356 mm) in height need not be considered a *story*.

2. Maximum floor-to-floor height shall not exceed 11 feet, 7 inches (3531 mm). Bearing wall height shall not exceed a stud height of 10 feet (3048 mm).
3. Loads as determined in Chapter 16 shall not exceed the following:
 - 3.1. Average dead loads shall not exceed 15 psf (718 N/m²) for combined roof and ceiling, exterior walls, floors and partitions.

Exceptions:

1. Subject to the limitations of Sections 2308.11.2 and 2308.12.2, stone or masonry veneer up to the lesser of 5 inches (127 mm) thick or 50 psf (2395 N/m²) and installed in accordance with Chapter 14 is permitted to a height of 30 feet (9144 mm) above a noncombustible foundation, with an additional 8 feet (2438 mm) permitted for gable ends.
2. Concrete or masonry fireplaces, heaters and chimneys shall be permitted in accordance with the provisions of this code.

- 3.2. Live loads shall not exceed 40 psf (1916 N/m²) for floors.
- 3.3. Ground snow loads shall not exceed 50 psf (2395 N/m²).

4. Ultimate Design Wind Speeds, V_{ult} shall not exceed 130 miles per hour (mph) (44 m/s) (3-second gust).

Exception: Wind speeds shall not exceed 140 mph (48.4 m/s) (3-second gust) for buildings in Exposure Category B that are not located in a *hurricane-prone region*.

5. Roof trusses and rafters shall not span more than 40 feet (12 192 mm) between points of vertical support.
6. The use of the provisions for *conventional light-frame construction* in this section shall not be permitted for *Occupancy Category IV* buildings assigned to *Seismic Design Category B, C, D, E or F*, as determined in Section 1613.
7. *Conventional light-frame construction* is limited in irregular structures in *Seismic Design Category D or E*, as specified in Section 2308.12.6.

2308.2.1 Ultimate Design Basic Wind Speed, V_{ult} greater than 130 140 mph (3-second gust). Where the Ultimate Design basic Wind Speed, V_{ult} exceeds 130 mph (3-second gust), the provisions of either AF&PA WFCM, or the ICC 600 are permitted to be used. Wind speeds in Figure 1609A, 1609B, and 1609C shall be converted in accordance with Section 1609.3.1 for use with AF&PA WFCM or ICC 600.

Add new text as follows:

1405.6.3 Wind requirements. Sections 6.2.2.1 and 6.2.2.11 of TMS 402/ACI 530/ASCE 5 shall be modified as follows:

- 6.2.2.1 Except as provided in Section 6.2.2.11, prescriptive requirements for anchored masonry veneer shall not be used in areas where the basic wind speed exceeds 130 mph (177 km/hr) as given in ASCE 7.
- 6.2.2.11 Requirements in areas of high winds — The following requirements apply in areas where the basic wind speed exceeds 130 mph (177 km/hr) but does not exceed 150 mph (209 km/hr) and the building's mean roof height is less than or equal to 60 ft (18.3 m):
 - (a) Reduce the maximum wall area supported by each anchor to 70 percent of that required in Sections 6.2.2.5.6.1 and 6.2.2.5.6.2.
 - (b) Space anchors at a maximum 18 in. (457 mm) horizontally and vertically.
 - (c) Provide additional anchors around openings larger than 16 in. (406 mm) in either direction. Space anchors around perimeter of opening at a maximum of 24 in. (610 mm) on center. Place anchors within 12 in. (305 mm) of openings.

(Portions of the proposal not shown remain unchanged)

Commenter's Reason: The purpose of this code change is to correlate the prescriptive limits for empirical design of masonry and conventional wood frame construction with other proposals that are updating the wind speed maps in the IBC and the IRC. The wind speed maps in ASCE 7 are being updated to ultimate wind speeds as opposed to the ASD level wind speed maps that currently exist in ASCE 7 and in the IBC and IRC. See IBC code change for information on why the wind speed maps are being updated. While a way to convert the ultimate wind speeds to ASD level wind speeds is proposed in the IBC, the converted wind speeds do not match, from a geographic standpoint, the limitations the code previously imposed. Since the empirical provisions and conventional methods for wood frame construction, typically can't be calculated to equate to the lower level wind speeds at the current limit, including the fact that these provisions are missing some of the key wind resistant construction design

methods (e.g. gable endwall bracing, bond beam reinforcement, vertical wall reinforcement, etc.), the proposed limitations will roughly, maintain the current limitations on empirical and conventional methods that currently exist in terms of geographic location on the wind speed map. While some areas of the country will see a reduction in areas where empirical design of masonry or conventional construction would be allowed, other areas will see an increase in areas where these methods would be allowed.

The modification proposed by this public comment simply makes some editorial corrections and provides a similar correlation for brick veneer as described above.

Final Action: AS AM AMPC_____ D

S186-09/10 2208.1

Proposed Change as Submitted

Proponent: Bonnie Manley, representing American Iron and Steel Institute

Revise as follows:

2208.1 Storage racks. The design, testing and utilization of industrial steel storage racks made of cold-formed or hot-rolled steel structural members, shall be in accordance with RMI/ANSI MH 16.1. Where required by ASCE 7, the seismic design of storage racks shall be in accordance with the additional provisions of Section 15.5.3 of ASCE 7; ~~except that items (1), (2) and (3) of Section 15.5.3 of ASCE 7 do not apply when the rack design satisfies RMI/ANSI MH 16.1.~~

Reason: The exception recommended for deletion was inserted last cycle in Proposal S205-07/08 in order to coordinate the 2008 edition of RMI's ANSI/MH 16.1, *Specification for Design, Testing and Utilization of Industrial Steel Storage Racks*, with ASCE 7-05, which had originally adopted the 2002 edition of the RMI standard. The 2010 edition of ASCE 7 adopts and modifies the 2008 edition of ANSI/MH16.1. Consequently, the list of exceptions is no longer needed. Also, the word "additional" is added to emphasize that, for seismic design, steel storage racks must also be designed in accordance with the modifications contained in ASCE 7, Section 15.5.3.

Cost Impact: There is no anticipated impact on the cost of construction.

ICCFILENAME: Manley-S4-2208.1

Public Hearing Results

Committee Action:

Approved as Modified

Modify the proposal as follows:

2208.1 Storage racks. The design, testing and utilization of industrial steel storage racks made of cold-formed or hot-rolled steel structural members, shall be in accordance with the RMI/ANSI MH 16.1. Where required by ASCE 7, the seismic design of storage racks shall be in accordance with the ~~additional~~ provisions of Section 15.5.3 of ASCE 7.

Committee Reason: This proposal will correlate the reference to the RMI rack standard with the earthquake load requirements of ASCE 7. The modification removes a word that would cause confusion.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment 1:

Steven Winkel, FAIA, PE, and Robert E. Bachman, SE, representing the Federal Emergency Management Agency/Building Seismic Safety Council Code Resource Support Committee (FEMA/BSSC CRSC), requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

2208.1 Storage racks. The design, testing and utilization of industrial steel storage racks made of cold-formed or hot-rolled steel structural members, shall be in accordance with the RMI/ANSI MH 16.1. Where required by ASCE 7, the seismic design of storage racks shall be in

accordance with the provisions of Section 15.5.3 of ASCE 7 except that the mapped acceleration parameters S_s and S_1 shall be determined in accordance in Section 1613.5.1.

Commenter's Reason: The modification is proposed so that storage racks will be designed to the latest seismic maps that have been adopted in the 2012 IBC and ASCE 7-10. MH16.1 currently is a self-contained document that includes seismic maps that are same as the 2009 IBC and ASCE 7-05 seismic maps.

Public Comment 2:

Bonnie Manley, American Iron and Steel Institute, representing RMI, requests Disapproval.

Commenter's Reason: The 2010 edition of the RMI standard has been completed and is recommended for adoption in Proposal S187-09/10. A comment has been submitted to Proposal S187-09/10 requesting approval as modified. If that comment is successful, then this proposal (Proposal S186-09/10) is not needed and the action on this proposal should be disapproved.

Final Action: AS AM AMPC____ D

S187-09/10
2208.1, Chapter 35

Proposed Change as Submitted

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

1. Revise as follows:

2208.1 Storage racks. The design, testing and utilization of industrial steel storage racks made of cold-formed or hot-rolled steel structural members, shall be in accordance with the RMI/ANSI MH 16.1. Where required by ASCE 7, the seismic design of storage racks shall be in accordance with the additional provisions of Section 15.5.3 of ASCE 7, except that items (1), (2) and (3) of Section 15.5.3 of ASCE 7 do not apply when the rack design satisfies RMI/ANSI MH 16.1.

2. Revise Chapter 35 standard as follows:

RMI
ANSI/MH 16.1-08 11 Specification for Design, Testing and Utilization of Industrial Steel Storage Rack, 2011

Reason: This proposal updates the edition year of RMI's ANSI/MH 16.1, *Specification for Design, Testing and Utilization of Industrial Steel Storage Racks*, from 2008 to 2011. The document is expected to be completed in early 2010. The modification to the last sentence of Section 2208.1 coordinates the 2011 edition of the RMI standard with ASCE 7-10, which adopts the 2008 edition of the RMI standard. Also, the word "additional" is added to emphasize that, for seismic design, steel storage racks must also be designed in accordance with the applicable modifications contained in ASCE 7, Section 15.5.3.

Cost Impact: There is no anticipated impact on the cost of construction..

ICCFILENAME: Manley-S5-2208.1

Public Hearing Results

Committee Action:

Disapproved

Committee Reason: The proposal was disapproved at the request of the proponent while work continues on the next edition of the RMI Steel Rack Standard.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment:

Bonnie Manley, American Iron and Steel Institute, representing RMI, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

SECTION 2208 STEEL STORAGE RACKS

2208.1 Storage racks. The design, testing and utilization of industrial steel storage racks made of cold-formed or hot-rolled steel structural members, shall be in accordance with the RMI/ANSI MH 16.1. Where required by ASCE 7, the seismic design of storage racks shall be in accordance with the additional provisions of Section 15.5.3 of ASCE 7 ~~except that items (2) and (3) of Section 15.5.3 of ASCE 7 do not apply when the rack design satisfies RMI/ANSI MH 16.1.~~

RMI

ANSI/MH16.1-44 10 *Specification for Design, Testing and Utilization of Industrial Steel Storage Racks, 2010*~~2014~~

Commenter's Reason: The 2010 edition of the RMI standard has been completed (instead of the 2011 year as originally thought) and is recommended for adoption in this comment. Also, the exception in the last sentence of Section 2208.1 has been deleted. This language is not needed for coordination between the 2010 edition of MH16.1-10 and ASCE 7-10.

If this comment is approved, then the coordinating comment on Proposal S186-09/10, which requests disapproval, needs to be approved to ensure full coordination.

Final Action: AS AM AMPC _____ D

S188-09/10 1604.3.3, 2209.2.1

Proposed Change as Submitted

Proponent: Edwin Huston, representing National Council of Structural Engineers Associations- Code Advisory Committee - General Requirements Subcommittee

1. Delete without substitution:

~~**2209.2.1 Composite slabs on steel decks.** Composite slabs of concrete and steel deck shall be designed and constructed in accordance with ASCE 3.~~

(Renumber subsequent sections)

2. Revise as follows:

1604.3.3 Steel. The deflection of steel structural members shall not exceed that permitted by AISC 360, AISI S100, ~~ASCE 3~~, ASCE 8, SJI CJ-1.0, SJI JG-1.1, SJI K-1.1 or SJI LH/DLH-1.1, as applicable.

Reason: The referenced standard will be 21 years old by the time the 2012 IBC is available for use. We found no evidence of recent updates. We understand that the standard development organization, ASCE/SEI is beginning the process to update the standard. Until it is updated, it should be removed as a reference standard.

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

ICCFILENAME: HUSTON-S8-2209.2

Public Hearing Results

Committee Action:

Approved as Submitted

Committee Reason: This code removes the ASCE 3 standard for composite slab construction. The standard is out of print and availability is a problem. There are also some concerns such as not addressing serviceability.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment:

Todd Hawkinson, Hawkinson Associates, LLC, requests Disapproval.

Commenter's Reason: 1. The Section 2209.2.1 should remain requiring the engineer to follow an accepted standard, ASCE 3, for the design of composite slabs on steel deck. The standard, ASCE 3 as well as ASCE 9, "Standard for the Structural Design of Composite Slabs (ANSI/ASCE 3-91)" and "Standard Practice for Construction and Inspection of Composite Slabs (ANSI / ASCE 9-91)", respectively, are available and copies can be purchased from the Linda Hall Library, located at 5109 Cherry Street, Kansas City, MO 64110-2498, Phone: (816) 363-4600 or (800) 662-1545. permission for Reuse can be obtained from the American Society of Civil Engineers, ASCE.

2. Though the standard ASCE 3, as well ASCE 9, while technically do not meet the ICC criteria, the requirements are still valid, the standards are industry accepted and have been proven over time. Given that ASCE/SEI intends to update this standard in the future, maintaining the standard in this code, the International Building Code, requires engineers designing composite slabs on steel deck to adhere to the design requirements for the construction of such slabs and provides code officials design requirements that engineers must adhere to.

Final Action: AS AM AMPC____ D

S189-09/10

2209.1 through 2209.1.1.3, 2209.2 through 2209.3.6 (New)

Proposed Change as Submitted

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

1. Revise as follows:

2209.1 General. The design of cold-formed carbon and low-alloy steel structural members shall be in accordance with AISI S100. The design of cold-formed stainless-steel structural members shall be in accordance with ASCE 8. Cold-formed steel light-frame construction shall also comply with Section 2210. Where required, the seismic design of cold-formed steel structures shall be in accordance with the additional provisions of Section 2209.2.

~~2209.2~~ **2209.1.1 Steel decks.** The design and construction of cold-formed steel decks shall be in accordance with this section.

~~2209.2.4~~ **2209.1.1.1 Composite slabs on steel decks.** Composite slabs of concrete and steel deck shall be designed and constructed in accordance with ASCE 3.

~~2209.2.2~~ **2209.1.1.2 Noncomposite steel floor decks.** Noncomposite steel floor decks shall be permitted to be designed and constructed in accordance with ANSI/SDI-NC1.0, as modified in Section ~~2209.2.2.4~~ 2209.1.1.2.1.

~~2209.2.2.4~~ **2209.1.1.2.1 ANSI/SDI-NC1.0 Section 2.4B1.** Replace Section 2.4B1 of ANSI/SDI-NC1.0 with the following:

1. General: The design of the concrete slabs shall be done in accordance with the ACI *Building Code Requirements for Reinforced Concrete*. The minimum concrete thickness above the top of the deck shall be 1 1/2 inches (38 mm).

~~2209.2.3~~ **2209.1.1.3 Steel roof deck.** Steel roof decks shall be permitted to be designed and constructed in accordance with ANSI/SDI-RD1.0.

2. Add new text as follows:

2209.2 Seismic requirements for cold-formed steel structures. Where a response modification coefficient, R, in accordance with ASCE 7, Table 12.2-1 is used for the design of cold-formed steel structures, the structures shall be designed and detailed in accordance with the requirements of AISI S100, ASCE 8, or AISI S110 as modified in Section 2209.3.

2209.3 Modifications to AISI S110. The text of AISI S110 shall be modified as indicated in Sections 2209.3.1 through 2209.3.6.

2209.3.1 AISI S110, Section D1. Modify AISI S110, Section D1 to read as follows:

D1 Cold-Formed Steel Special Bolted Moment Frames (CFS-SBMF). Cold-formed steel–special bolted moment frames (CFS-SBMF) systems shall withstand significant inelastic deformations through friction and bearing at their bolted connections. Beams, columns, and connections shall satisfy the requirements in this section. CFS-SBMF systems shall be limited to one-story structures, no greater than 35 feet in height, without column splices and satisfying the requirements in this section. The CFS-SBMF shall engage all columns supporting the roof or floor above. The single size beam and single size column with the same bolted moment connection detail shall be used for each frame. The frame is to be supported on a level floor or foundation.

2209.3.2 AISI S110, Section D1.1.1. Modify AISI S110, Section D1.1.1 to read as follows:

D1.1.1 Connection Limitations. Beam-to-column connections in CFS-SBMF systems shall be bolted connections with snug-tight high-strength bolts. The bolt spacing and edge distance shall be in accordance with the limits of AISI S100, Section E3. The 8-bolt configuration shown in Table D1-1 shall be used. The faying surfaces of the beam and column in the bolted moment connection region shall be free of lubricants or debris.

2209.3.3 AISI S110, Section D1.2.1. Modify AISI S110, Section D1.2.1 to read as follows:

D1.2.1 Beam Limitations. In addition to the requirements of Section D1.2.3, beams in CFS-SBMF systems shall be ASTM A 653 galvanized 55 ksi (374 MPa) yield stress cold-formed steel C-sections members with lips, and designed in accordance with Chapter C of AISI S100. The beams shall have a minimum design thickness of 0.105 inches (2.67 mm). The beam depth shall be not less than 12 in (305 mm) or greater than 20 in (508 mm). The flat depth-to-thickness ratio of the web shall not exceed $6.18 \sqrt{E/F_y}$.

2209.3.4 AISI S110, Section D1.2.2. Modify AISI S110, Section D1.2.2 to read as follows:

D1.2.2 Column Limitations. In addition to the requirements of D1.2.3, columns in CFS-SBMF systems shall be ASTM A 500 Grade B cold-formed steel hollow structural section (HSS) members painted with a standard industrial finished surface, and designed in accordance with Chapter C of AISI S100. The column depth shall be not less than 8 in (203 mm) or greater than 12 in (305 mm). The flat depth-to-thickness ratio shall not exceed $1.40 \sqrt{E/F_y}$.

2209.3.5 AISI S110, Section D1.3. Modify AISI S110, Section D1.3 to read as follows:

D1.3 Design Story Drift. Where the applicable building code does not contain design coefficients for CSF-SBMF systems, the provisions of Appendix 1 shall apply. For structures having a period less than T_s , as defined in the applicable building code, alternate methods of computing Δ shall be permitted, provided such alternate methods are acceptable to the authority having jurisdiction.

2209.3.6 AISI S110, Section D1.5. Add a new Section D1.5 to read as follows:

D1.5 Period Determination. The fundamental period of the structure, T , in the direction under consideration shall be established in accordance with the applicable building code using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. Use of the approximate building period, T_a , as an alternative fundamental period shall not be permitted.

3. Add standard to Chapter 35 as follows:

AISI

S110-07 Standard for Seismic Design Of Cold-Formed Steel Structural Systems – Special Bolted Moment Frames.

Reason: This proposal introduces a reference to the first edition of AISI S110, *Standard For Seismic Design Of Cold-Formed Steel Structural Systems – Special Bolted Moment Frames*, which is based upon research conducted by Drs. Uang and Sato at UCSD (2007). Specifically, the standard focuses on providing design provisions for a newly defined seismic force resisting system entitled “Cold-formed Steel – Special Bolted Moment Frame” or CFS-SBMFs. This type of system is expected to experience substantial inelastic deformation during significant seismic events. It is intended that most of the inelastic deformation will take place at the bolted connections, due to slip and bearing. In order to develop the

designated mechanism, requirements based on the capacity design principles are provided for the design of the beams, columns and associated connections. Additionally, AISI S110 has specific requirements for the application of quality assurance and quality control procedures.

This system has been vetted through the BSSC process (Proposal 6-4R) and will be included in Part I of the 2009 NEHRP Provisions. Additionally, it has been introduced for consideration in the 2010 edition of ASCE 7 (Proposal TC-6-CH12-102-R3). As a first pass, Appendix 1 of AISI S110 makes recommendations on the seismic design coefficients of the CFS-SBMF system. These parameters have been introduced for consideration in the ASCE 7-10 proposal. The Response Modification Coefficient, R , is set at 3.5. Cyclic testing has shown that CFS-SBMFs have very large ductility capacity and significant hardening. This justifies the use of a value of 3.5 for the R -factor. The derivation of the deflection amplification factor, C_d , can be found in the AISI S110 Commentary, Section D1.3. Furthermore, a capacity design procedure has been provided in Section D1.5 of AISI S110 Commentary so that the designer can explicitly calculate the seismic load effect with overstrength, E_{mh} , at the design story drift level. Alternatively, a conservative system overstrength factor, γ , is also provided to be compatible with the conventional approach to compute E_{mh} in ASCE 7. Finally the height limitation of 35 feet for all SDCs is based on practical use only and not from any limits on the CFS-SBMF system strength.

Modifications to AISI S110 (2007 edition) were developed primarily in the BSSC's NEHRP process and adopted by ASCE 7 in Chapter 14. Since ASCE 7, Chapter 14 is not adopted in the IBC, these modifications need to be included within this proposal. The reasons for the modifications are as follows:

In Section 2209.3.1, the language was modified to reflect that CFS-SBMF needs to use the same-size beams and same-size columns throughout. In addition, the system needs to engage all primary columns, which support the roof or floor above, and those columns need to be supported on a level floor or foundation.

In Section 2209.3.2, the modifications were made for consistency with the test database.

In Section 2209.3.3, the modifications were made to be consistent with the test database (Uang and Sato, 2007), and limitations on the beam depth, steel grade, and surface treatment are added in Section D1.2.1 of AISI S110.

In Section 2209.3.4 the language was modified to be consistent with the test database (Uang and Sato, 2007), and limitations on column depth, steel grade, and surface treatment are added in Section D1.2.2 of AISI S110. The width-thickness ratio was reduced based upon further review of the test specimens.

In Section 2209.3.5, AISI S110 is intended primarily for industrial platforms; however, the standard is not limited to these non-building structures and does not prohibit architectural attachments (such as partition walls). As approved by the BSSC PUC, Proposal 6-4R reduced the $0.05h$ drift limit in Section D1.3 of AISI S110 to $0.03h$ in order to more closely align with the $0.025h$ drift limit of ASCE 7. Also, the BSSC PUC inserted the sentence, "In no case shall the design story drift exceed $0.05h$." to ensure an absolute upper bound on the drift limit. However, the ASCE 7 Seismic Subcommittee did not agree with the BSSC PUC and, instead, requested that ASCE 7, Section 12.12 not be overwritten by AISI S110. Therefore, the $0.05h$ drift limit in Section D1.3 of AISI S110 has been eliminated in deference to the design story drift limits found in ASCE 7, Section 12.12. In addition, the first sentence of the AISI S110, Section D1.3 was deleted because it was considered commentary.

Two additional modifications are presented in this proposal which are not being considered for inclusion in ASCE 7-10, Chapter 14, but were deemed important enough to be included in the IBC. These two items resulted from discussions with SEAOC. First in Section 2209.3.3, a minimum thickness for the beams was added to reflect the test database. Secondly, Section 2209.3.6 clarifies that the approximate fundamental period, T_a , in accordance with ASCE 7 Section 12.8.2.1, should not be used in the design of CFS-SBMF systems. Instead, the fundamental period of the structure, T , needs to be based upon the structural properties and deformational characteristics of the resisting elements. The approximate fundamental period in ASCE 7, Section 12.8.1 simply does not predict the period as accurately as needed for the variety of uses of this framing system.

Cost Impact: There is no anticipated impact on the cost of construction.

Analysis: A review of the standard(s) proposed for inclusion in the code, AISI S110-07, 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 September 24, 2009.

ICCFILENAME: Manley-S3-2209

Public Hearing Results

Note: The following analysis was not in the Code Change monograph but was published on the ICC website at <http://www.iccsafe.org/cs/codes/Documents/2009-10cycle/ProposedChanges/Standards-Analysis.pdf> :

Analysis: Review of proposed new standard AISI S110 indicated that, in the opinion of ICC Staff, the standard complies with ICC standards criteria.

Committee Action:

Approved as Modified

Modify the proposal as follows:

2209.3.1 AISI S110, Section D1. Modify Section D1 by revising to read as follows.

D1 Cold-Formed Steel Special Bolted Moment Frames (CFS-SBMF)

Cold-formed steel–special bolted moment frames (CFS-SBMF) systems shall withstand significant inelastic deformations through friction and bearing at their bolted connections. Beams, columns, and connections shall satisfy the requirements in this section. CFS-SBMF systems shall be limited to one-story structures, no greater than 35 feet in height, without column splices and satisfying the requirements in this section. The CFS-SBMF shall engage all columns supporting the roof or floor above. The single size beam and single size column with the same bolted moment connection detail shall be used for each frame. The frame ~~is to~~ shall be supported on a level floor or foundation.

...

2209.3.3 AISI S110, Section D1.2.1. Modify Section D1.2.1 by revising to read as follows.

D1.2.1 Beam Limitations

In addition to the requirements of Section D1.2.3, beams in CFS-SBMF systems shall be ASTM A653 galvanized 55 ksi (374 MPa) yield stress cold-formed steel C-sections members with lips, and designed in accordance with Chapter C of AISI S100. The beams

shall have a minimum design thickness of 0.105 inches (2.67 mm). The beam depth shall be not less than 12 in (305 mm) or greater than 20 in (508 mm). The flat depth-to-thickness ratio of the web shall not exceed $6.18 \sqrt{E / F_y}$.

D1.2.1.1 Single C-Section Beam Limitations

In addition to the requirements of Section D1.2.1, when single C-section beams are used, torsional effects shall be accounted for in the design.

...
~~2209.3.6 AISI S110, Section D1.5. Add a new Section D1.5 as follows.~~

D1.5 Period Determination

~~The fundamental period of the structure, T, in the direction under consideration shall be established in accordance with the applicable building code using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. Use of the approximate building period, T_a, as an alternative fundamental period shall not be permitted.~~

(Portions of proposal not shown are unchanged)

Committee Reason: This proposal adds requirements for cold-formed steel special bolted moment frames by reference to AISI S110. The modification coordinates the AISI S110 modifications for consistency with the updated earthquake load provisions in ASCE 7.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment:

Bonnie Manley, American Iron and Steel Institute, requests Approval as Modified by this Public Comment.

Further modify the proposal as follows:

2209.1 General. The design of cold-formed carbon and low-alloy steel structural members shall be in accordance with AISI S100. The design of cold-formed stainless-steel structural members shall be in accordance with ASCE 8. Cold-formed steel light-frame construction shall also comply with Section 2210. Where required, the seismic design of cold-formed steel structures shall be in accordance with the additional provisions of Section 2209.2.

2209.1.1 Steel decks. The design and construction of cold-formed steel decks shall be in accordance with this section.

2209.1.1.1 Composite slabs on steel decks. Composite slabs of concrete and steel deck shall be designed and constructed in accordance with ASCE 3.

2209.1.1.2 Noncomposite steel floor decks. Noncomposite steel floor decks shall be permitted to be designed and constructed in accordance with ANSI/SDI-NC1.0, as modified in Section 2209.1.1.2.1.

2209.1.1.2.1 ANSI/SDI-NC1.0 Section 2.4B1. Replace Section 2.4B1 of ANSI/SDI-NC1.0 with the following:

1. General: The design of the concrete slabs shall be done in accordance with the ACI *Building Code Requirements for Reinforced Concrete*. The minimum concrete thickness above the top of the deck shall be 1 1/2 inches (38 mm).

2209.1.1.3 Steel roof deck. Steel roof decks shall be permitted to be designed and constructed in accordance with ANSI/SBI-RD1.0.

2209.2 Seismic Requirements for Cold-Formed Steel Structures. Where a response modification coefficient, R, in accordance with ASCE 7, Table 12.2-1 is used for the design of cold-formed steel structures, the structures shall be designed and detailed in accordance with the requirements of AISI S100, ASCE 8, ~~or and, for cold-formed steel special bolted moment frames, AISI S110 as modified in Section 2209.3.~~

2209.3 Modifications to AISI S110. The text of AISI S110 shall be modified as indicated in Sections 2209.3.1 through 2209.3.5.

2209.3.1 AISI S110, Section D1. Modify AISI S110, Section D1 to read as follows:

~~D1 Cold-Formed Steel Special Bolted Moment Frames (CFS-SBMF).~~ Cold-formed steel special bolted moment frames (CFS-SBMF) systems shall withstand significant inelastic deformations through friction and bearing at their bolted connections. ~~Beams, columns, and connections shall satisfy the requirements in this section. CFS-SBMF systems shall be limited to one-story structures, no greater than 35 feet in height, without column splices and satisfying the requirements in this section. The CFS-SBMF shall engage all columns supporting the roof or floor above. The single size beam and single size column with the same bolted moment connection detail shall be used for each frame. The frame shall be supported on a level floor or foundation.~~

2209.3.2 AISI S110, Section D1.1.1. Modify AISI S110, Section D1.1.1 to read as follows:

~~D1.1.1 Connection Limitations.~~ Beam to column connections in CFS-SBMF systems shall be bolted connections with snug-tight high-strength bolts. ~~The bolt spacing and edge distance shall be in accordance with the limits of AISI S100, Section E3. The 8-bolt configuration shown in Table D1-1 shall be used. The faying surfaces of the beam and column in the bolted moment connection region shall be free of lubricants or debris.~~

2209.3.3 AISI S110, Section D1.2.1. Modify AISI S110, Section D1.2.1 to read as follows.

D1.2.1 Beam Limitations. In addition to the requirements of Section D1.2.3, beams in CFS-SBMF systems shall be ASTM A653 galvanized 55 ksi (374 MPa) yield stress cold-formed steel C-sections members with lips, and designed in accordance with Chapter C of AISI S100. The beams shall have a minimum design thickness of 0.105 inches (2.67 mm). The beam depth shall be not less than 12 in (305 mm) or greater than 20 in (508 mm). The flat depth to thickness ratio of the web shall not exceed $6.18 \sqrt{E/F_y}$.

D1.2.1.1 Single C-Section Beam Limitations. In addition to the requirements of Section D1.2.1, when single C-section beams are used, torsional effects shall be accounted for in the design.

2209.3.4 AISI S110, Section D1.2.2. Modify AISI S110, Section D1.2.2 to read as follows.

D1.2.2 Column Limitations. In addition to the requirements of D1.2.3, columns in CFS-SBMF systems shall be ASTM A500 Grade B cold-formed steel hollow structural section (HSS) members painted with a standard industrial finished surface, and designed in accordance with Chapter C of AISI S100. The column depth shall be not less than 8 in (203 mm) or greater than 12 in (305 mm). The flat depth to thickness ratio shall not exceed $1.40 \sqrt{E/F_y}$.

2209.3.5 AISI S110, Section D1.3. Modify AISI S110, Section D1.3 to read as follows.

D1.3 Design Story Drift. Where the applicable building code does not contain design coefficients for CFS-SBMF systems, the provisions of Appendix 1 shall apply. For structures having a period less than T_s , as defined in the applicable building code, alternate methods of computing Δ shall be permitted, provided such alternate methods are acceptable to the authority having jurisdiction.

AISI

AISI S110-07 Standard for Seismic Design Of Cold-Formed Steel Structural Systems – Special Bolted Moment Frames, with Supplement 1, dated 2009.

Commenter's Reason: The modifications made to AISI S110-07 in Proposal S189-09/10 resulted from the review of the document in the development of the 2009 NEHRP Recommended Provisions (FEMA P-750) and ASCE 7-10. To improve the usability of the IBC, AISI's Committee on Specification recently completed AISI S110-07 Supplement 1-09, which adopts all of the technical changes presented in IBC Section 2209.3 of Proposal S121-09/10. Consequently, this Public Comment recommends the deletion of these modifications to AISI S110-07 from the IBC.

Please note this supplement has been issued by AISI and is available for review at www.steel.org or <http://www.steel.org//AM/TemplateRedirect.cfm?Template=/CM/ContentDisplay.cfm&ContentID=36728>.

Final Action: AS AM AMPC D

S191-09/10

2209.2.1, 2209.2.1.1 (New) Chapter 35

Proposed Change as Submitted

Proponent: Thomas Sputo, Ph.D., PE, SE, Steel Deck Institute, representing Steel Deck Institute

1. Revise as follows:

2209.2.1 Composite slabs on steel decks. Composite slabs of concrete and steel deck shall be permitted to be designed and constructed in accordance with ASCE 3 ANSI/SDI-C1.0, as modified in Section 2209.2.1.1.

2. Add new text as follows:

2209.2.1.1 ANSI/SDI-C1.0 Section 2.4B6a. Replace Section 2.4B6a of ANSI/SDI-C1.0 with the following:

- a. Temperature and shrinkage reinforcement, consisting of welded wire fabric or reinforcing bars, shall have a minimum area of 0.00075 times the area of the concrete above the deck (per foot or meter of width), but shall not be less than the area provided by 6 x 6 – W1.4 x W1.4 welded wire fabric.

3. Add standard to Chapter 35 as follows:

SDI

C1.0-06 Standard for Composite Steel Floor Deck

Reason: ASCE 3-91 is proposed for deletion because it does not meet the criteria set forth in CP#28-05, revised 2/27/09 for referenced standards. Section 3.6.3.2 requires a reference standard to be maintained. This standard has not been reaffirmed since its approval by ANSI in 1992. The ASCE committee responsible for this standard has been inactive since approximately 1997 and has taken no action on this standard since then. "ASCE Rules for Standards Committees" (2006) require standards to be reaffirmed at intervals not to exceed 5 years (Section 5.8). Additionally, this standard is out-of-print and is therefore not readily available to code officials, designers, or users of the code.

ANSI/SDI C1.0 is proposed for inclusion because it is the current standard for the design of composite steel deck. This standard is readily available to code officials, designers, and other users of the code, both in print form and as a free download from the Steel Deck Institute website. Section 2.4B6a is modified to delete the option for the use of fibers because of lack of complete consensus among all interested parties on proper specification of fibers for the purpose of control of shrinkage and temperature fluctuation effects in concrete on composite steel deck.

Section 2.4B6a – Text as it appears in SDI-C1.0:

a. Temperature and shrinkage reinforcement, consisting of welded wire fabric or reinforcing bars, shall have a minimum area of 0.00075 times the area of the concrete above the deck (per foot or meter of width), but shall not be less than the area provided by 6 x 6 – W1.4 x W1.4 welded wire fabric.

Fibers shall be permitted as a suitable alternative to the welded wire fabric specified for temperature and shrinkage reinforcement. Cold-drawn steel fibers meeting the criteria of ASTM A820, at a minimum addition rate of 25 lb/cu yd (14.8 kg/cu meter), or macro synthetic fibers “Coarse fibers” (per ASTM Subcommittee C09.42), made from virgin polyolefin, shall have an equivalent diameter between 0.4 mm (0.016 in.) and 1.25 mm (0.05 in.), having a minimum aspect ratio (length/equivalent diameter) of 50, at a minimum addition rate of 4 lb./cu yd (2.4 kg/m³) are suitable to be used as minimum temperature and shrinkage reinforcement.

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, SDI C1.0-06, 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 September 24, 2009.

ICCFILENAME: Sputo-S1-2209.2.1

Public Hearing Results

Note: The following analysis was not in the Code Change monograph but was published on the ICC website at <http://www.iccsafe.org/cs/codes/Documents/2009-10cycle/ProposedChanges/Standards-Analysis.pdf> :

Analysis: Review of proposed new standard SDI-C1.0 indicated that, in the opinion of ICC Staff, the standard complies with ICC standards criteria.

Committee Action:

Disapproved

Committee Reason: The proposed reference standard, SDI-C1.0 is still in need of work. Questions have been raised on its treatment of serviceability and wheel loads. The need to exclude fiber reinforcement should be clarified.

Assembly Action:

None

Individual Consideration Agenda

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

Public Comment:

Thomas Sputo, Ph.D., PE, SE representing Steel Deck Institute, requests Approval as Modified by this Public Comment.

Modify the proposal as follows:

2209.2.1 Composite slabs on steel decks. Composite slabs of concrete and steel deck shall be permitted to be designed and constructed to resist loads other than wheel loads in accordance with ANSI/SDI-C1.0, Standard for Composite Steel Floor Deck, as modified in Section 2209.2.1.1.

2209.2.1.1 ANSI/SDI-C1.0 Section 2.4B6a. Replace Section 2.4B6a of ANSI/SDI-C1.0 with the following:

- a. Temperature and shrinkage reinforcement, consisting of welded wire fabric or reinforcing bars, shall have a minimum area of 0.00075 times the area of the concrete above the deck (per foot or meter of width), but shall not be less than the area provided by 6 x 6 – W1.4 x W1.4 welded wire fabric.

(Portions of proposal not shown remain unchanged)

Commenter's Reason: ASCE 3-91 was approved for deletion under Proposals S-188 and S-193. The fact that it has not been revised or reaffirmed since 1991 is sufficient reason for this action.

ASCE 3-91 should have been replaced with ANSI/SDI C1.0-2006, which is current, actively supported and today used almost exclusively by designers of composite floor slabs. The ANSI/SDI C1.0-2006 standard is also readily available as a free download by any building official or design professional at the SDI website. However, reasons were cited for not taking this action including serviceability, wheel loads and fibers. These reasons are not sufficient justification to create a void in the building code for the design of composite floor slabs.

SDI C1.0-2006 addresses serviceability of floor slabs in the same empirical manner as does other standards included in the IBC, such as ACI-318. It requires empirically based minimum amounts of reinforcement to control the effects of temperature and shrinkage. No consensus based analytical method exists to further address this issue for any type of temperature and shrinkage reinforcement. It is not a safety issue, and should not have been considered as a valid reason to not include ANSI/SDI C1.0-2006 in the IBC.

Designing for wheel loads is an analytical procedure that is outside the scope of ANSI/SDI C1.0-2006, which is intended to cover typical static loads. A designer following good engineering practice would use rational analysis combined with the requirements of a standard such as ANSI/SDI C1.0-2006 to properly address this rare, atypical condition. However, for clarification, we are proposing to modify S-191 to specifically exclude wheel loads.

Fibers are included in ANSI/SDI C1.0-2006 as an optional means to control the effects of temperature and shrinkage. However, inclusion of fibers in the building code has been controversial. Since this option is not essential to the function of ANSI/SDI C1.0-2006, SDI has most recently proposed to not include it in order to gain incorporation of the remaining parts of a standard that is already accepted by the design community into the IBC. However, S-191 may be modified to include the fiber option by deleting section 2209.2.1.1 from the proposal.

Final Action: AS AM AMPC____ D
