INTERNATIONAL CODE COUNCIL
2009/2010 CODE DEVELOPMENT CYCLE

PROPOSED CHANGES TO THE
2009 EDITIONS OF THE

INTERNATIONAL BUILDING CODE®

INTERNATIONAL ENERGY CONSERVATION CODE®

INTERNATIONAL EXISTING BUILDING CODE®

INTERNATIONAL FIRE CODE®

INTERNATIONAL FUEL GAS CODE®

INTERNATIONAL MECHANICAL CODE®

INTERNATIONAL PLUMBING CODE®

INTERNATIONAL PRIVATE SEWAGE DISPOSAL CODE®

INTERNATIONAL PROPERTY MAINTENANCE CODE®

INTERNATIONAL RESIDENTIAL CODE®

INTERNATIONAL WILDLAND-URBAN INTERFACE CODE®

INTERNATIONAL ZONING CODE®

October 24 2009 – November 11, 2009
Hilton Baltimore
Baltimore, MD

ICC
INTERNATIONAL
CODE COUNCIL®

First Printing

Publication Date: August 2009

PRINTED IN THE U.S.A.

TABLE OF CONTENTS

INTRODUCTION

The proposed changes published herein have been submitted in accordance with established procedures and are distributed for review. The publication of these changes constitutes neither endorsement nor question of them but is in accordance with established procedures so that any interested individuals may make their views known to the relevant code committee and others similarly interested. In furtherance of this purpose, the committee will hold an open public hearing at the date and place shown below for the purpose of receiving comments and arguments for or against such proposed changes. Those who are interested in testifying on any of the published changes are expected to be represented at these hearings.

This compilation of code change proposals is available in electronic form only. As part of ICC's green initiative, ICC will no longer print and distribute this document. The compilation of code change proposals will be posted on the ICC website, and CD copies will be distributed to all interested parties on our list.

2009 ICC CODE DEVELOPMENT HEARINGS

These proposed changes will be discussed in public hearings to be held on October 24, 2009 through October 31, 2009 and November 4-11, 2009 at the Hilton Baltimore, Baltimore, Maryland. The code committees will conduct their public hearings in accordance with the schedule shown on page xxxii.

REGISTRATION AND VOTING

All members of ICC may vote on any assembly motion on proposed code changes to all International Codes. **For identification purposes, eligible voting members must register, at no cost, in order to vote.** The registration desk will be open in the lobby of the convention center according to the following schedule:

Friday, October 23rd	3:00 pm to 6:00 pm
Saturday, October 24th through Wednesday November 11th	7:30 am to 5:00 pm

Council Policy #28-Code Development (page xii) requires that ICC's membership records regarding ICC members reflect the eligible voters 10 days prior to the start of the Code Development Hearings. This process includes new as well as changes to voting status. Section 5.7.4 of CP #28 (page xix) reads as follows:

> **5.7.4 Eligible Voters:** All members of ICC in attendance at the public hearing shall be eligible to vote on floor motions. Only one vote authorized for each eligible attendee. Code Development Committee member shall be eligible to vote on floor motions. Application, whether new or updated, for ICC membership must be received by the Code Council ten days prior to the commencement of the first day of the public hearing.

As such, new membership application as well as renewal applications must be received by ICC's Member Services Department by October 14, 2009. These records will be used to verify eligible voter status for the Code Development Hearings. Members are strongly encouraged to review their membership records for accuracy well in advance of the hearings so that any necessary changes are made prior to the October 14, 2009 deadline. For information on application for new membership and membership renewal, please go to www.iccsafe.org/membership/join.html or call ICC Member Services at 1-888-ICC SAFE (422-7233)

It should be noted that a corporate member has a single vote. Only one representative of a corporate member will be issued a voting badge. ICC Staff will be contacting corporate members regarding who the designated voting representative will be.

ADVANCED REGISTRATION

You are encouraged to advance register by filling out the registration form available at www.iccsafe.org/codesforum.

CODE DEVELOPMENT PROCESS CHANGES

As noted in the posted Advisory Statement of February 4, 2009, the revised Code Development Process includes maintaining the current 3-year publication cycle with a single cycle of code development between code editions. The schedule for the 2009/2010 Code Development Cycle is the transitional schedule for the revised code development process. As noted, there will be two Final Action Hearings in 2010—one for the modified Group A, and one for the modified Group B. The codes that will comprise the Group A and Group B hearings will be announced prior to the Code Development Hearings in Baltimore. See the Code Development Process Notes included with the Schedule on page viii.

PROCEDURES

The procedures for the conduct of the public hearing are published in *Council Policy #28-Code Development (CP#28)* ("Procedures") on page xii. The attention of interested parties is specifically directed to Section 5.0 of the Procedures. These procedures indicate the conduct of, and opportunity to participate in the ICC Code Development Process. Please review these procedures carefully to familiarize yourself with the process.

There have been a number of revisions to the procedures. Included among these revisions are the following:

Section 2.3: **Supplements:** ICC will no longer produce a Supplement to each edition of the I-Codes. A new edition of the I-Codes will be based upon activity of a single code change cycle.

Section 3.3.3: **Multiple code change proposals:** A proponent is not permitted to submit multiple code changes to one section of a code unless the subject matter of each proposal is different.

Section 4.5.1: **Administrative update of standards:** Updating of standards without a change to code text (administrative update) shall be a code change proposal dealt with by the Administrative Code Development Committee. The updating of standards procedures have also changed. See discussion on updating of standards on page vi.

Section 4.7: **Code change posting:** All code change proposals are required to be posted on the ICC website 30 days before the code development hearings. Published copies will not be provided.

Section 5.2.2: **Conflict of interest:** Clarification is added that a committee member who steps down from the dais because of a conflict of interest is allowed to provide testimony from the floor on that code change proposal.

Section 5.4.6.2: **Proponent rebuttal testimony:** Where the code change proposal is submitted by multiple proponents, only one proponent of the joint submittal to be allotted additional time for rebuttal.

Section 5.5.2: **Modifications:** The chair rules a modification in or out of order. The chair's decision is final. No challenge in a point of order is allowed for this ruling.

Section 5.7.3: **Assembly Actions:** Several changes have been made to assembly actions. See explanation page v

Section 7.3.8.2: **Initial motion at final action hearings:** A successful assembly action becomes the initial motion at the final action hearings. See explanation page v.

ASSEMBLY ACTION

The procedures regarding assembly action at the Code Development Hearings have been revised to place more weight on the results of that action (see Section 5.7 of CP #28 on page viii). Some important items to note regarding assembly action are:

- A successful assembly action now requires a 2/3 majority rather than a simple majority.
- After the committee decision on a code change proposal is announced by the moderator, any one in the assembly may make a motion for assembly action.
- After a motion for assembly action is made and seconded, the moderator calls for a floor vote in accordance with Section 5.7.2. *No additional testimony will be permitted.*
- A successful assembly action becomes the initial motion considered at the Final Action Hearings. This also means that the required vote at the Final Action Hearings to uphold the assembly action is a simple majority.

MULTIPLE PART CODE CHANGE PROPOSALS

It is common for ICC to receive code change proposals for more than one code or more than 1 part of a code that is the responsibility of more than one committee. For instance, a code change proposal could be proposing related changes to the text of IBC Chapter 4 (IBC-General), IBC Chapter 7 (IBC-Fire Safety), and the IFC Chapter 27 (IFC). When this occurs, a single committee will now hear all of the parts, unless one of the parts is a change to the IRC, in which case the respective IRC committee will hear that part separately.

ADMINISTRATIVE CODE DEVELOPMENT COMMITTEE

A new committee for the 2009/2010 Code Change Cycle and going forward is the Administrative Code Development Committee. This committee will hear code change proposals to the administrative provisions of the I-Codes (Chapter 1 of each code.) The purpose of this committee is to achieve, inasmuch as possible, uniformity in the administrative provisions of all I-Codes when such uniformity is warranted.

ANALYSIS STATEMENTS

Various proposed changes published herein contain an “analysis” that appears after the proponent’s reason. These comments do not advocate action by the code committees or the voting membership for or against a proposal. The purpose of such comments is to identify pertinent information that is relevant to the consideration of the proposed change by all interested parties, including those testifying, the code committees and the voting membership. Staff analyses customarily identify such things as: conflicts and duplication within a proposed change and with other proposed changes and/or current code text; deficiencies in proposed text and/or substantiation; text problems such as wording defects and vagueness; background information on the development of current text; and staff’s review of proposed reference standards for compliance with the Procedures. Lack of an analysis indicates neither support for, nor opposition to a proposal.

REFERENCE STANDARDS

Proposed changes that include the addition of a reference to a new standard (i.e. a standard that is not currently referenced in the I-Codes.) will include in the proposal the number, title and edition of the proposed standard. This identifies to all interested parties the precise document that is being proposed and which would be included in the referenced standards chapter of the code if the proposed change is approved. Proponents of code changes which propose a new standard have been directed to forward copies of the standard to the Code Committee and an analysis statement will be posted on the ICC website indication the status of compliance of the standard with the ICC referenced standards criteria in Section 3.6 of CP #28 (see page xiv). (See the ICC Website page xi) The analysis statements for referenced standards will be posted on or before September 24, 2009. This information will also be published and made available at the hearings.

REFERENCED STANDARDS UPDATES

At the end of the agenda of the Administrative Code Development Committee is a code change proposal that is an administrative update of the referenced standards contained in the I-Codes. This code change proposal, ADM39-09/10 contains a list of standards for which the respective promulgators have indicated that the standard has been updated. The codes that these standards appear in are indicated beside each listed referenced standard. This update will then apply to every code in which the standard appears.

It should be noted that in accordance with Section 4.5.1 of CP #28 (see page xvi), standards promulgators have until December 1, 2011 to finalize and publish any updates to standards in the administrative update. If the standard is not finalized by December 1, 2011, the code will be revised to reference the previously listed year edition of that standard.

MODIFICATIONS

Those who are submitting modification for consideration by the respective Code Development Committee are required to submit a Copyright Release in order to have their modifications considered (Section 3.3.4.5 of CP #28). It is preferred that such release be executed in advance – the form is at http://www.iccsafe.org/cs/codes/publicforms.htm. Copyright release forms will also be available at the hearings. Please note that an individual need only sign one copyright release for submittals of all code change proposals, modification, and public comments in this code change cycle for which the individual might be responsible. **Please be sure to review Section 5.5.2 of CP #28 for the modification process.** The Chair of the respective code development committee rules a modification in or out of order. That ruling is final, with no challenge allowed. The proponent submitting a modification is required to supply 20 printed copies. The minimum font size must be 12 point.

CODE CORRELATION COMMITTEE

In every code change cycle, there are code change proposals that are strictly editorial. The Code Correlation Committee approves all proposals deemed editorial. A list of code correlation committee actions will be posted on the ICC website by September 24, 2009.

2009/2010 ICC CODE DEVELOPMENT SCHEDULE

STEP IN CODE DEVELOPMENT CYCLE	DATE	
DEADLINE FOR RECEIPT OF APPLICATIONS FOR CODE COMMITTEES	January 2, 2009	
DEADLINE FOR RECEIPT OF CODE CHANGE PROPOSALS	June 1, 2009	
WEB POSTING OF "PROPOSED CHANGES TO THE I-CODES"	August 24, 2009	
DISTRIBUTION DATE OF "PROPOSED CHANGES TO THE I-CODES" (Limited distribution – see notes)	October 3, 2009	
CODE DEVELOPMENT HEARING (CDH) ALL CODES – see notes	**October 24 2009 – November 11, 2009** Hilton Baltimore Baltimore, MD	
WEB POSTING OF "REPORT OF THE PUBLIC HEARING"	December 16, 2009	
DISTRIBUTION DATE OF "REPORT OF THE PUBLIC HEARING" (Limited distribution – see notes)	January 11, 2010	
IN ACCORDANCE WITH THE NEW CODE DEVELOPMENT PROCESS (see notes), THE CODES WILL BE SPLIT INTO TWO GROUPS WITH SEPARATE PUBLIC COMMENT DEADLINES AND FINAL ACTION HEARINGS		
	GROUP A (see notes)	GROUP B (see notes)
DEADLINE FOR RECEIPT OF PUBLIC COMMENTS	February 8, 2010	July 1, 2010
WEB POSTING OF PUBLIC COMMENTS "FINAL ACTION AGENDA"	March 15, 2010	August 26, 2010
DISTRIBUTION DATE OF PUBLIC COMMENTS "FINAL ACTION AGENDA" (Limited distribution see notes)	April 16, 2010	September 27, 2010
FINAL ACTION HEARINGS (FAH)	**May 14 – 23, 2010** Dallas, TX	**Oct 28 – Nov 1, 1020** Charlotte, NC
ANNUAL CONFERENCES	**October 24 – November 11, 2009** 2009 ICC Annual Conference and Code Development Hearing Balitmore, MD **October 25 – November 1, 2010** 2010 ICC Annual Conference and Final Action Hearing Charlotte, NC	
RESULTING PUBLICATION	2012 – I-Codes (available April, 2011)	

Code Development Process Notes:

As noted in the posted Advisory Statement of February 4, 2009, the revised Code Development Process includes maintaining the current 3-year publication cycle with a single cycle of code development between code editions. Implemented as follows:

- Transitional Process – 2009/2010 only
    - Single Code Development Hearing (CDH) for all codes in 2009
    - Two Final Action Hearings (FAH) in 2010 – modified Groups A and B (see below)
    - Public 2012 edition in April, 2011
- New Process – 2012/2013 and going forward
    - Code Committee application deadline (all codes); June 1, 2011
    - Codes split into two groups: Group A and Group B
    - Group A: IBC; IFGC; IMC; IPC; IPSDC
        - Code change deadline: January 3, 2012
        - Code Development Hearing: April/May 2012
        - Final Action Hearing: October/November 2012 (in conjunction with Annual Conference)
    - Group B: Admin (Ch. 1 of I-Codes); IEBC; IECC; IFC; IPerfC; IPMC; IRC; IWUIC; IZC
        - Code change deadline: January 3, 2013
        - Code Development Hearing: April/May 2013
        - Final Action Hearing: October/November 2013 (in conjunction with Annual Conference)
    - Publish 2015 edition in April, 2014
    - Repeat for subsequent editions

2009/2010 Cycle Notes:

- Revised code change deadline of June 1st posted on March 19th

- Distribution date: Complimentary code development cycle document distribution will be limited to CD's mailed to those who are on ICC's code change document mailing list.

- Code Development Hearings: The Baltimore Code Development Hearings will include 12 I-Codes (no changes to the ICC Performance Code. The hearings will be held in the conventional two track format with the hearings split before and after the Annual Conference during the periods of October 24 – 31 and November 4 – 11. The specific codes and hearing order to be determined based on code change volume.

- Final Action Hearing Groupings: Final Action Hearing logistics dictate that the hearings will not be split along established Group A and B codes (see above) due to hotel commitments which limit the amount of hearing time at the October/2010 FAH versus the May/2010 FAH. Tentatively, the May/2010 FAH will include Group A codes plus certain Group B codes to be determined based on code change volume.

2009/2010 STAFF SECRETARIES

IBC-General **Chapters 1-6. 12, 13, 27-34**	**IBC-Fire Safety** **Chapters 7, 8, 9, 14, 26**	**IBC-Means of Egress** **Chapters 10, 11**	**IBC-Structural** **Chapters 15-25**
Kermit Robinson ICC Whittier District Office 1-888-ICC-SAFE, ext. 3317 FAX: 562/699-4522 krobinson@iccsafe.org	Ed Wirtschoreck ICC Chicago District Office 1-888-ICC-SAFE, ext 4317 FAX: 708/799-0320 ewirtschoreck@iccsafe.org	Kim Paarlberg ICC Indianapolis Field Office 1-888-ICC-SAFE, ext 4306 FAX: 708/799-0320 kpearlberg@iccsafe.org	Alan Carr ICC NW Resource Center 1-888-ICC-SAFE, ext 7601 FAX: 425/637-8939 acarr@iccsafe.org

IEBC	**IECC**	**IFC**	**IFGC**
BethTubbs ICC Northbridge Field Office 1-888-ICC-SAFE, ext 7708 FAX: 419/ 730-6531 btubbs@iccsafe.org	Dave Bowman ICC Chicago District Office 1-888-ICC-SAFE, ext 4323 FAX: 708/799-0320 dmeyers@iccsafe.org	Bill Rehr/ Beth Tubbs ICC Chicago District Office 1-888-ICC-SAFE, ext 4342 FAX: 708/799-0320 brehr@iccsafe.org btubbs@iccsafe.org	Gregg Gress ICC Chicago District Office 1-888-ICC-SAFE, ext 4343 FAX: 708/799-0320 ggress@iccsafe.org

IMC	**ICC PC**	**IPMC**	**IPC/IPSDC**
Gregg Gress ICC Chicago District Office 1-888-ICC-SAFE, ext 4343 FAX: 708/799-0320 ggress@iccsafe.org	BethTubbs ICC Northbridge Field Office 1-888-ICC-SAFE, ext 7708 FAX: 419/ 730-6531 btubbs@iccsafe.org	Ed Wirtschoreck ICC Chicago District Office 1-888-ICC-SAFE, ext 4317 FAX: 708/799-0320 ewirtschoreck@iccsafe.org	Fred Grable ICC Chicago District Office 1-888-ICC-SAFE, ext 4359 FAX: 708/799-0320 fgrable@iccsafe.org

IRC-Building/Energy	**IRC Mechanical**	**IRC Plumbing**	**IWUIC**
Larry Franks/ Dave Bowman ICC Northbridge Field Office 1-888-ICC-SAFE, ext 5279 FAX: 205/592-7001 lfranks@iccsafe.org dbowman@iccsafe.org	Gregg Gress ICC Chicago District Office 1-888-ICC-SAFE, ext 4343 FAX: 708/799-0320 ggress@iccsafe.org	Fred Grable ICC Chicago District Office 1-888-ICC-SAFE, ext 4359 FAX: 708/799-0320 fgrable@iccsafe.org	Bill Rehr ICC Chicago District Office 1-888-ICC-SAFE, ext 4342 FAX: 708/799-0320 brehr@iccsafe.org

IZC	**ADMINISTRATIVE** **Chapter 1** **All Codes Except IRC**
Ed Wirtschoreck ICC Chicago District Office 1-888-ICC-SAFE, ext 4317 FAX: 708/799-0320 ewirtschoreck@iccsafe.org	Dave Bowman ICC Chicago District Office 1-888-ICC-SAFE, ext 4323 FAX: 708/799-0320 dbowman@iccsafe.org

SCOPING REVISIONS – WITHIN THE IBC

The 2009/2010 Staff Secretaries assignments on page ix indicate which chapters of the International Building Code are generally within the responsibility of each IBC Code Committee. However, within each of these IBC Chapters are subjects that are most appropriately maintained by another IBC Code Committee. For example, the provisions of Section 3008.1 deal with occupant evacuation elevators. Therefore, even though Chapter 30 is within the responsibility of the IBC General Committee, this section would most appropriately be maintained by the IBC Means of Egress Committee. The following table indicates responsibilities by IBC Code Committees other than the main committee for those chapters, for code changes submitted for the 2009/2010 Cycle.

SECTION	CHAPTER MAINTAINED BY	SECTION MAINTAINED BY	CODE CHANGES
403.2.3	IBC-General	IBC-Structural	E5 Part I (Heard by IBC-MOE)
403.5.1	IBC-General	IBC-Structural	E5 Part I (Heard by IBC-MOE)
403.5.2	IBC-General	IBC-Means of Egress	G46
403.5.4	IBC-General	IBC-Structural	E5 Part I (Heard by IBC-MOE)
403.5.4	IBC-General	IBC-Means of Egress	G47
403.6.1	IBC-General	IBC-Means of Egress	G48, G49
408.3.8	IBC-General	IBC-Structural	E5 Part I (Heard by IBC-MOE)
410.5.3.1	IBC-General	IBC-Structural	E5 Part I (Heard by IBC-MOE)
419.3.0	IBC-General	IBC-Means of Egress	G79
1505.1.0	IBC-Structural	IBC-Fire Safety	S10, S11
1505.8.0	IBC-Structural	IBC-Fire Safety	S12, S13
1507.16.0	IBC-Structural	IBC-Fire Safety	S10, S11
1508.1.0	IBC-Structural	IBC-Fire Safety	S24
1508.2.0	IBC-Structural	IBC-Fire Safety	S25
1509.0.0	IBC-Structural	IBC-General	S26, S27
1509.6.(new)	IBC-Structural	IBC-General	S28
1704.15.0	IBC-Structural	IBC-Fire Safety	S126, S127,S128
3007.1.0	IBC-General	IBC-Means of Egress	G48,G157
3007.2.(new)	IBC-General	IBC-Means of Egress	G158, G159
3007.2.0	IBC-General	IBC-Means of Egress	G160
3007.3.(new)	IBC-General	IBC-Means of Egress	G158, G161
3007.4.(new)	IBC-General	IBC-Means of Egress	G162
3007.4.2	IBC-General	IBC-Means of Egress	G163
3007.4.3	IBC-General	IBC-Means of Egress	G176
3007.5.1.(NEW)	IBC-General	IBC-Means of Egress	G164
3007.7.1	IBC-General	IBC-Means of Egress	G165, G166
3007.8.0	IBC-General	IBC-Means of Egress	G167
3008.1.0	IBC-General	IBC-Means of Egress	G168, G170
3008.1.1	IBC-General	IBC-Means of Egress	G169
3008.10.0	IBC-General	IBC-Means of Egress	G174
3008.10.1	IBC-General	IBC-Means of Egress	G175
3008.11.3	IBC-General	IBC-Means of Egress	G176
3008.11.5	IBC-General	IBC-Means of Egress	G177
3008.3.(NEW)	IBC-General	IBC-Means of Egress	G165, G166
3008.4.(NEW)	IBC-General	IBC-Means of Egress	G171
3008.4.0	IBC-General	IBC-Means of Egress	G46
3008.7.0	IBC-General	IBC-Means of Egress	G172
3008.9.0	IBC-General	IBC-Means of Egress	G173
3401.4.0	IBC-General	IBC-Structural	G190
3401.4.1	IBC-General	IBC-Structural	G191
3401.4.3	IBC-General	IBC-Structural	G190
3401.5.(NEW)	IBC-General	IBC-Structural	G192

SECTION	CHAPTER MAINTAINED BY	SECTION MAINTAINED BY	CODE CHANGES
3402.1.0	IBC-General	IBC-Structural	G193
3403.4.1	IBC-General	IBC-Structural	G190
3404.4.1	IBC-General	IBC-Structural	G190
3405.1.1	IBC-General	IBC-Structural	G192
3405.2.0	IBC-General	IBC-Structural	G193, G194
3405.2.1	IBC-General	IBC-Structural	G193, G190
3405.2.2	IBC-General	IBC-Structural	G193
3405.2.3	IBC-General	IBC-Structural	G193, G195
3405.3.0	IBC-General	IBC-Structural	G193
3405.4.0	IBC-General	IBC-Structural	G193, G194
3405.5.0	IBC-General	IBC-Structural	G196
3408.4.0	IBC-General	IBC-Structural	G190, G197
3408.4.0	IBC-General	IBC-Structural	G190
403.2.3	IBC-General	IBC-Structural	E5 Part I (Heard by IBC-MOE)
403.5.1	IBC-General	IBC-Structural	E5 Part I (Heard by IBC-MOE)
403.5.2	IBC-General	IBC-Means of Egress	G46
403.5.4	IBC-General	IBC-Structural	E5 Part I (Heard by IBC-MOE)
403.5.4	IBC-General	IBC-Means of Egress	G47
403.6.1	IBC-General	IBC-Means of Egress	G48, G49
408.3.8	IBC-General	IBC-Structural	E5 Part I (Heard by IBC-MOE)
410.5.3.1	IBC-General	IBC-Structural	E5 Part I (Heard by IBC-MOE)
419.3.0	IBC-General	IBC-Means of Egress	G79

ICC WEBSITE – WWW.ICCSAFE.ORG

While great care has been exercised in the publication of this document, errata to proposed changes may occur. Errata, if any, identified prior to the Code Development Hearings will be posted on the ICC website at http://www.iccsafe.org. Users are encouraged to periodically review the ICC Website for updates to errata to the 2009/2010 Code Development Cycle Proposed Changes. Additionally, analysis statements for code changes which propose a new referenced standard will be updated to reflect the staff review of the standard for compliance with Section 3.6 of the Procedures.

CP# 28-05 CODE DEVELOPMENT

Approved: 9/24/05
Revised: 2/27/09

CP # 28-05 is an update to *ICC's Code Development Process for the International Codes* dated May 15, 2004.

1.0 Introduction

1.1 Purpose: The purpose of this Council Policy is to prescribe the Rules of Procedure utilized in the continued development and maintenance of the International Codes (Codes).

1.2 Objectives: The ICC Code Development Process has the following objectives:

1.2.1 The timely evaluation and recognition of technological developments pertaining to construction regulations.
1.2.2 The open discussion of proposals by all parties desiring to participate.
1.2.3 The final determination of Code text by officials representing code enforcement and regulatory agencies and by honorary members.

1.3 Code Publication: The ICC Board of Directors (ICC Board) shall determine the title and the general purpose and scope of each Code published by the ICC.

1.3.1 Code Correlation: The provisions of all Codes shall be consistent with one another so that conflicts between the Codes do not occur. Where a given subject matter or code text could appear in more than one Code, the ICC Board shall determine which Code shall be the primary document, and therefore which code development committee shall be responsible for review and maintenance of the code text. Duplication of content or text between Codes shall be limited to the minimum extent necessary for practical usability of the Codes, as determined in accordance with Section 4.4.

1.4 Process Maintenance: The review and maintenance of the Code Development Process and these Rules of Procedure shall be by the ICC Board. The manner in which ICC codes are developed embodies core principles of the organization. One of those principles is that the final content of ICC codes is determined by a majority vote of the governmental and honorary members. It is the policy of the Board that there shall be no change to this principle without the affirmation of two-thirds of the governmental and honorary members responding.

1.5 Secretariat: The Chief Executive Officer shall assign a Secretariat for each of the Codes. All correspondence relating to code change proposals and public comments shall be addressed to the Secretariat.

1.6 Video Taping: Individuals requesting permission to video tape any meeting, or portion thereof, shall be required to provide the ICC with a release of responsibility disclaimer and shall acknowledge that they have insurance coverage for liability and misuse of video tape materials. Equipment and the process used to video tape shall, in the judgment of the ICC Secretariat, be conducted in a manner that is not disruptive to the meeting. The ICC shall not be responsible for equipment, personnel or any other provision necessary to accomplish the videotaping. An unedited copy of the video tape shall be forwarded to ICC within 30 days of the meeting.

2.0 Code Development Cycle

2.1 Intent: The code development cycle shall consist of the complete consideration of code change proposals in accordance with the procedures herein specified, commencing with the deadline for submission of code change proposals (see Section 3.5) and ending with publication of final action on the code change proposals (see Section 7.6).

2.2 **New Editions:** The ICC Board shall determine the schedule for publishing new editions of the Codes. Each new edition shall incorporate the results of the code development activity since the last edition.

2.3 **Supplements:** The results of code development activity between editions may be published.

2.4 **Emergency Procedures:** In the event that the ICC Board determines that an emergency amendment to any Code is warranted, the same may be adopted by the ICC Board. Such action shall require an affirmative vote of at least two-thirds of the ICC Board.

The ICC membership shall be notified within ten days after the ICC Boards' official action of any emergency amendment. At the next Annual Business Meeting, any emergency amendment shall be presented to the members for ratification by a majority of the ICC Governmental Member Representatives and Honorary Members present and voting.

All code revisions pursuant to these emergency procedures and the reasons for such corrective action shall be published as soon as practicable after ICC Board action. Such revisions shall be identified as an emergency amendment.

Emergency amendments to any Code shall not be considered as a retro-active requirement to the Code. Incorporation of the emergency amendment into the adopted Code shall be subjected to the process established by the adopting authority.

3.0 Submittal of Code Change Proposals

3.1 **Intent:** Any interested person, persons or group may submit a code change proposal which will be duly considered when in conformance to these Rules of Procedure.

3.2 **Withdrawal of Proposal:** A code change proposal may be withdrawn by the proponent (WP) at any time prior to Final Action Consideration of that proposal. A withdrawn code change proposal shall not be subject to a public hearing, motions, or Final Action Consideration.

3.3 **Form and Content of Code Change Submittals:** Each code change proposal shall be submitted separately and shall be complete in itself. Each submittal shall contain the following information:

3.3.1 **Proponent:** Each code change proposal shall include the name, title, mailing address, telephone number, and email address of the proponent.

3.3.1.1 If a group, organization or committee submits a code change proposal, an individual with prime responsibility shall be indicated.

3.3.1.2 If a proponent submits a code change on behalf of a client, group, organization or committee, the name and mailing address of the client, group, organization or committee shall be indicated.

3.3.2 **Code Reference:** Each code change proposal shall relate to the applicable code sections(s) in the latest edition of the Code.

3.3.2.1 If more than one section in the Code is affected by a code change proposal, appropriate proposals shall be included for all such affected sections.

3.3.2.2 If more than one Code is affected by a code change proposal, appropriate proposals shall be included for all such affected Codes and appropriate cross referencing shall be included in the supporting information.

3.3.3 **Multiple code change proposals to a code section.** A proponent shall not submit multiple code change proposals to the same code section. When a proponent submits multiple code change proposals to the same section, the proposals shall be considered as incomplete proposals and processed in accordance with Section 4.3. This restriction shall not apply to code change proposals that attempt to address differing subject matter within a code section.

3.3.4 **Text Presentation:** The text proposal shall be presented in the specific wording desired with deletions shown struck out with a single line and additions shown underlined with a single line.

3.3.4.1 A charging statement shall indicate the referenced code section(s) and whether the proposal is intended to be an addition, a deletion or a revision to existing Code text.
3.3.4.2 Whenever practical, the existing wording of the text shall be preserved with only such deletions and additions as necessary to accomplish the desired change.
3.3.4.3 Each proposal shall be in proper code format and terminology.
3.3.4.4 Each proposal shall be complete and specific in the text to eliminate unnecessary confusion or misinterpretation.
3.3.4.5 The proposed text shall be in mandatory terms.

3.3.5 **Supporting Information:** Each code change proposal shall include sufficient supporting information to indicate how the proposal is intended to affect the intent and application of the Code.

3.3.5.1 **Purpose:** The proponent shall clearly state the purpose of the proposed code change (e.g. clarify the Code; revise outdated material; substitute new or revised material for current provisions of the Code; add new requirements to the Code; delete current requirements, etc.)
3.3.5.2 Reasons: The proponent shall justify changing the current Code provisions, stating why the proposal is superior to the current provisions of the Code. Proposals which add or delete requirements shall be supported by a logical explanation which clearly shows why the current Code provisions are inadequate or overly restrictive, specifies the shortcomings of the current Code provisions and explains how such proposals will improve the Code.
3.3.5.3 **Substantiation:** The proponent shall substantiate the proposed code change based on technical information and substantiation. Substantiation provided which is reviewed in accordance with Section 4.2 and determined as not germane to the technical issues addressed in the proposed code change shall be identified as such. The proponent shall be notified that the proposal is considered an incomplete proposal in accordance with Section 4.3 and the proposal shall be held until the deficiencies are corrected. The proponent shall have the right to appeal this action in accordance with the policy of the ICC Board. The burden of providing substantiating material lies with the proponent of the code change proposal.
3.3.5.4 **Bibliography:** The proponent shall submit a bibliography of any substantiating material submitted with the code change proposal. The bibliography shall be published with the code change and the proponent shall make the substantiating materials available for review at the appropriate ICC office and during the public hearing.
3.3.5.5 Copyright Release: The proponent of code change proposals, floor modifications and public comments shall sign a copyright release reading: “I hereby grant and assign to ICC all rights in copyright I may have in any authorship contributions I make to ICC in connection with any proposal and public comment, in its original form submitted or revised form, including written and verbal modifications submitted in accordance Section 5.5.2. I understand that I will have no rights in any ICC publications that use such contributions in the form submitted by me or another similar form and certify that such contributions are not protected by the copyright of any other person or entity.”
3.3.5.6 **Cost Impact:** The proponent shall indicate one of the following regarding the cost impact of the code change proposal: 1) the code change proposal will increase the cost of construction; or 2) the code change proposal will not increase the cost of construction. This information will be included in the published code change proposal.

3.4 **Number:** One copy of each code change proposal, two copies of each proposed new referenced standard and one copy of all substantiating information shall be submitted. Additional copies may be requested when determined necessary by the Secretariat to allow such information to be distributed to the code development committee. Where such additional copies are requested, it shall be the responsibility of the proponent to send such copies to the respective code development committee. A copy of the code change proposal in electronic form is preferred.

3.5 **Submittal Deadline:** Each code change proposal shall be received at the office of the Secretariat by the posted deadline. Such posting shall occur no later than 120 days prior to the code change deadline. The submitter of a proposed code change is responsible for the proper and timely receipt of all pertinent materials by the Secretariat.

3.6 **Referenced Standards:** In order for a standard to be considered for reference or to continue to be referenced by the Codes, a standard shall meet the following criteria:

3.6.1 Code References:

3.6.1.1 The standard, including title and date, and the manner in which it is to be utilized shall be specifically referenced in the Code text.
3.6.1.2 The need for the standard to be referenced shall be established.

3.6.2 Standard Content:

3.6.2.1 A standard or portions of a standard intended to be enforced shall be written in mandatory language.
3.6.2.2 The standard shall be appropriate for the subject covered.
3.6.2.3 All terms shall be defined when they deviate from an ordinarily accepted meaning or a dictionary definition.
3.6.2.4 The scope or application of a standard shall be clearly described.
3.6.2.5 The standard shall not have the effect of requiring proprietary materials.
3.6.2.6 The standard shall not prescribe a proprietary agency for quality control or testing.
3.6.2.7 The test standard shall describe, in detail, preparation of the test sample, sample selection or both.
3.6.2.8 The test standard shall prescribe the reporting format for the test results. The format shall identify the key performance criteria for the element(s) tested.
3.6.2.9 The measure of performance for which the test is conducted shall be clearly defined in either the test standard or in Code text.
3.6.2.10 The standard shall not state that its provisions shall govern whenever the referenced standard is in conflict with the requirements of the referencing Code.
3.6.2.11 The preface to the standard shall announce that the standard is promulgated according to a consensus procedure.

3.6.3 Standard Promulgation:

3.6.3.1 Code change proposals with corresponding changes to the code text which include a reference to a proposed new standard or a proposed update of an existing referenced shall comply with this section. The standard shall be completed and readily available prior to Final Action Consideration based on the cycle of code development which includes the proposed code change proposal. In order for a new standard to be considered for reference by the Code, such standard shall be submitted in at least a consensus draft form in accordance with Section 3.4. Updating of standards without corresponding code text changes shall be accomplished administratively in accordance with Section 4.5.
3.6.3.2 The standard shall be developed and maintained through a consensus process such as ASTM or ANSI.

4.0 Processing of Proposals

4.1 Intent: The processing of code change proposals is intended to ensure that each proposal complies with these Rules of Procedure and that the resulting published proposal accurately reflects that proponent's intent.

4.2 Review: Upon receipt in the Secretariat's office, the code change proposals will be checked for compliance with these Rules of Procedure as to division, separation, number of copies, form, language, terminology, supporting statements and substantiating data. Where a code change proposal consists of multiple parts which fall under the maintenance responsibilities of different code committees, the Secretariat shall determine the code committee responsible for determining the committee action in accordance with Section 5.6.

4.3 Incomplete Proposals: When a code change proposal is submitted with incorrect format, without the required information or judged as not in compliance with these Rules of Procedure, the Secretariat shall notify the proponent of the specific deficiencies and the proposal shall be held until the deficiencies are corrected, with a final date set for receipt of a corrected submittal. If the Secretariat receives the corrected proposal after the final date, the proposal shall be held over until the next code development cycle. Where there are otherwise no deficiencies addressed by this section, a proposal that incorporates a new referenced standard shall be processed with an analysis of referenced standard's compliance with the criteria set forth in Section 3.6.

4.4 Editorial: The Chief Executive Officer shall have the authority at all times to make editorial and format changes to the Code text, or any approved changes, consistent with the intent, provisions and style of the Code. An editorial or format change is a text change that does not affect the scope or application of the code requirements.

4.5 Updating Standards:

4.5.1 Standards referenced in the 2012 Edition of the I-Codes: The updating of standards referenced by the Codes shall be accomplished administratively by the Administrative code development committee in accordance with these full procedures except that the deadline for availability of the updated standard and receipt by the Secretariat shall be December 1, 2011. The published version of the 2012 Code which references the standard will refer to the updated edition of the standard. If the standard is not available by the deadline, the edition of the standard as referenced by the newly published Code shall revert back to the reference contained in the previous edition and an errata to the Code issued Multiple standards to be updated may be included in a single proposal.

4.5.2 Standards referenced in the 2015 Edition and following Editions of the I-Codes: The updating of standards referenced by the Codes shall be accomplished administratively by the Administrative code development committee in accordance with these full procedures except that multiple standards to be updated may be included in a single proposal. The standard shall be completed and readily available prior to Final Action Consideration of the Administrative code change proposal which includes the proposed update.

4.6 Preparation: All code change proposals in compliance with these procedures shall be prepared in a standard manner by the Secretariat and be assigned separate, distinct and consecutive numbers. The Secretariat shall coordinate related proposals submitted in accordance with Section 3.3.2 to facilitate the hearing process.

4.7 Publication: All code change proposals shall be posted on the ICC website at least 30 days prior to the public hearing on those proposals and shall constitute the agenda for the public hearing. Code change proposals which have not been published shall not be considered.

5.0 Public Hearing

5.1 Intent: The intent of the public hearing is to permit interested parties to present their views including the cost and benefits on the code change proposals on the published agenda. The code development committee will consider such comments as may be presented in the development of their action on the disposition of such proposals. At the conclusion of the code development committee deliberations, the committee action on each code change proposal shall be placed before the hearing assembly for consideration in accordance with Section 5.7.

5.2 Committee: The Code Development Committees shall be appointed by the applicable ICC Council.

5.2.1 Chairman/Moderator: The Chairman and Vice-Chairman shall be appointed by the Steering Committee on Councils from the appointed members of the committee. The ICC President shall appoint one or more Moderators who shall act as presiding officer for the public hearing.

5.2.2 Conflict of Interest: A committee member shall withdraw from and take no part in those matters with which the committee member has an undisclosed financial, business or property interest. The committee member shall not participate in any committee discussion on the matter or any committee vote. Violation thereof shall result in the immediate removal of the committee member from the committee. A committee member who is a proponent of a proposal shall not participate in any committee discussion on the matter or any committee vote. Such committee member shall be permitted to participate in the floor discussion in accordance with Section 5.5 by stepping down from the dais.

5.2.3 Representation of Interest: Committee members shall not represent themselves as official or unofficial representatives of the ICC except at regularly convened meetings of the committee.

5.2.4 Committee Composition: The committee may consist of representation from multiple interests. A minimum of thirty-three and one-third percent (33.3%) of the committee members shall be regulators.

5.3 Date and Location: The date and location of each public hearing shall be announced not less than 60 days prior to the date of the public hearing.

5.4 General Procedures: *The Robert's Rules of Order* shall be the formal procedure for the conduct of the public hearing except as a specific provision of these Rules of Procedure may otherwise dictate. A quorum shall consist of a majority of the voting members of the committee.

5.4.1 Chair Voting: The Chairman of the committee shall vote only when the vote cast will break a tie vote of the committee.

5.4.2 Open Meetings: Public hearings of the Code Development Committees are open meetings. ` Any interested person may attend and participate in the Floor Discussion and Assembly Consideration portions of the hearing. Only eligible voters (see Section 5.7.4) are permitted to vote on Assembly Considerations. Only Code Development Committee members may participate in the Committee Action portion of the hearings (see Section 5.6).

5.4.3 Presentation of Material at the Public Hearing: Information to be provided at the hearing shall be limited to verbal presentations and modifications submitted in accordance with Section 5.5.2. Audio-visual presentations are not permitted. Substantiating material submitted in accordance with Section 3.3.4.4 and other material submitted in response to a code change proposal shall be located in a designated area in the hearing room and shall not be distributed to the code development committee at the public hearing.

5.4.4 Agenda Order: The Secretariat shall publish an agenda for each public hearing, placing individual code change proposals in a logical order to facilitate the hearing. Any public hearing attendee may move to revise the agenda order as the first order of business at the public hearing, or at any time during the hearing except while another proposal is being discussed. Preference shall be given to grouping like subjects together, and for moving items back to a later position on the agenda as opposed to moving items forward to an earlier position. A motion to revise the agenda order is subject to a 2/3 vote of those present and voting.

5.4.5 Reconsideration: There shall be no reconsideration of a proposed code change after it has been voted on by the committee in accordance with Section 5.6; or, in the case of assembly consideration, there shall be no reconsideration of a proposed code change after it has been voted on by the assembly in accordance with Section 5.7.

5.4.6 Time Limits: Time limits shall be established as part of the agenda for testimony on all proposed changes at the beginning of each hearing session. Each person requesting to testify on a change shall be given equal time. In the interest of time and fairness to all hearing participants, the Moderator shall have limited authority to modify time limitations on debate. The Moderator shall have the authority to adjust time limits as necessary in order to complete the hearing agenda.

5.4.6.1 Time Keeping: Keeping of time for testimony by an individual shall be by an automatic timing device. Remaining time shall be evident to the person testifying. Interruptions during testimony shall not be tolerated. The Moderator shall maintain appropriate decorum during all testimony.

5.4.6.2 Proponent Testimony: The Proponent is permitted to waive an initial statement. The Proponent shall be permitted to have the amount of time that would have been allocated during the initial testimony period plus the amount of time that would be allocated for rebuttal. Where the code change proposal is submitted by multiple proponents, this provision shall permit only one proponent of the joint submittal to be allotted additional time for rebuttal.

5.4.7 Points of Order: Any person participating in the public hearing may challenge a procedural ruling of the Moderator or the Chairman. A majority vote of the eligible voters as determined in Section 5.7.4 shall determine the decision.

5.5 Floor Discussion: The Moderator shall place each code change proposal before the hearing for discussion by identifying the proposal and by regulating discussion as follows:

5.5.1 Discussion Order:

1. *Proponents.* The Moderator shall begin by asking the proponent and then others in support of the proposal for their comments.
2. *Opponents.* After discussion by those in support of a proposal, those opposed hereto, if any, shall have the opportunity to present their views.
3. *Rebuttal in support.* Proponents shall then have the opportunity to rebut points raised by the opponents.
4. *Rerebuttal in opposition.* Opponents shall then have the opportunity to respond to the proponent's rebuttal.

5.5.2 Modifications: Modifications to proposals may be suggested from the floor by any person participating in the public hearing. The person proposing the modification is deemed to be the proponent of the modification.

5.5.2.1 Submission and Written Copies. All modifications must be written, unless determined by the Chairman to be either editorial or minor in nature. The modification proponent shall provide 20 copies to the Secretariat for distribution to the committee.

5.5.2.2 Criteria. The Chairman shall rule proposed modifications in or out of order before they are discussed on the floor. A proposed modification shall be ruled out of order if it:

1. is not legible, unless not required to be written in accordance with Section 5.5.2.1; or
2. changes the scope of the original proposal; or
3. is not readily understood to allow a proper assessment of its impact on the original proposal or the code.

The ruling of the Chairman on whether or not the modification is in or out of order shall be final and is not subject to a point of order in accordance with Section 5.4.7.

5.5.2.3 Testimony. When a modification is offered from the floor and ruled in order by the Chairman, a specific floor discussion on that modification is to commence in accordance with the procedures listed in Section 5.5.1.

5.6 Committee Action: Following the floor discussion of each code change proposal, one of the following motions shall be made and seconded by members of the committee.

1. Approve the code change proposal as submitted (AS) or
2. Approve the code change proposal as modified with specific modifications (AM), or
3. Disapprove the code change proposal (D)

Discussion on this motion shall be limited to Code Development Committee members. If a committee member proposes a modification which had not been proposed during floor discussion, the Chairman shall rule on the modification in accordance with Section 5.5.2.2 If a committee member raises a matter of issue, including a proposed modification, which has not been proposed or discussed during the floor discussion, the Moderator shall suspend the committee discussion and shall reopen the floor discussion for comments on the specific matter or issue. Upon receipt of all comments from the floor, the Moderator shall resume committee discussion.

The Code Development Committee shall vote on each motion with the majority dictating the committee's action. Committee action on each code change proposal shall be completed when one of the motions noted above has been approved. Each committee vote shall be supported by a reason.

The Code Development Committee shall maintain a record of its proceedings including the action on each code change proposal.

5.7 Assembly Consideration: At the conclusion of the committee's action on a code change proposal and before the next code change proposal is called to the floor, the Moderator shall ask for a motion from the public hearing attendees who may object to the committee's action. If a motion in accordance with Section 5.7.1 is not brought forward on the committee's action, the results of the public hearing shall be established by the committee's action. If a motion in accordance with Section 5.7.1 is brought forward and

is sustained in accordance with Section 5.7.3, both the committee's action and the assemblies' action shall be reported as the results of the public hearing. Where a motion is sustained in accordance with Section 5.7.3, such action shall be the initial motion considered at Final Action Consideration in accordance with Section 7.3.8.2.

5.7.1 Floor Motion: Any attendee may raise an objection to the committee's action in which case the attendee will be able to make a motion to:

1. Approve the code change proposal as submitted from the floor (ASF), or
2. Approve the code change proposal as modified from the floor (AMF) with a specific modification that has been previously offered from the floor and ruled in order by the Chairman during floor discussion (see Section 5.5.2) or has been offered by a member of the Committee and ruled in order by the Chairman during committee discussion (see Section 5.6), or
3. Disapprove the code change proposal from the floor (DF).

5.7.2 Discussion: On receipt of a second to the floor motion, the Moderator shall place the motion before the assembly for a vote. No additional testimony shall be permitted.

5.7.3 Assembly Action: The assembly action shall be in accordance with the following majorities based on the number of votes cast by eligible voters (See 5.7.4).

Committee Action	Desired Assembly Action		
	ASF	AMF	DF
AS	--	2/3 Majority	2/3 Majority
AM	2/3 Majority	2/3 Majority	2/3 Majority
D	2/3 Majority	2/3 Majority	--

5.7.4 Eligible Voters: All members of ICC in attendance at the public hearing shall be eligible to vote on floor motions. Only one vote authorized for each eligible attendee. Code Development Committee members shall be eligible to vote on floor motions. Application, whether new or updated, for ICC membership must be received by the Code Council ten days prior to the commencement of the first day of the public hearing.

5.8 Report of the Public Hearing: The results of the public hearing, including committee action and successful assembly action, shall be posted on the ICC website not less than 60 days prior to Final Action Consideration except as approved by the ICC Board.

6.0 Public Comments

6.1 Intent: The public comment process gives attendees at the Final Action Hearing an opportunity to consider specific objections to the results of the public hearing and more thoughtfully prepare for the discussion for Final Action Consideration. The public comment process expedites the Final Action Consideration at the Final Action Hearing by limiting the items discussed to the following:

6.1.1 Consideration of items for which a public comment has been submitted; and
6.1.2 Consideration of items which received a successful assembly action at the public hearing.

6.2 Deadline: The deadline for receipt of a public comment to the results of the public hearing shall be announced at the public hearing but shall not be less than 30 days from the availability of the report of the results of the public hearing (see Section 5.8).

6.3 Withdrawal of Public Comment: A public comment may be withdrawn by the public commenter at any time prior to Final Action Consideration of that comment. A withdrawn public comment shall not be subject to Final Action Consideration. If the only public comment to a code change proposal is withdrawn by the public commenter prior to the vote on the consent agenda in accordance with Section 7.3.4, the proposal shall be considered as part of the consent agenda. If the only public comment to a code change proposal is withdrawn by the public commenter after the vote on the consent agenda in accordance with Section 7.3.4, the proposal shall continue as part of the individual consent agenda in accordance with Section 7.3.5, however the public comment shall not be subject to Final Action Consideration.

6.4 Form and Content of Public Comments: Any interested person, persons, or group may submit a public comment to the results of the public hearing which will be considered when in conformance to these requirements. Each public comment to a code change proposal shall be submitted separately and shall be complete in itself. Each public comment shall contain the following information:

6.4.1 Public comment: Each public comment shall include the name, title, mailing address, telephone number and email address of the public commenter. If group, organization, or committee submits a public comment, an individual with prime responsibility shall be indicated. If a public comment is submitted on behalf a client, group, organization or committee, the name and mailing address of the client, group, organization or committee shall be indicated. The scope of the public comment shall be consistent with the scope of the original code change proposal, committee action or successful assembly action. Public comments which are determined as not within the scope of the code change proposal, committee action or successful assembly action shall be identified as such. The public commenter shall be notified that the public comment is considered an incomplete public comment in accordance with Section 6.5.1 and the public comment shall be held until the deficiencies are corrected. A copyright release in accordance with Section 3.3.4.5 shall be provided with the public comment.

6.4.2 Code Reference: Each public comment shall include the code change proposal number and the results of the public hearing, including successful assembly actions, on the code change proposal to which the public comment is directed.

6.4.3 Multiple public comments to a code change proposal. A proponent shall not submit multiple public comments to the same code change proposal. When a proponent submits multiple public comments to the same code change proposal, the public comments shall be considered as incomplete public comments and processed in accordance with Section 6.5.1. This restriction shall not apply to public comments that attempt to address differing subject matter within a code section.

6.4.4 Desired Final Action: The public comment shall indicate the desired final action as one of the following:

1. Approve the code change proposal as submitted (AS), or
2. Approve the code change proposal as modified (AM) by one or more specific modifications published in the Results of the Public Hearing or published in a public comment, or
3. Disapprove the code change proposal (D)

6.4.5 Supporting Information: The public comment shall include in a statement containing a reason and justification for the desired final action on the code change proposal. Reasons and justification which are reviewed in accordance with Section 6.4 and determined as not germane to the technical issues addressed in the code change proposal or committee action shall be identified as such. The public commenter shall be notified that the public comment is considered an incomplete public comment in accordance with Section 6.5.1 and the public comment shall be held until the deficiencies are corrected. The public commenter shall have the right to appeal this action in accordance with the policy of the ICC Board. A bibliography of any substantiating material submitted with a public comment shall be published with the public comment and the substantiating material shall be made available at the Final Action Hearing.

6.4.6 Number: One copy of each public comment and one copy of all substantiating information shall be submitted. Additional copies may be requested when determined necessary by the Secretariat. A copy of the public comment in electronic form is preferred.

6.5 Review: The Secretariat shall be responsible for reviewing all submitted public comments from an editorial and technical viewpoint similar to the review of code change proposals (See Section 4.2).

6.5.1 Incomplete Public Comment: When a public comment is submitted with incorrect format, without the required information or judged as not in compliance with these Rules of Procedure, the public comment shall not be processed. The Secretariat shall notify the public commenter of the specific deficiencies and the public comment shall be held until the deficiencies are corrected, or the public comment shall be returned to the public commenter with instructions to correct the deficiencies with a final date set for receipt of the corrected public comment.

6.5.2 Duplications: On receipt of duplicate or parallel public comments, the Secretariat may consolidate such public comments for Final Action Consideration. Each public commenter shall be notified of this action when it occurs.

6.5.3 Deadline: Public comments received by the Secretariat after the deadline set for receipt shall not be published and shall not be considered as part of the Final Action Consideration.

6.6 Publication: The public hearing results on code change proposals that have not been public commented and the code change proposals with public commented public hearing results and successful assembly actions shall constitute the Final Action Agenda. The Final Action Agenda shall be posted on the ICC website at least 30 days prior to Final Action consideration.

7.0 Final Action Consideration

7.1 Intent: The purpose of Final Action Consideration is to make a final determination of all code change proposals which have been considered in a code development cycle by a vote cast by eligible voters (see Section 7.4).

7.2 Agenda: The final action consent agenda shall be comprised of proposals which have neither an assembly action nor public comment. The agenda for public testimony and individual consideration shall be comprised of proposals which have a successful assembly action or public comment (see Sections 5.7 and 6.0).

7.3 Procedure: *The Robert's Rules of Order* shall be the formal procedure for the conduct of the Final Action Consideration except as these Rules of Procedure may otherwise dictate.

7.3.1 **Open Meetings:** Public hearings for Final Action Consideration are open meetings. Any interested person may attend and participate in the Floor Discussion.

7.3.2 **Agenda Order:** The Secretariat shall publish an agenda for Final Action Consideration, placing individual code change proposals and public comments in a logical order to facilitate the hearing. The proponents or opponents of any proposal or public comment may move to revise the agenda order as the first order of business at the public hearing, or at any time during the hearing except while another proposal is being discussed. Preference shall be given to grouping like subjects together and for moving items back to a later position on the agenda as opposed to moving items forward to an earlier position. A motion to revise the agenda order is subject to a 2/3 vote of those present and voting.

7.3.3 **Presentation of Material at the Public Hearing:** Information to be provided at the hearing shall be limited to verbal presentations. Audio-visual presentations are not permitted. Substantiating material submitted in accordance with Section 6.4.4 and other material submitted in response to a code change proposal or public comment shall be located in a designated area in the hearing room.

7.3.4 **Final Action Consent Agenda:** The final action consent agenda (see Section 7.2) shall be placed before the assembly with a single motion for final action in accordance with the results of the public hearing. When the motion has been seconded, the vote shall be taken with no testimony being allowed. A simple majority (50% plus one) based on the number of votes cast by eligible voters shall decide the motion.

7.3.5 **Individual Consideration Agenda:** Upon completion of the final action consent vote, all proposed changes not on the final action consent agenda shall be placed before the assembly for individual consideration of each item (see Section 7.2).

7.3.6 **Reconsideration:** There shall be no reconsideration of a proposed code change after it has been voted on in accordance with Section 7.3.8.

7.3.7 **Time Limits:** Time limits shall be established as part of the agenda for testimony on all proposed changes at the beginning of each hearing session. Each person requesting to testify on a change shall be given equal time. In the interest of time and fairness to all hearing participants, the Moderator shall have limited authority to modify time limitations on debate. The Moderator shall have the authority to adjust time limits as necessary in order to complete the hearing agenda.

7.3.7.1 **Time Keeping:** Keeping of time for testimony by an individual shall be by an automatic timing device. Remaining time shall be evident to the person testifying. Interruptions during testimony shall not be tolerated. The Moderator shall maintain appropriate decorum during all testimony.

7.3.8 **Discussion and Voting:** Discussion and voting on proposals being individually considered shall be in accordance with the following procedures:

7.3.8.1 **Allowable Final Action Motions:** The only allowable motions for final action are Approval as Submitted, Approval as Modified by one or more modifications published in the Final Action Agenda, and Disapproval.

7.3.8.2 **Initial Motion:** The Code Development Committee action shall be the initial motion considered, unless there was a successful assembly action in accordance with Section 5.7.3. If there was a successful assembly action, it shall be the initial motion considered. If the assembly action motion fails, the code development committee action shall become the next motion considered.

7.3.8.3 **Motions for Modifications:** Whenever a motion under consideration is for Approval as Submitted or Approval as Modified, a subsequent motion and second for a modification published in the Final Action Agenda may be made (see Section 6.4.3). Each subsequent motion for modification, if any, shall be individually discussed and voted before returning to the main motion. A two-thirds majority based on the number of votes cast by eligible voters shall be required for a successful motion on all modifications.

7.3.8.4 **Voting:** After dispensing with all motions for modifications, if any, and upon completion of discussion on the main motion, the Moderator shall then ask for the vote on the main motion. If the motion fails to receive the majority required in Section 7.5, the Moderator shall ask for a new motion.

7.3.8.5 **Subsequent Motion:** If the initial motion is unsuccessful, a motion for one of the other allowable final actions shall be made (see Section 7.3.8.1) and dispensed with until a successful final action is achieved. If a successful final action is not achieved, Section 7.5.1 shall apply.

7.3.9 **Proponent testimony:** The Proponent of a public comment is permitted to waive an initial statement. The Proponent of the public comment shall be permitted to have the amount of time that would have been allocated during the initial testimony period plus the amount of time that would be allocated for rebuttal. Where a public comment is submitted by multiple proponents, this provision shall permit only one proponent of the joint submittal to waive an initial statement.

7.3.10 **Points of Order:** Any person participating in the public hearing may challenge a procedural ruling of the Moderator. A majority vote of the eligible voters as determined in Section 5.7.4 shall determine the decision.

7.4 **Eligible voters:** ICC Governmental Member Representatives and Honorary Members in attendance at the Final Action Hearing shall have one vote per eligible attendee on all International Codes. Applications, whether new or updated, for governmental member voting representative status must be received by the Code Council ten days prior to the commencement of the first day of the Final Action Hearing in order for any designated representative to be eligible to vote.

7.5 **Majorities for Final Action:** The required voting majority based on the number of votes cast of eligible voters shall be in accordance with the following table:

Public Hearing Action (see note)	**Desired Final Action**		
	AS	**AM**	**D**
AS	Simple Majority	2/3 Majority	Simple Majority
AM	2/3 Majority	Simple Majority to sustain the Public Hearing Action or; 2/3 Majority on additional modifications and 2/3 on overall AM	Simple Majority
D	2/3 Majority	2/3 Majority	Simple Majority

Note: The Public Hearing Action includes the committee action and successful assembly action.

7.5.1 **Failure to Achieve Majority Vote:** In the event that a code change proposal does not receive any of the required majorities for final action in Section 7.5, final action on the code change proposal in question shall be disapproval.

7.6 **Publication:** The Final action on all proposed code changes shall be published as soon as practicable after the determination of final action. The exact wording of any resulting text modifications shall be made available to any interested party.

8.0 Appeals

8.1 **Right to Appeal:** Any person may appeal an action or inaction in accordance with CP-1.

2009/2010 ICC CODE DEVELOPMENT CYCLE
CROSS INDEX OF PROPOSED CODE CHANGES

Some of the proposed code changes include sections that are outside of the scope of the chapters or the code listed in the table of 2009/2010 Staff Secretaries on page ix. This is done in order to facilitate coordination among the International Codes which is one of the fundamental principles of the International Codes.

Listed in this cross index are proposed code changes that include sections of codes or codes other than those listed on page ix. For example, IBC Section 402.16.5 is proposed for revision in Part II of code change F58-09/10, which is to be heard by the IFC Committee. This section of the IBC is typically the responsibility of the IBC General Committee as listed in the table of 2009/2010 Staff Secretaries. It is therefore identified in this cross index. Another example is Section 905.4 of the International Fire Code. The International Fire Code is normally maintained by the IFC Committee, but Section 905.4 will be considered for revision in proposed code change G31-09/10 and will be placed on the IBC General Committee agenda. In some instances, there are other subsections that are revised by an identified code change that is not included in the cross index. For example, numerous sections in Chapter 10 of the International Fire Code would be revised by the proposed changes to Chapter 10 of the IBC. This was done to keep the cross index brief enough for easy reference.

This information is provided to assist users in locating all of the proposed code changes that would affect a certain section or chapter. For example, to find all of the proposed code changes that would affect Chapter 7 of the IBC, review the proposed code changes in the Volume 1 monograph for the IBC Fire Safety Committee (listed with a FS prefix) then review this cross reference for Chapter 7 of the IBC for proposed code changes published in other code change groups. While care has been taken to be accurate, there may be some omissions in this list.

Letter prefix: Each proposed change number has a letter prefix that will identify where the proposal is published. The letter designations for proposed changes and the corresponding publications are as follows:

PREFIX	**PROPOSED CHANGE GROUP (see monograph table of contents for location)**
ADM	Administrative
E	International Building Code - Means of Egress
EB	International Existing Building Code
EC	International Energy Conservation Code
F	International Fire Code
FG	International Fuel Gas Code
FS	International Building Code - Fire Safety
G	International Building Code - General
M	International Mechanical Code
PC	ICC Performance Code
P	International Plumbing Code
PSD	International Private Sewage Disposal Code
PM	International Property Maintenance Code
RB	International Residential Code - Building
RE	International Residential Code - Energy
RM	International Residential Code - Mechanical
RP	International Residential Code - Plumbing
S	International Building Code - Structural
WUIC	International Wildland-Urban Interface Code
Z	International Zoning Code

INTERNATIONAL BUILDING CODE	
Chapter 1	ADM1 Part I
101.2	ADM2
101.3	ADM3
102.4	ADM4 Part I
104.10.1(New)	ADM5
105.2	ADM6 Part I
105.2.4	ADM7 Part I
106.1	S55-09/10
107.2	ADM9 Part I
107.2.2	ADM10
107.2.3	ADM11
107.2.6	ADM12
108.1	ADM13
109.3.10.1	ADM14 Part I
110.3	ADM8 Part II
110.3.6	ADM23 Part I (Heard by IBC-FS Committee)
110.6	ADM15 Part II
113.2.1	ADM5
117 (New)	ADM16 Part I
R202	EB3 , EB4
202	G2 Part I– Heard by Structural
303.1	E140, E141
Table 307.1(1)	F186, F187
307.2	F186, F190, F196
307.4	F187
402.11	F58, Part II
402.12.1	F58, Part II
402.16.5	F58, Part II
403.2.3	E5 – Part I
403.2.3.1	E5 – Part I
403.2.3.2	E5 – Part I
403.3.1.1 (IFC 914.3.1.1.1)	E5 – Part II
403.5.1	E5 – Part I
403.5.4	E5 – Part I
406.2.2	E151 Part I
406.6.6.1	F178
406.6.6.1.1 (New)	F178
408.3.8	E5 – Part I
410.5.3.1 (New)	E5 – Part I
414.2.1	F189
414.2.2	F190
Table 414.2.2	F189
414.2.4	F189
414.7.2 (IFC 2705.4.4)	E5 – Part II
Table 415.8.2.1.1	F165
415.8.2.6	F162, Part I
415.8.3	F167
415.8.4.6.2 (IFC 1803.12.1.2)	E5 – Part II
415.8.11.2	F161
416	F155
501.2	F18
505.3	E6
505.4	E6
705.2	E5 – Part I, E137
705.11	G81
707.3.2	E5 – Part I
707.3.3 (New)	E5 – Part I
707.3.10 new	G81
707.3.10 (New)	E132
707.4	E5 – Part I
707.5.1	G178
707.5.1	E5 – Part I
707.6	E5 – Part I
707.7.1	E5 – Part I
708.1	E5 – Part I
708.2	E5 – Part I
708.3 (New)	E5 – Part I
708.6	E5 – Part I
708.14.1	G44 Part I
709.1	G82
709.3	G81
709.4	G81
709.5	E5 – Part I
710.5	G15, G21
712.4	G178
717.3.2	G81
717.4.2	G81, G82
712.4	E5 – Part I
Table 715.4	E5 – Part I
715.4.4	E5 – Part I
715.4.6.1	E5 – Part I
715.4.7.2	E5 – Part I
716.5.1	F162, Part II
716.5.2	E5 – Part I
Table 803.9	E5 – Part I
804.4	E5 – Part I
804.4.1	E5 – Part I
806.1	F57
901.2.1	F62
901.6.3	F193, Part II
907.5.2.3.4	E151 Part II
909.5 (IFC 909.5, IMC 513.5)	E5 – Part II
911.1.2	F22
911.1.5	F23, F24, F25
911.1.5	G44 Part II
1006.1	G21
1007.1	EB10
1008.1.9.6	G65
1009.7	G67 – Heard by MOE
1013.1 (New)	FS154
1013.8 (New)	FS154
1015.1 and Table 1015.1	G16
1015.6	G67 – Heard by MOE
1015.6.1	G67 – Heard by MOE
1015.7 new	G16
1021.2	G16, G20
Table 1021.2	G20
1022.1	G52
1022.1	G67 – Heard by MOE

IBC (continued)	
1103.2.12	G16
1106.3	G65
1106.4	G65
1107.5.2	G22, G23
1107.5.3	G20
1107.6.4.1	G21
1110.3	G177 – Heard by MOE
Table 1604.5	G65
1703.5.4 (New)	FS101
1704.16	F135
1704.16.1	F135
1704.16.2	F135
2303.3 (New)	FS133
2606.7	E5 – Part I
2702 (New)	F31
2702.2.21 (New)	F33
2902.4	E151 Part III
2902.1	P20
Table 2902.1	P21, P22, P23, P24
2902.2	P25
2902.2.1 (New)	P26
2902.3	P27
2902.4.1	P28
2902.3.5 (New)	P30
2902.5 (New)	P31
2902.3.2	P35
2902.1.3	P46
2902.3	P162
3001.3	E151 Part I
3003.3 (New)	F20, Part II
3007.4.1	E5 – Part I
3007.5	E5 – Part I
3008.11.1	E5 – Part I
3108.1	S85-09/10
3403.5 (New)	F114, Part II
3404.6	E20 Part I, E21 Part I
3404.7 (New)	F114, Part II
3405.2	EB6
3405.2.1	EB6
3405.3.1	EB8 , EB9
3408.4	S41-09/10
3411.1	EB11 , EB14
3411.1	E156 Part I
3411.4	EB10 , EB11 , EB14 , EB32
3411.4.1	EB10 , EB32
3411.4.2	EB10 , EB14 , EB32 , EB33
3411.5	EB10 , EB11
3411.6	EB10 , EB11 , EB14
3411.6	E151 Part I
3411.7	EB14
3411.7 (New)	EB11
3411.8	EB11
3411.8	E 152 Part I
3411.8 (New)	EB10
3411.8.1(New)	EB10
3411.8.5	EB11
3411.8.8	EB14
3411.8.9	EB11 , EB14
3411.8.15 (New)	E 152 Part I
3411.8.15.1 (New)	E 152 Part I
3411.8.15.2 (New)	E 152 Part I
3411.8.16 (New)	E 152 Part I
3411.9	EB10 , EB14
3412.2.5	EB14
3412.6.11	E20 Part I, E21 Part I
Table 3412.6.11(1)	E20 Part I, E21 Part I
Chapter 35	F58, Part II
Chapter 35	ADM39
K101.3	ADM3
Appendix I	G2 – Heard by IBC -S
Appendix L (New)	F236, Part II
Appendix L (New)	S108-09/10

INTERNATIONALENERGY CONSERVATION CODE	
Chapter 1	ADM1 Part III
101.3	ADM3, ADM24 (Heard by IECC Committee)
101.4.4	ADM27 (Heard by IECC Committee)
101.4.6	ADM25 (Heard by IECC Committee)
101.5.1	ADM17
101.5.2	ADM26 (Heard by IECC Committee)
102.1.1	ADM28 (Heard by IECC Committee) ADM31 (Heard by IECC Committee)
102.1.2	ADM29 (Heard by IECC Committee)
103.1	ADM17
103.2	ADM9 Part I
104.2	ADM15 Part I
106.1	ADM4
110	ADM16 Part I
Ch. 6	ADM39

INTERNATIONAL EXISTING BUILDING CODE	
Chapter 1	ADM1 Part II
101.2	ADM2
101.3	ADM3
Table 101.5.4.1	S41-09/10
101.5.4.2	S41-09/10
Table 101.5.4.2	S41-09/10
101.5.4.2	ADM32 (Heard by IBC-S Committee) ADM33 (Heard by IBC-S Committee)
102.4	ADM4
106.2.1	ADM9 Part I
107.1	ADM13
109.3.6(New)	ADM23
109.6	ADM15 Part I
118	ADM16 Part I

IEBC (continued)	
202	S91-09/10
301.1	G187
301.1.1	G188
301.2	G190 – Heard by IBC –S
301.2.1	G191 – Heard by IBC-S
301.2.3	G190 – Heard by IBC – S
301.3	G192 – Heard by IBC-S
302.4.1	G190 – Heard by IBC – S
302.5 (New)	F114, Part III
303.4.1	G190 – Heard by IBC-S
303.6	E20 Part I, E21 Part I
303.7 (New)	F114, Part III
304.1.1	G192 – Heard by IBC-S
304.2	G193, G194 – Both heard by IBC-S
304.2.1	G190, G193, G194 – All heard by IBC - S
304.2.2	G193, G194 – Both heard by IBC-S
304.2.3	G193, G194, G195 – All heard by IBC-S
304.3	G193 – Heard by IBC-S
304.3.1	G193 – Heard by IBC-S
304.3.2	G193 – Heard by IBC-S
304.4	G193, G194 – Both heard by IBC-S
304.5	G193, G196 – Both heard by IBC-S
307.4	G190, G197 – Both heard by IBC-S
307.4	S41-09/10
309.1	G198, G199
310.1	E156 Part I
310.6	E151 Part I
310.8	E152
310.8.8	G200 – Heard by MOE
310.8.15 (New)	E152
310.8.15.1 (New)	E152
310.8.15.2 (New)	E152
310.8.16 (New)	E152
605.1	E 151 Part IV, E152 Part II, E156 Part II
605.1.15 (New)	E152 Part II
605.1.15.1 (New)	E152 Part II
605.1.15.2 (New)	E152 Part II
605.1.16 (New)	E152 Part II
704.4.3	F114, Part III
907.3.1	S41-09/10
907.3.2	S41-09/10
1004.1	F114, Part III
1202.2	S146-09/10
1202.2.1	S146-09/10
1301.6.2.1	G201
1301.6.14	G202
1301.6.14.1	G202
1301.6.19	G203
1301.6.11	E20 Part I, E21 Part I
Table 1301.6.11(1)	E20 Part I, E21 Part I

1401.3.1 new	G184
1401.5 new	G185
Chapter 15	ADM39
A102.2	S41-09/10
INTERNATIONAL FIRE CODE	
Chapter 1	ADM1 Part IV
101.2	ADM19
101.3	ADM3
102.5	ADM20
102.7	ADM4
105.1.1	ADM21
105.4.2	ADM9 Part I
105.4.2.1	ADM10
105.4.3	ADM9 Part I
105.6.2	ADM34 (Heard by IFC Committee)
107.2.1	ADM35 (Heard by IFC Committee)
114 (New)	ADM16 Part I
202	
Def of Group A	G10, G11, G12, G13, G14
Def of Group B	G6, G15
Def of Group E	G16
Def of Group F	G18, G19
Def of Group I	G16, G20, G21, G22, G23. G24
Def of Group M	G25
Def of Group R	G20, G21, G22, G23, G26, G27, G28, G29
Def of Group S	G19
508.1.5 (IBC 911.1.5)	G44, Part II
603.4	M8 PII
607.4	G153, Part II
803.8	FS136 Part II
901.4.3 (New)	FS29
903.2.2	G15
903.2.3	G15
903.2.4.2 (new)	G19
903.2.6	G16, G20, G21
903.2.6.1	G21
903.2.8	G20
903.2.9.1	G19
903.3.1.3	G20
903.3.2	G20
904.5.2.3.3	G21
905.3.3.	G31
905.4	G31
907.2.2	G15
907.2.2.1	G15
907.2.6	G20
907.2.6.2	G20
907.5.2.3.4	E151 Part II
909.5 (IBC 909.5, IMC 513.5)	E5 – Part II
914.3.1.1.1 (IBC 403.3.1.1)	E5 – Part II

IFC (continued)	
914.6.1	G70 – Heard by IFC
914.8.2.2	G71 – Heard by IFC
Chapter 10 See IBC MOE changes	
1007.1	EB10
1030.4.1	E93 Part II
1404.5	G185 Part II
IFC 1803.12.1.2 (IBC 415.8.4.6.2)	E5 – Part II
2303.2	G64
IFC 2705.4.4 (IBC 414.7.2)	E5 – Part II
3904.1.2	G73 Part II – Heard by IFC
4604.7	E20 Part II, E21 Part II
Table 4604.7	E20 Part II, E21 Part II
Chapter 47	ADM39
INTERNATIONAL FUEL GAS CODE	
Chapter 1	ADM1 Part V
101.4	ADM3
102.8	ADM4
107.2	ADM8 Part I
111 (New)	ADM16 Part I
301.11	S92-09/10, Part III
306.5	M11, M12
306.5.1	M13
410.4 (New)	F148, Part II
Chapter 8	ADM39
INTERNATIONAL MECHANICAL CODE	
Chapter 1	ADM1 Part VI
102.8	ADM4
102.3	ADM36 (Heard by IMC Committee)
102.4	ADM37 (Heard by IMC Committee)
107.2	ADM8 Part I
202	FG14 PII
301.3 thru 301.5	FG14 PII
301.6	FG10 PI
301.13	S92-09/10, Part IV
307.3	FG11 PI
401.4	S92-09/10, Part IV
501.2.1	S92-09/10, Part IV
502.4	F43
502.5	F43
502.5.2	F43
502.8.4	F194
502.10.2	F162, Part I
510.7	F161, Part II
513.3	F135
513.10.2	F137
513.12	F138
513.12.1	F139
513.13.1	F140
602.4	S92-09/10, Part IV
603.13	S92-09/10, Part IV
606.2	F120

606.2.1 (New)	F120
606.2.2	F120
607.1	FS108, FS117
607.1.1	FS108, FS117
607.2	FS108, FS117
607.2.1	FS117
607.2.2	FS108, FS117
607.3.1	FS70, FS109
607.3.2.2	FS110
607.3.2.3	FS110
607.5	FS117
607.5.1	FS117
607.5.1	F162, Part II
607.5.1.1	FS117
607.5.2	FS117
607.5.2.1	FS117
607.5.3	FS114, FS117
607.5.4	FS117
607.5.5	FS111, FS112, FS113, FS117
607.5.6	FS117
607.5.7	FS117
607.6	FS117
607.6.1	FS115, FS117
607.6.2	FS117
607.6.2.1	FS117
607.6.3	FS116, FS117
607.7	FS117
918.6	FG32 PII
513.5 (IBC 909.5, IFC 909.5)	E5 – Part II
IMC 601.2 (IBC 1018.5, IFC 1018.5)	E116
1106.5	F39
1106.5.1	F39
1305.2.1	S92-09/10, Part IV
INTERNATIONAL PLUMBING CODE	
Chapter 1	ADM1 Part VII
101.3	ADM3
102.8	ADM4
107.2	ADM8 Part I
111 (New)	ADM16 Part I
202	FS124 Part II
309.2	S92-09/10, Part II
Table 403.1	G16, G20, G65
403.1	G16
403.2	G16
403.4	E151 Part III
1107.1	S2-09/10, Part I (Heard by IPC)
Chapter 13	ADM39
INT. PRIVATE SEWAGE DISPOSAL CODE	
Chapter 1	ADM1 Part IX
101.3	ADM3
102.10	ADM4
105.4	P1 Part II

IPSDC (continued)	
105.4.1	P1 Part II
105.4.2	P1 Part II
105.4.3	P1 Part II
105.4.4	P1 Part II
105.4.5	P1 Part II
105.4.6	P1 Part II
111 (New)	ADM 16 Part I
Section 304 (New)	P1 Part II
INTERNATIONAL PROPERTY MAINTENANCE CODE	
Chapter 1	ADM1 Part VIII
101.3	ADM3
102.3	ADM22
102.7	ADM4
108.1.3	ADM38 (Heard by IPMC Committee)
110.1	ADM38 (Heard by IPMC Committee)
113 (New)	ADM16 Part I
304.18.1	E60 Part II
606.1	G153 Part III
704.2	F114, Part I
704.3	F114, Part I
704.4	F114, Part II ; F115
Chapter 8	ADM39
INTERNATIONAL RESIDENTIAL CODE	***Note:*** All Code Change Parts for IRC are heard by the applicable IRC Committee except ADM39
Chapter 1	ADM 1 Part XII
R101.2	G28 Part II
R101.4	ADM3 Part II
R102.4	ADM4 Part II
R105.2	ADM6 Part II
R105.2.4 (New)	ADM7 Part II
R106.1.1	ADM9 Part II
R109.1.6.1	ADM14 Part II
R109.4	ADM15 Part II
110.3	ADM8 Part I
R115 (New)	ADM16 Part II
R202	FS124 Part III
R202	FG14 PIII
R202	M1 PII
R202	E8 Part II, E100 Part II, E156 Part III, E194 Part II
R202	F108, Part II; F132, Part II
R202	P2 Part II, P92 Part II, P128 Part II, P152 Part II
R202	G2 Part II, G5 Part II, G28 Part II
301.13	S92-09/10, Part IV
Figure R301.2(2)	S97-09/10, Part II
R301.2.1.1	S87-09/10, Part II
Table R301.5	S57-09/10, Part II, S61-09/10, Part II, S62-09/10, Part II, S66-09/10, Part II

R302.1	FS155 Part II
R302.1.2	FS155 Part II
R302.6	G56 Part II
R302.11.1	FS118 Part II
R308.3.1	S219-09/10, Part II
R308.4	S218-09/10, Part II
R308.4.1 (New)	S218-09/10, Part II
R308.4.2 (New)	S218-09/10, Part II
R308.4.3 (New)	S218-09/10, Part II
R308.4.4 (New)	S218-09/10, Part II
R308.4.5 (New)	S218-09/10, Part II
R308.4.6 (New)	S218-09/10, Part II
R308.4.7 (New)	S218-09/10, Part II
R308.6.1	S144-09/10, Part II
R310.1	E150 Part II
R311.2	E60 Part III
R311.2.1 (New)	E60 Part III
R311.3.1	E58 Part II
R311.4	E122 Part II
R311.7.4	E70 Part II, E71 Part II, E72 Part II
R311.7.4.1	E74 Part II
R311.7.4.2	E74 Part II
R311.7.4.3	E75 Part II
R311.7.4.3.1 (New)	E75 Part II
R311.7.4.3.2 (New)	E75 Part II
R311.7.4.3.3 (New)	E75 Part II
R311.7.7.3	E97 Part II
R312.2	E100 Part II
R314.1	F108, Part II; F112, Part II
R314.2	F108, Part II
R314.3	F108, Part II; F115, Part II
R314.4	F108, Part II; F115, Part II
R314.5	F115, Part II
R314.5 (New)	F116, Part II
R314.5.1 (New)	F116, Part II
R314.5.2 (New)	F116, Part II
R314.5.3 (New)	F116, Part II
R315	F132, Part II
R316.4	FS160 Part II
R316.5.3	FS168 Part II, FS169 Part II
R316.5.4	FS168 Part II, FS169 Part II
R316.5.13 (New)	FS171 Part II
R316.7	FS176 Part II
R316.8	FS176 Part II
R317.3	S203-09/10, Part II
R317.3.1	S203-09/10, Part II
R317.3.2	S203-09/10, Part II
R317.3.3	S203-09/10, Part II
R317.3.4	S203-09/10, Part II
R317.4.1 (New)	S207-09/10, Part II
R317.4.2	S207-09/10, Part II
R320.2 (New)	E156 Part III
R402.2	S162-09/10, Part II
R403.3.4	FS176 Part II
R404.1.2.3.6.1	FS176 Part II
R503.2.1	S200-09/10, Part II
R503.2.1.1	S200-09/10, Part II

IRC (continued)	
Table R601.3.1	FS147 Part II
R602.3	S200-09/10, Part II
R602.9	S214-09/10, Part II
R604.1	S199-09/10, Part II
R606.1	S171-09/10, Part II
R606.1.1	S171-09/10, Part II
R606.12.1	S171-09/10, Part II
R606.12.3.1	S171-09/10, Part II
R702.2.1	S222-09/10, Part II
R702.2.2	S222-09/10, Part II
R702.3.1	S222-09/10, Part II
R702.4.2	S224-09/10, Part II
R703.1.1	FS140 Part II
R703.3	FS156 Part II
R703.4	FS156 Part II
R703.4	S199-09/10, Part II
Table R703.4	FS156 Part II
R703.5.1	FS156 Part II
R703.6.1	FS156 Part II
R703.6.3	S225-09/10, Part II
R703.7.4.1	FS156 Part II
R703.11.2	FS156 Part II
R703.11.2.1	FS156 Part II
R703.11.2.2	FS156 Part II
R703.11.2.3	FS156 Part II
R703.12	FS150 Part II, FS151 Part II
703.12.1	FS150 Part II, FS151 Part II
R802.1.3	S201-09/10, Part II
R802.1.3.1	S201-09/10, Part II
R802.1.3.2	S201-09/10, Part II
R802.1.3.3	S201-09/10, Part II
R803.2.1	S200-09/10, Part II
R806.1	G146 Part II
R806.2	G145 Part II
R903.2.2	S3-09/10, Part II
R903.4	S2-09/10, Part III (heard by IRC Plumbing)
R903.4.1	S2-09/10, Part III (heard by IRC Plumbing)
Table R905.2.4.1(2)	S14-09/10, Part II
R905.2.7.2	S15-09/10, Part II
R905.2.8.5 (New)	S16-09/10, Part II
R905.3.3.3	S15-09/10, Part II
R905.4.3.2 (New)	S15-09/10, Part II
R905.4.5.1 (New)	S17-09/10, Part II
R905.5.3.2 (New)	S15-09/10, Part II
R905.6.3.2 (New)	S15-09/10, Part II
R905.7.3.2 (New)	S15-09/10, Part II
R905.8.3.2 (New)	S15-09/10, Part II
R905.9.2	S18-09/10, Part II
R905.10.5.1 (New)	S15-09/10, Part II
R905.14.3	S20-09/10, Part II
Table R905.14.3 (New)	S20-09/10, Part II
R905.15	S21-09/10, Part II
R905.15.1	S21-09/10, Part II
R905.15.2	S21-09/10, Part II

R905.15.3	S21-09/10, Part II
R905.16 (New)	S22-09/10, Part III, S23-09/10. Part, II
R905.16.1 (New)	S22-09/10, Part III, S23-09/10. Part II
R905.16.1.1 (New)	S23-09/10, Part II
R905.16.2 (New)	S22-09/10, Part III
R905.16.3 (New)	S22-09/10, Part III
R907.3	S30-09/10, Part II
R1003.9.1 (New)	S182-09/10, Part II
R1003.9.3 (New)	S182-09/10, Part II
R1003.11.1	M114 PII
R1005.7	M117 PII
R1004.2	M119 PII
T N1101.2	EC1 Part II
N1101.4.2.1(New)	EC2 Part II
N1101.6	EC4
Chapter 11	EC11 Part II, EC 13 Part II, EC16, Part II, EC19 Part II, EC25 Part II
N1101.2.2	EC21
N1101.7	EC28
N1101.9	EC22 Part II, EC23 Part II
N1102	EC26
N1102.1	EC31
N1103.2.1	EC26
Table N1102.1, Table N1102.1.2, Table1102.2.5	EC27, EC29, EC30, EC31, EC32, EC34, EC35, EC36, EC38, EC39, EC40, EC41, EC42, EC43, EC45, EC46, EC47, EC48, EC50, EC54, EC55, EC60, EC102 (All Part II)
Table N1102.1.4 (New) N1102.1.4(New)	EC56 Part II
N1102.2.2	EC59 Part II
N1102.2.2.1(New)	EC64 Part II
N1102.2.3 (New)	EC63 Part II
Table N1102.2.5	EC66 Part II
N1102.2.11	EC68 Part II
N1102.2.12(New)	EC69 Part II
Table N1102.4.2	EC26 Part II, EC59 Part II
Table N1102.1.4 (New) N1102.1.4(New)	EC57 Part II
N1102.3 (New)	EC71 Part II
N1102.3.3 (New)	EC72 Part II
N1102.3.3 (New)	EC73 Part II
N1102.3.3 (New)	EC74 Part II
N1102.3.4	EC76 Part II
N1102.3.5	EC68 Part II
N1102.3.6 (New)	EC96 Part II
N1102.3.7(New)	EC78 Part II
N1102.4.1	EC79, EC82, EC83
N1102.4.1.1(New)	EC79 Part II
N1102.4.1.2 (New)	EC79 Part II
N1102.4.2	EC81, EC82, EC83, EC86, EC90

IRC (continued)	
N1102.4.2.1.1 (New)	EC80 Part II
N1102.4.2.1.1 (New)	EC87 Part II
N1102.4.3	EC79 Part II, EC89 Part II
N1102.4.4	EC91 Part II
N1102.4.5	EC92 Part II
N1102.4.6	EC84
N1103.1	EC100 Part II
N1103.1.1	EC101 Part II
N1103.1.3 (New)	EC100 Part II
N1103.2.1	EC103 Part II
N1103.2.2	EC103, EC104, EC107 (All Part II)
N1103	
N1103.2.3	EC103 Part II, EC109 Part II,
N1103.3	EC117 Part II
N1103.3	EC123 Part II
N1103.4	EC115 Part II, EC116
N1103.4 (New)	EC114 Part II
N1103.4 (New)	EC118 Part II
N1103.4.1	EC112 Part II
N1103.4.2	EC112 Part II
N1103.5	EC79 Part II, EC131 Part II
N1103.5 (New)	EC119 Part II
N1103.5.1	EC99 Part II
N1103.6	EC120 Part II
T N1103.6 (New)	EC121 Part II
N1103.8	EC124 Part II
N1103.9	EC125 Part II
N1103.10 (New)	EC126 Part II
N1104 (New)	EC131 Part II
N1104.1	EC127, EC129, EC130 (All Part II)
N1104.1.1	EC18
M1301.2 thru M1301.5	FG14 PIII
M1303.1	M6 PII
M1303.1	FG10 PIII
M1307.3, P2801.6	M10 PII
M1401.1	M9 PII
M1407.1	M121 PII
M1411.5	FG11 PIII
M1411.6	M130 PII, M131 PII
M1411.6.1	M133 PII
M1413.1	M126 PII
M1413.2	M126 PII
M1502.4.1	M35 PII, M36 PII
M1502.4.2	M35 PII
M1502.4.4.1	M35 PII
M1502.4.4	M38 PII, M39 PII, M40 PII
M1502.4.4.2	M38 PII
M1502.4.4.3	M39 PII, M40 PII
M1503.1	M45 PII
M1503.2	M46 PII
M1506.1	M31 PII
M1601.1.1	M97 PII
Table M1601.1.1(2)	M98 PII

M1601.1.2	M102 PII, M103 PII
M1601.4.1	M105 PII
M1601.4	M110 PII
M1601.3	M111 PII, M112 PII
M1602.2	FG32 PIII
M2005.1	M127 PII
M2001.1.1	M128 PII
Table M2101.1	M140 PII, M141 PII, M142 PII
M2104.5	M148 PII
M2201.5	M151 PII
M2201.5	M151 PII
M2204.2	M152 PII
P2503.6	P18 Part II
P2503.8.2	P19 Part II, P106 Part II
P2601.2	P152 Part II
P2603.3	P8 Part II
P2603.4	P16 Part II
P2603.5	P10 Part II
Table P2605.1	P70 Part II
P2608.1	P7 Part II
P2608.4	P7 Part II
Table P2608.4	P7 Part II
Table P2701.1	P37 Part II
P2706.1	P121 Part II
P2708.1.1	P52 Part II
P2708.4 (New)	P53 Part II
P2709.2.1	P54 PartII
P2709.2.2	P54 PartII
P2709.2.4 (New)	P55 PartII
P2713.1	P40 Part II, P41 Part II
P2721.2	P42 Part II
P2722.5 (New)	P60 Part II
P2724.1 (New)	P61 Part II
P2803.6	P62 Part II
P2803.6 (New)	P63 Part II
P2803.6.1	P64 Part II
P2801.1.2	EC122 Part II
P2801.5	P65 Part II. P66 Part II P67 Part II
P2801.5.1	P67 Part II
P2801.5.3 (New)	P158 Part II
P2901.1	P87 Part II
P2902.1	P102 Part II
Table P2902.3	P83 Part II, P93 Part II
P2902.3.2	P93 Part II
P2902.3.3	P103 Part II
P2902.3.4	P96 Part II, P104 Part II
P2902.3.5	P95 Part II
P2902.3.6	P94 Part II
P2902.4	P94 Part II
P2902.4.2	P94 Part II
P2902.4.3	P86 Part II
P2902.5.1	P103 Part II
P2902.5.2	P154 Part II, P160 Part II
P2902.5.3	P100 Part II

IRC (continued)	
P2902.5.4	P99 Part II
P2902.5.4.1	P99 Part II
P2902.5.5	P94 Part II
P2902.6	P90 Part II
P2903.3.1	P157 Part II
P2903.5	P72 Part II
P2503.5.1	P156 Part II
Table P2903.6(1)	P153 Part II
P2903.9.5	P86 Part II
P2903.11 (New)	P75 Part II
P2904.3.1	P70 Part II
P2904.4.3	P94 Part II
Table P2904.6.2(8)	P70 Part II
Table P2904.6.2(9)	P70 Part II
P2905.4	P69 Part II
Table P2905.4	P68 Part II, P70 Part II
Table P2905.5	P70 Part II, P71 Part II
Table P2905.6	P70 Part II
P2905.9.1.2	P84 Part II
P2905.19 (New)	P70 Part II, P73 Part II
P2905.19.1(New)	P70 Part II
P2905.19.2 (New)	P70 Part II
P2908.1	P108 Part II
P2908.2	P108 Part II
P3001.4 (New)	P109 Part II
P3002.3.1	P111 Part II
P3003.9.2	P110 Part II
P3003.14.2	P110 Part II
P3003.19	P36 Part II
P3007.3.2.1 (New)	P114 Part II
P3007.3.3 (New)	P115 Part II
P3007.3.3.1 (New)	P115 Part II
P3007.3.3.2 (New)	P115 Part II
P3007.3.5	P116 Part II
P3009 (New)	P152 Part II
P3103.4	P124 Part II
P3103.5	P159 Part II
P3111.2	P128 Part II
P3111.3	P127 Part II
P3113.4.1	P131 Part II
P3201.5	P135 Part II
P3201.2	P136 Part II
Chapter 44	P60 Part II, P68 Part II, P69 Part II, P70 Part II, P71 Part II, P73 Part II, P83 Part II, P106 Part II, P108 Part II, P135 Part II, P136 Part II, P157 Part II
Chapter 44	F108, Part II; F132, Part II
Chapter 44	ADM39
Appendix H	G2 Part II
Appendix K	G147 Part II
Appendix L	G204 Part II

INT. WILDLAND-URBAN INTERFACE CODE	
Chapter 1	ADM1 Part X
101.3	ADM3
102.4	ADM4
115 (New)	ADM16 Part I
Chapter 15	ADM39
INTERNATIONAL ZONING CODE	
Chapter 1	ADM1 Part XI
101.2	ADM3
112 (New)	ADM16 Part I
Chapter 14	ADM39

2009/2010 ICC CODE DEVELOPMENT HEARING SCHEDULE

October 24 – November 11, 2009
Hilton Baltimore

Unless noted by "Start no earlier than X am/pm," each Code Committee will begin immediately upon completion of the hearings for the prior Committee. Thus the actual start times for the various Code Committees are tentative. The hearing volume is higher than previous cycles. The schedule anticipates that the hearings will finish by the times noted as "Finish" for each track and each week.

CODE DEVELOPMENT HEARINGS: OCTOBER 24 - 31

	Saturday October 24	Sunday October 25	Monday October 26	Tuesday October 27	Wednesday October 28	Thursday October 29	Friday October 30	Saturday October 31
TRACK 1	Start 8 am IWUIC IFC End 8 pm	Start 10 am IFC End 8 pm	Start 8 am IFC IRC-Energy (Start no earlier than 1 pm) End 8 pm	Start 8 am IRC – Energy End 8 pm	Start 8 am IRC-Building (Start no earlier than 8 am) End 8 pm	Start 8 am IRC- Building End 8 pm	Start 8 am IRC – Building Admin (Start no earlier than 3 pm) End 8 pm	Start 8 am Admin Finish 3 pm
TRACK 2	Start 8 am IBC-Structural End 8 pm	Start 10 am IBC-Structural End 8 pm	Start 8 am IBC-Structural End 8 pm	Start 8 am IBC-Structural End 8 pm	Start 8 am IECC (Start no earlier than 8 am) End 8 pm	Start 8 am IECC End 8 pm	Start 8 am IECC End 8 pm	Start 8 am IECC Finish 8 pm

ANNUAL CONFERENCE: NOVEMBER 1 - 4

CODE DEVELOPMENT HEARINGS: NOVEMBER 4 - 11

	Wednesday November 4	Thursday November 5	Friday November 6	Saturday November 7	Sunday November 8	Monday November 9	Tuesday November 10	Wednesday November 11
TRACK 1	Start 8 am IPM/ZC IEBC IBC-Fire Safety End 5 pm	Start 8 am IBC-Fire Safety End 8 pm	Start 8 am IBC – Fire Safety IBC – General (Start no earlier than 3 pm) End 8 pm	Start 8 am IBC - General End 8 pm	Start 10 am IBC – General IBC – Egress (Start no earlier than 3 pm) End 8 pm	Start 8 am IBC - Egress End 8 pm	Start 8 am IBC - Egress End 8 pm	Start 8 am IBC - Egress Finish 12 pm
TRACK 2	Start 8 am IPC/IPSDC End 5 pm	Start 8 am IPC/IPSDC End 9 pm	Start 8 am IMC (Start no earlier than 8 am) End 9 pm	Start 8 am IMC IRC-Plumbing/ Mechanical (Start no earlier than 1 pm) End 9 pm	Start 10 am IRC – Plumbing/ Mechanical End 9 pm	Start 8 am IRC – Plumbing/ Mechanical IFGC (Start no earlier than 8 am) Finish 9 pm	NO HEARINGS TRACK 2 COMPLETED	

Notes:

1. Hearing times may be modified at the discretion of the Chairman. Breaks will be announced.
2. Proposed code changes submitted to the International Wildland-Urban Interface Code (IWUIC) to be heard by the IFC Committee.
3. Proposed code changes submitted to the International Zoning (Z) and Property Maintenance (PM) Codes to be heard by the IPM/Z Committee.
4. "Admin" is a new code committee who will hear changes that affect coordination of Chapter 1 of all the I-Codes, except the IRC, and referenced standards updates.

2009/2010 PROPOSED CHANGES TO THE INTERNATIONAL CODES

Registration
Delegate

2009 Annual Conference and Code Development Hearings
Hearings: October 24–31 and November 4–11
Hilton Baltimore
Conference: November 1–4
Baltimore Convention Center

FIRST NAME AND M.I. | LAST NAME/SURNAME

JOB TITLE

JURISDICTION/ORGANIZATION

MAILING ADDRESS

CITY | STATE/PROVINCE | ZIP/POSTAL CODE

COUNTRY | E-MAIL (MUST PROVIDE TO RECEIVE CONFIRMATION)

PHONE (SPECIFY COUNTRY AND CITY CODE IF OUTSIDE THE U.S.) | FAX (SPECIFY COUNTRY AND CITY CODE IF OUTSIDE THE U.S.)

Are you an ICC Member? ☐ NO ☐ YES, my ICC Membership Number is: ___________ ☐ Check here if this is your first ICC Conference.

Type of Registration	ICC Member (BEFORE SEPTEMBER 1)	Nonmember (BEFORE SEPTEMBER 1)	ICC Member (AFTER SEPTEMBER 1)	Nonmember (AFTER SEPTEMBER 1)
☐ Full Conference Registration (includes all business, education and social functions)	$495*	$625*	$560*	$685*
☐ Code Development Hearings only (Registration is required to verify voting status)	FREE Registration		FREE Registration	
☐ One-Day Education ☐ Monday, November 2 ☐ Tuesday, November 3	$125	$160	$160	$190
☐ Golf Tournament (per person)** Handicap ______	$75	$75	$125	$125
☐ Golf Club Rental** ☐ Men's ☐ Women's ☐ Left ☐ Right	$25	$25	$25	$25

REGISTER BY SEPTEMBER 1 AND SAVE!

All fees are in U.S. dollars.

TOTAL $ ______________

Payment Options:

☐ BILL ME (ICC MEMBERS ONLY)
☐ CHECK (PAYABLE TO ICC)
☐ VISA ☐ MASTERCARD
☐ AMERICAN EXPRESS

SIGNATURE

CREDIT CARD NUMBER | EXP. DATE

The Code Council reserves the right to photograph or videotape events for promotional purposes. Your registration serves as permission for ICC to copyright, publish and use your likeness in print, online or in other media. If you do not wish to be photographed or videotaped, please tell the camera operator.

Cancellation Policy: *All cancellation requests must be received in writing. Cancellations received prior to September 1 will receive a full refund. Requests received between September 2–October 5 will be refunded, less a $50 administrative charge. Cancellations received after October 5 will not be eligible for a refund.*

**Take $10 off when you register online.*

***Payment is required with registration.*

TO ATTEND EDUCATION SESSIONS, PLEASE COMPLETE THE EDUCATION PROGRAM FORM ON REVERSE.

Save $10 When You Register Online

Register online: www.iccsafe.org/conference

Fax to: (708) 799-2307

Mail to: 2009 ICC Annual Conference
International Code Council
4051 W. Flossmoor Road
Country Club Hills, IL 60478

Phone registrations are not accepted.
Please do not fax AND mail your registration.

Lodging information available online.

If you have any questions, please call 1-888-ICC-SAFE, x4226 or x4229.

09-01697

REQUIRED INFORMATION FOR EDUCATION PROGRAM

Last Name ______________________ First Name ______________________

SESSION SELECTION

If you are registering for the full conference, please enter a session number for each time slot.
If you are registering for one day of education only, please check the day you will be attending and enter your session selection number.

☐ **Monday, November 2**
1:15 pm–4:15 pm
Session selection: # ______________

☐ **Tuesday, November 3**
1:15 pm–4:15 pm
Session selection: # ______________

EARN CEUs

Earn continuing education recognition for attending sessions at the Conference. Indicate your choice(s) and provide your license or credential number (ID number) for each:

ALABAMA
☐ Board of Heating & Air Conditioning Contractors
ID Number ______________

CALIFORNIA
☐ Council for Interior Design Certification/ CCIDC
ID Number ______________

CONNECTICUT
☐ Department of Public Safety, Office of Education & Data Management
ID Number ______________

FLORIDA
☐ Building Code Administrators & Inspectors Board
ID Number ______________

☐ Florida Professional Engineers Board
ID Number ______________

GEORGIA
☐ Fire Fighter Standards and Training Council
ID Number ______________

KANSAS
☐ Johnson County Contractor Licensing
ID Number ______________

KENTUCKY
☐ Division of Building Code Enforcement, Department of Housing, Buildings, & Construction
ID Number ______________

MAINE
☐ State Planning Office
ID Number ______________

MASSACHUSETTS
☐ Board of Building Regulations and Standards
ID Number ______________

MARYLAND
☐ Hartford County Department of Inspections, License & Permits, Building Services
ID Number ______________

MICHIGAN
☐ Office of Fire Safety
ID Number ______________

☐ Bureau of Construction Codes
ID Number ______________

MISSOURI
☐ Board of Professional Registration – APELSLA
ID Number ______________

NEW JERSEY
☐ Department of Community Affairs, Division of Codes and Standards
ID Number ______________

☐ Department of Community Affairs, Division of Fire Safety
ID Number ______________

NEW YORK
☐ Department of State, Codes Division
Requires Social Security # ______________
ID Number ______________

☐ Department of State, Office of Fire Prevention
Requires Social Security # ______________
FDID #/City Code ______________
County Code ______________
ID Number ______________

NORTH CAROLINA
☐ Code Officials Qualification Board
Requires Driver's License # ______________
ID Number ______________

OHIO
☐ Ohio Department of Commerce, Board of Building Standards
ID Number ______________

☐ Ohio Department of Commerce, Division of Industrial Compliance, Plumbing Section
ID Number ______________

OKLAHOMA
☐ Construction Industries Board, Inspector Examining Committee
ID Number ______________

PENNSYLVANIA
☐ Department of Labor and Industry
ID Number ______________

RHODE ISLAND
☐ State Building Code Commission
ID Number ______________

SOUTH CAROLINA
☐ Department of Labor, Licensing and Regulation Board of Building Codes Council
ID Number ______________

TENNESSEE
☐ Commerce and Insurance, Fire Prevention Division (aka State Fire Marshal's Office)
ID Number ______________

TEXAS
☐ Department of Licensing and Regulation, Electrical Safety and Licensing Advisory Board
ID Number ______________

UTAH
☐ Division of Occupational and Professional Licensing, Contractor Licensing
ID Number ______________

WISCONSIN
☐ Safety and Buildings Division
ID Number ______________

☐ **AMERICAN INSTITUTE OF ARCHITECTS**
ID Number ______________

☐ **AMERICAN SOCIETY OF HOME INSPECTORS**
ID Number ______________

☐ **INTERNATIONAL CODE COUNCIL**
ID Number ______________

☐ **OTHER**
ID Number ______________

Many professional organizations, boards, and state agencies recognize ICC educational offerings. If you do not find your professional organization or agency listed above, you may still be able to earn continuing education credit by attending these educational sessions. To find out if a specific ICC offering has been recognized by a specific board/agency for continuing education credit, contact the applicable agency/board. ICC cannot guarantee that a specific professional board, organization or agency will recognize an ICC educational offering.

2009/2010 PROPOSED CHANGES TO THE INTERNATIONAL BUILDING CODE — STRUCTURAL

STRUCTURAL CODE COMMITTEE

David P. Tyree, PE, CBO - ***Chair***
Director, Codes and Standards
Building Owners and Managers Association, International
Washington, DC

Marcelino Iglesias – ***Vice Chair***
Code Specialist
State of New Jersey
Dept. of Community Affairs; Division
Trenton, NJ

Alexander Abel, PE
Plans Review Engineer
City and County of Denver
Denver, CO

Lindsey Carter, PE, CBO
Inspection Services Supervisor
Shelby County Department of Development Services
Pelham, AL

Wanda Edwards, PE
Director of Building Code Development
Institute of Business & Home Safety
Tampa, FL

Thomas Faulkner, PE
President
Falcon Engineering
Raleigh, NC

Frank Golon
Rep: National Association of Home Builders
The Kleiner Group LLC
Eatontown, NJ

Cole Graveen, PE, SE
Engineer
Raths & Johnson Inc.
Willowbrook, IL

Kirk Grundahl, PE
President
Qualtim, Inc.
Madison, WI

John Loscheider, PE, SE
Principal
Loscheider Engineering Company
Renton, WA

James Robinson, Jr., PE, SE
Rep: National Council of Structural Engineering Associations
President
Robinson Associates Consulting Engineers Inc.
Atlanta, GA

Gary Searer, PE, SE
Associate Principal
Wiss, Janney, Elstner Associates, Inc.
Burbank, CA

Constadino Sirakis, PE
Acting Director of Engineering
Technical Affairs Division
New York City Dept. of Bldgs
New York, NY

Gary Walker, PE
President
Walker Engineering Inc.
Birmingham, AL

Can Xiao, MS, PE
Camelback Infrastructure Development Manager, Water Services Department
City of Phoenix
Phoenix, AZ

Staff Secretariat:
Alan Carr, SE
Code and Standards
International Code Council

TENTATIVE ORDER OF DISCUSSION

2009/2010 PROPOSED CHANGES TO THE INTERNATIONAL BUILDING CODE

STRUCTURAL

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

Proposed change numbers that are indented are those which are being heard out of numerical order. Indentation does **not** necessarily indicate that one change is related to another. Proposed changes may be grouped for purposes of discussion at the hearing at the discretion of the chair.

- S1-09/10, Part I
- S3-09/10, Part I
- S4-09/10
- S5-09/10
- S6-09/10
- S7-09/10
- S8-09/10
- S9-09/10
- S14-09/10, Part I
- S15-09/10, Part I
- S16-09/10, Part I
- S17-09/10, Part I
- S18-09/10, Part I
- S19-09/10
- S20-09/10, Part I
- S21-09/10, Part I
- S22-09/10, Part I
- S22-09/10, Part II
- S23-09/10, Part I
- S29-09/10
- S30-09/10, Part I
  - S43-09/10
- S32-09/10
- S33-09/10
- S34-09/10
- S35-09/10
- S36-09/10
- S37-09/10
- S38-09/10
- S39-09/10
- S40-09/10
- S41-09/10, Part I
- S41-09/10, Part II
- S42-09/10
- S45-09/10
- S47-09/10
- S49-09/10
- S50-09/10
- S52-09/10
- S53-09/10
- S54-09/10
- S55-09/10
- S56-09/10
- S58-09/10
- S59-09/10
- S60-09/10
- S62-09/10, Part I
  - S64-09/10
- S63-09/10
- S65-09/10
- S66-09/10, Part I
  - S57-09/10, Part I
  - S61-09/10, Part I
- S67-09/10
- S68-09/10
- S69-09/10
- S70-09/10
- S71-09/10
- S72-09/10
- S73-09/10
  - S31-09/10
- S74-09/10
- S75-09/10
- S76-09/10
- S77-09/10
- S78-09/10
- S79-09/10
- S80-09/10
- S81-09/10
- S82-09/10
- S83-09/10
- S84-09/10
  - S51-09/10
- S85-09/10
- S86-09/10
- S87-09/10, Part I
  - S179-09/10
  - EB71-09/10
  - EB72-09/10
  - EB16-09/10
  - EB17-09/10
- S88-09/10
- S89-09/10
- S90-09/10
- S92-09/10, Part I
- S92-09/40, Part II
- S92-09/10, Part III
- S92-09/10, Part IV
- S93-09/10
- S94-09/10
  - S91-09/10, Part I
  - S91-09/10, Part II
  - G196-09/10
  - G193-09/10
  - G194-09/10
  - EB4-09/10, Part I
  - EB4-09/10, Part II
  - EB6-09/10, Part I
  - EB6-09/10, Part II
  - EB7-09/10
  - EB8-09/10, Part I
  - EB8-09/10, Part II
  - EB9-09/10, Part I
  - EB9-09/10, Part II
  - G195-09/10
  - G192-09/10
  - G191-09/10
  - EB3-09/10, Part I
  - EB3-09/10, Part II
  - G190-09/10
  - G197-09/10
  - ADM32-09/10
  - ADM33-09/10
  - EB15-09/10
- EB21-09/10
- EB22-09/10
- EB24-09/10
- EB5-09/10
- EB36-09/10
- EB37-09/10
- EB38-09/10
- EB25-09/10
- EB40-09/10
- EB41-09/10
- EB42-09/10
- EB43-09/10
- EB44-09/10
- EB45-09/10
- EB46-09/10
- EB47-09/10
- EB48-09/10
- EB49-09/10
- EB50-09/10
- EB51-09/10
- EB52-09/10
- EB53-09/10
- EB54-09/10
- EB55-09/10
- EB56-09/10
- EB57-09/10
- EB58-09/10
- EB59-09/10
- EB60-09/10
- EB61-09/10
- EB62-09/10
- EB63-09/10
- EB64-09/10
- EB65-09/10
- EB66-09/10
- EB67-09/10
- EB68-09/10
- EB69-09/10

EB70-09/10
S95-09/10
S96-09/10
S97-09/10, Part I
S98-09/10
S99-09/10
S100-09/10
S101-09/10
S102-09/10
S103-09/10
S104-09/10
S105-09/10
S106-09/10
S44-09/10
S48-09/10
S107-09/10
S108-09/10
S109-09/10
G181-09/10
S110-09/10
G40-09/10
S112-09/10
S113-09/10
S114-09/10
S111-09/10
S115-09/10
S116-09/10
S117-09/10
S118-09/10
S119-09/10
S120-09/10
S121-09/10
S122-09/10
S123-09/10
S124-09/10
S125-09/10
S129-09/10
S130-09/10
S131-09/10
S132-09/10
S133-09/10
S134-09/10
S135-09/10
S136-09/10
S137-09/10
S138-09/10
S139-09/10
S140-09/10
S141-09/10
S142-09/10
S143-09/10
S144-09/10, Part I
S145-09/10
S146-09/10, Part I
S146-09/10, Part II
S147-09/10
S148-09/10
S149-09/10
S150-09/10
S151-09/10
S152-09/10
S153-09/10
S154-09/10
S155-09/10
S156-09/10
S157-09/10
S158-09/10
S159-09/10
S160-09/10
S161-09/10
S162-09/10, Part I
S163-09/10
S164-09/10
S165-09/10
S166-09/10
S167-09/10
S168-09/10
S170-09/10
S171-09/10, Part I
S172-09/10
S173-09/10
S174-09/10
S175-09/10
FS148-09/10
FS149-09/10
S176-09/10
S177-09/10
S178-09/10
S180-09/10
S181-09/10
S182-09/10, Part I
S183-09/10
S184-09/10
S185-09/10
S186-09/10
S187-09/10
S189-09/10
S190-09/10
S191-09/10
S192-09/10
S193-09/10
S188-09/10
S194-09/10
S195-09/10
S196-09/10
S197-09/10
S198-09/10
S199-09/10, Part I
S200-09/10, Part I
S201-09/10, Part I
S202-09/10
S203-09/10, Part I
S204-09/10
S205-09/10
S206-09/10
S207-09/01, Part I
FS189-09/10
S209-09/10
S208-09/10
S210-09/10
S211-09/10
S212-09/10
S213-09/10
S214-09/10, Part I
S215-09/10
S216-09/10
S217-09/10
S218-09/10, Part I
S219-09/10, Part I
S220-09/10
S221-09/10
S222-09/10, Part I
S223-09/10
S224-09/10, Part I
S225-09/10, Part I
FS156-09/10, Part I
FS180-09/10
G2-09/10, Part I
S46-09/10
S169-09/10

S1–09/10
1502.1; IRC R202

Proponent: Bob Eugene, representing Underwriters Laboratories Inc

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

1502.1 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.

ROOF ASSEMBLY. A system designed to provide weather protection and resistance to design loads, wind and fire. The system consists of a roof covering and roof deck or a single component serving as both the roof covering and the roof deck. A roof assembly includes the roof deck, vapor retarder, substrate or thermal barrier, insulation, vapor retarder and roof covering.

The definition of "Roof assembly" is limited in application to the provisions of Chapter 15.

PART II – IRC BUILDING/ENERGY

Revise as follows:

SECTION R202
DEFINITIONS

ROOF ASSEMBLY. A system designed to provide weather protection and resistance to design loads, wind and fire. The system consists of a roof covering and roof deck or a single component serving as both the roof covering and the roof deck. A roof assembly includes the roof deck, vapor retarder, substrate or thermal barrier, insulation, vapor retarder and roof covering.

Reason: Roof assemblies are also designed to provide wind resistance and fire resistance.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: EUGENE-S3-1502.1

S2–09/10
1503.4, 1503.4.1; IPC 1107.1; IRC R903.4, R903.4.1

Proponent: Don Surrena, CBO, National Association of Home Builders (NAHB)

THIS IS A 3 PART CODE CHANGE. PARTS I & II WILL BE HEARD BY THE IPC COMMITTEE. PART III WILL BE HEARD BY THE IRC PLUMBING COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IPC

Revise as follows:

1107.1 ~~Secondary drainage required~~. Secondary (emergency overflow) drains or scuppers. Where roof drains are required, secondary (emergency overflow) roof drains or scuppers shall be provided where the roof perimeter construction extends above the roof in such a manner that water will be entrapped if the primary drains allow buildup for any reason.

PART II – IBC

Revise as follows:

[P] 1503.4 Roof drainage. Design and installation of roof drainage systems shall comply with Section 1503 of this code and Sections 1106 and 1107 as applicable of the *International Plumbing Code.*

[P] 1503.4.1 ~~Secondary drainage required~~. Secondary (emergency overflow) drains or scuppers. Where roof drains are required, secondary (emergency overflow) roof drains or scuppers shall be provided where the roof perimeter construction extends above the roof in such a manner that water will be entrapped if the primary drains allow buildup for any reason. The installation and sizing of secondary emergency overflow drains, leaders and conductors shall comply with Sections 1106 and 1107 as applicable of the *International Plumbing Code.*

PART III – IRC PLUMBING

Revise as follows:

R903.4 Roof drainage. Unless roofs are sloped to drain over roof edges, roof drains shall be installed at each low point of the roof. ~~Where required for roof drainage, scuppers shall be placed level with the roof surface in a wall or parapet. The scupper shall be located as determined by the roof slope and contributing roof area.~~

R903.4.1 Secondary (emergency overflow) drains ~~and~~ or scuppers. Where roof drains are required, secondary emergency overflow roof drains or scuppers shall be provided where the roof perimeter construction extends above the roof in such a manner that water will be entrapped if the primary drains allow buildup for any reason. Overflow drains having the same size as the roof drains shall be installed with the inlet flow line located 2 inches (51 mm) above the low point of the roof, or overflow scuppers having three times the size of the roof drains and having a minimum opening height of 4 inches (102 mm) shall be installed in the adjacent parapet walls with the inlet flow located 2 inches (51 mm) above the low point of the roof served. The installation and sizing of overflow drains, leaders and conductors shall comply with Sections 1106 and 1107 as applicable of the *International Plumbing Code*.

Overflow drains shall discharge to an *approved* location and shall not be connected to roof drain lines.

Reason: (IPC/IBC) The purpose of this proposal is to clarify the requirements for roof drains and the requirements for secondary emergency overflow drains, their sizing, location and quantity. The requirements currently in the IBC did not alert the roofer about their responsibility to size drains and or scuppers. This code modification and additional section helps to alert the roofer that additional information and requirements are to be followed and provides a section reference.
(IRC) The purpose of this proposal is to clarify the requirements for roof drains and the requirements for secondary emergency overflow drains, their sizing, location and quantity. The requirements currently in the IRC did not alert the roofer about their responsibility to size drains and or scuppers. This code modification and additional section helps to alert the roofer that additional information and requirements are to be followed.

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

PART I – IPC

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

PART II – IPC

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

PART III – IRC PLUMBING

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

S3–09/10
1503.6; IRC R903.2.2

Proponent: John Woestman, The Kellen Company representing the Window and Door Manufacturers Association (WDMA)

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC-STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

1503.6 Crickets and saddles. A cricket or saddle shall be installed on the ridge side of any chimney or penetration greater than 30 inches (762 mm) wide as measured perpendicular to the slope. Cricket or saddle coverings shall be sheet metal or of the same material as the roof covering.

<u>**Exception:** Skylights installed and flashed in accordance with the manufacturer's instructions.</u>

PART II – IRC BUILDING/ENERGY

Revise as follows:

R903.2.2 Crickets and saddles. A cricket or saddle shall be installed on the ridge side of any chimney or penetration more than 30 inches (762 mm) wide as measured perpendicular to the slope. Cricket or saddle coverings shall be sheet metal or of the same material as the roof covering.

<u>**Exception:** Skylights installed and flashed in accordance with the manufacturer's instructions.</u>

Reason: This code language, as written, precludes the use of engineered skylight systems that are designed to prevent water infiltration into the penetration without the use of a cricket. The proposed change addresses this unintended consequence of this language of the IBC and the IRC.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: WOESTMAN-S4-1503.6

S4–09/10
1504.3, Chapter 35

Proponent: Mike Ennis representing Single Ply Roofing Industry (SPRI, Inc.)

1. Revise as follows:

1504.3 Wind resistance of nonballasted roofs. Roof coverings installed on roofs in accordance with Section 1507 that are mechanically attached or adhered to the roof deck shall be designed to resist the design wind load pressures for components and cladding in accordance with Section 1609 <u>and shall be installed in accordance with ANSI/SPRI WD-1.</u>

2. Add standard to Chapter 35 as follows:

SPRI
<u>WD-1-08</u> <u>Wind Design Standard Practice for Roofing Assemblies</u>

Reason: The International Building Code provides specific requirements for calculating the wind uplift load pressure on the roof assembly. However it does not currently provide a prescriptive method to enhance the perimeter and corner attachment due to the higher wind loads in these regions. ANSI/SPRI WD-1 is a national consensus standard that has been reviewed by testing laboratories, membrane manufacturers, roofing system component suppliers, contractors and consultants. This standard provides prescriptive requirements for corner and perimeter enhancement. The user first identifies a suitable roof assembly that will resist the calculated wind uplift pressure for the field of the roof, then enhances the fastening pattern to meet the calculated corner and perimeter wind uplift load pressure. Designing the roof system to resist the higher wind loads at the perimeter and corner regions is accomplished by either adding additional fasteners or increasing the amount of adhesive used, depending upon the specific roof system chosen. This approach allows the user to work from one base assembly and enhance the attachment of the base assembly for perimeter and corner regions instead of trying to locate tested assemblies for each of these areas.

The ANSI/SPRI standard also requires that a 2.0 safety factor be applied to tested wind uplift values, unless another value is specified. So, for example, if a roof system passes a wind uplift test at 120 lbs/ft2, this value is divided by 2 before determining if the system will resist the calculated wind uplift pressure loads for the building. The IBC does not currently contain this requirement.

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, SPRI WD-1-08, 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.

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

ICCFILENAME: ENNIS-S1-1504.3

S5–09/10
1502.1, 1504.4, 1504.6, 1504.7

Proponent: Mark S. Graham, representing National Roofing Contractors Association (NRCA)

1. Add new definition as follows:

1502.1 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.

<u>**LOW SLOPE.** In roofing, that which commonly describes an incline of a roof which is less than two units vertical in 12 units horizontal (16.7-percent).</u>
<u>The definition of “Low slope” is limited in application to the provisions of Chapter 15.</u>

2. Revise as follows:

1504.4 Ballasted low-slope roof systems. Ballasted low-slope ~~(roof slope < 2:12)~~ single-ply roof system coverings installed in accordance with Sections 1507.12 and 1507.13 shall be designed in accordance with Section 1504.8 and ANSI/SPRI RP-4.

1504.6 Physical properties. Roof coverings installed on low-slope roofs ~~(roof slope < 2:12)~~ in accordance with Section 1507 shall demonstrate physical integrity over the working life of the roof based upon 2,000 hours of exposure to accelerated weathering tests conducted in accordance with ASTM G 152, ASTM G 155 or ASTM G 154. Those roof coverings that are subject to cyclical flexural response due to wind loads shall not demonstrate any significant loss of tensile strength for unreinforced membranes or breaking strength for reinforced membranes when tested as herein required.

1504.7 Impact resistance. Roof coverings installed on low-slope roofs ~~(roof slope < 2:12)~~ in accordance with Section 1507 shall resist impact damage based on the results of tests conducted in accordance with ASTM D 3746, ASTM D 4272, CGSB 37-GP-52M or the “Resistance to Foot Traffic Test” in Section 5.5 of FM 4470.

Reason: This proposed code change is intended to add clarity to the code by providing a specific definition in Section 1502—Definitions for the term “low slope” that is used in several instances in Chapter 15.

Currently in Chapter 15, there are several instances where usage of the term low-slope is defined parenthetically as “…(roof slope < 2:12)…”. In other instance in Chapter 15, the term is not specifically defined. Adding a specific definition for the term in Section 1502—Definitions provides for consistent interpretation throughout the chapter.

The addition of the notation limiting the applicability of the definition to Chapter 15 is necessary to avoid possible conflicts with other chapters; a similar notation is also included in Section 1502—Definitions for the term “Roof assembly.”

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

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

ICCFILENAME: GRAHAM-S1-1502.1

S6–09/10

1504.4, 1504.4.1 (New), 1504.4.2 (New), Table 1504.4 (New), 1504.8, Table 1504.8

Proponent: Thomas L Smith, AIA, RRC, TLSmith Consulting Inc. on behalf of the Roofing Industry Ad Hoc Working Group on Roof Aggregate (including, the Federal Emergency Management Agency, the Asphalt Roofing Manufacturers Association and SPRI).

1. Delete and substitute as follows:

~~**1504.4 Ballasted low-slope roof systems.** Ballasted low-slope (roof slope < 2:12) single-ply roof system coverings installed in accordance with Section 1507.12 and 1507.13 shall be designed in accordance with Section 1504.8 and ANSI/SPRI RP-4.~~

1504.4 Aggregate and paver surfaced low-slope roof coverings. Aggregate and paver surfaced roof system coverings shall be designed and installed in accordance with Section 1504.4.1 or 1504.4.2 as applicable.

2. Add new text as follows:

1504.4.1 Ballasted surfaced roof coverings. Aggregate and paver surfaced roof system coverings shall be designed and installed in accordance with ANSI/SPRI RP-4.

Exceptions:

1. Aggregate and concrete pavers are not permitted where the building height exceeds 150 feet (45 720 mm).
2. In hurricane-prone regions as defined in Section 1609.2, aggregate is not permitted on Occupancy Category III or IV buildings where the basic wind speed is greater than 100 mph (45 m/s).

1504.4.2 Aggregate surfaced roof coverings. Aggregate surfaced roof system coverings shall be designed and installed in accordance with Table 1504.4 based on the exposure category and basic wind speed at the building site. The aggregate shall comply with ASTM D 1863.

Exceptions:

1. In hurricane-prone regions as defined in Section 1609.2, aggregate is not permitted on Occupancy Category III or IV buildings where the basic wind speed is greater than 100 mph (45 m/s).
2. In hurricane-prone regions as defined in Section 1609.2, aggregate is not permitted on Occupancy Category I or II buildings when the basic wind speed is greater than 110 mph (49 m/s).

TABLE 1504.4
MINIMUM REQUIRED PARAPET HEIGHT (INCHES) FOR AGGREGATE SURFACED ROOF COVERINGS[a,b]
FOR OCCUPANCY CATEGORY I AND II BUILDINGS [c]

ASTM D1863 Gradation	Mean Roof Height[d] (ft)	WIND EXPOSURE AND BASIC WIND SPEED (MPH, GUST)[e]														
		Exposure Category B					Exposure Category C					Exposure Category D				
		85	90	100	110	120	85	90	100	110	120	85	90	100	110	120
No. 7 or No. 67	15	0	0	15	20	25	22	25	31	38	45	27	31	38	45	53
	20	0	12	17	23	28	23	27	33	40	47	29	32	40	47	55
	30	13	15	21	27	32	26	29	36	44	51	31	35	43	50	58
	40	15	18	24	29	35	28	31	39	46	53	33	37	45	52	60
	50	17	20	26	32	38	29	33	40	48	55	34	38	46	54	62
	60	18	21	28	34	40	30	34	42	49	57	35	39	47	56	64
	80	21	24	30	37	43	32	36	44	52	60	37	41	49	58	66
	100	23	26	33	40	46	34	38	46	54	62	38	43	51	60	68
	125	25	28	35	42	49	36	40	48	56	64	40	44	53	62	70
	150	27	30	37	45	52	37	41	50	58	66	41	45	54	63	72
No. 6	15	0	0	11	15	20	16	19	25	31	37	22	25	31	38	45
	20	0	0	13	17	22	18	21	27	34	40	23	26	33	40	47
	30	0	11	16	21	26	20	24	30	36	43	25	29	36	43	50
	40	0	13	18	24	29	22	25	32	39	45	27	30	37	45	52
	50	12	15	20	26	31	23	27	34	40	47	28	31	39	46	53
	60	13	16	22	28	33	24	28	35	42	49	29	33	40	47	55
	80	16	19	25	30	36	26	30	37	44	51	30	34	42	50	57
	100	18	21	27	33	39	28	31	39	46	53	32	36	43	51	59
	125	19	23	29	35	42	29	33	40	48	55	33	37	45	53	61
	150	21	24	31	37	44	30	34	42	50	57	34	38	46	54	62

SI: 1" = 25.4 mm, 1 ft = 0.3 m, 1 mph = 0.44 m/s

a. Interpolation between wind speeds and building heights shall be permitted.

b. Aggregate surfaced roofs shall not be permitted for basic wind speeds greater than 120 mph, or where the building height exceeds 150 feet.

c. For Occupancy Category III and IV buildings, use the next higher wind speed column.

d. Mean roof height shall be measured from the grade plane to the roof surface at the perimeter of the roof portion under consideration.

e. Wind exposure and basic wind speed shall be determined in accordance with ASCE 7.

3. Delete without substitution:

~~**1504.8 Aggregate.** Aggregate used as surfacing for roof coverings and aggregate, gravel or stone used as ballast shall not be used on the roof of a building located in a hurricane-prone region as defined in Section 1609.2, or on any other building with a mean roof height exceeding that permitted by Table 1504.8 based on the exposure category and basic wind speed at the site.~~

~~TABLE 1504.8~~
~~MAXIMUM ALLOWABLE MEAN ROOF HEIGHT PERMITTED FOR BUILDINGS WITH GRAVEL OR STONE ON THE ROOF IN AREAS OUTSIDE A HURRICANE-PRONE REGION~~

~~Basic Wind Speed From Figure 1609 (mph)~~ [b]	~~Maximum Mean Roof Height (ft)~~ [a,c]		
	~~Exposure category~~		
	~~B~~	~~C~~	~~D~~
~~85~~	~~170~~	~~60~~	~~30~~
~~90~~	~~110~~	~~35~~	~~15~~
~~95~~	~~75~~	~~20~~	~~NP~~
~~100~~	~~55~~	~~15~~	~~NP~~
~~105~~	~~40~~	~~NP~~	~~NP~~
~~110~~	~~30~~	~~NP~~	~~NP~~
~~115~~	~~20~~	~~NP~~	~~NP~~
~~120~~	~~15~~	~~NP~~	~~NP~~
~~Greater than 120~~	~~NP~~	~~NP~~	~~NP~~

~~Greater than 120 NP NP NP~~
~~For SI: 1 foot = 304.8 mm; 1 mile per hour = 0.447 m/s.~~
~~a. Mean roof height in accordance with Section 1609.2.~~
~~b. For intermediate values of basic wind speed, the height associated with the next higher value of wind speed shall be used, or direct interpolation is permitted.~~
~~c. NP = gravel and stone not permitted for any roof height.~~

Reason: Concern with roof aggregate blow-off is not new (Minor, 1977). It has continued to be reinforced by field observations, particularly in regard to damage caused to glazing on surrounding buildings as well as the building from which the aggregate was lifted into the airstream. Most problems have been associated with extreme wind events such as hurricanes and have involved roofs not in compliance with RP-4 and with aggregate surfaced roofs for which the RP4 standard was not intended to address. As a result, recent building code changes (i.e., IBC 2006 and 2009) have severely restricted the use of aggregate surfaced roofs. However, these new restrictions were not based on the K-W design method (Kind Wardlaw 1976), the wind tunnel studies underlying the K-W design method (Kind 1977), or a quantitative analysis of observed good and bad roofing system performances in real wind events. Instead, current building code limitations are based on variation in surface pressure with building height which is known to be an inappropriate predictor of aggregate blow-off or scour due to pressure equalization effects (Smith, 1997). Furthermore, these recent restrictions do not address critical parameters such as aggregate size and parapet height which govern performance.

This code change proposal addresses two types of roof coverings: ballasted single ply roofs and those with aggregate surfaces, such as Built-up roofs (BUR) and certain spray polyurethane roof systems. Reasoning statements are provided for each new section:

New section1504.4.1 - Over 6 billion square feet of ballasted single ply roofing applications have been installed over the last two decades The vast majority of these systems have performed very well with respect to their resistance to wind pressure loads. However some damage has been observed due to aggregate blowing off non-code compliant roofs during high wind events. The above proposals are based on over 200 wind tunnel tests in addition to over 40 years of field experience and observations from hurricane investigation teams. These proposals provide restrictions on the use of ballasted single ply roof systems that will allow for the responsible use of aggregate surfacing that is a cost effective method to keep the roof system in place and to improve the energy performance of the building.

ANSI/SPRI RP-4 is the code referenced design guide for ballasted single ply roof systems. The requirements contained in the guide are based on over 200 wind tunnel tests along with extensive field studies. One of the design criteria of ANSI/SPRI RP-4 is to prevent gravel blow-off. Wind tunnel testing conducted at the National Research Council Canada evaluated conventional stone ballasted and stone and paver ballasted protected membrane roofs. For the systems containing stone ballasting the primary objective was to determine 4 critical wind speeds:

1. U_{c1} – the wind speed at which one or more stones were first observed to move an appreciable distance (i.e. several inches)
2. U_{c2} – the wind speed above which scouring of stones would continue more or less indefinitely as long as the wind speed is maintained.
3. U_{c3} – the wind speed at which stones were first observed to leave the roof by going over the upstream parapet (this was the parapet adjacent to the wind direction)
4. U_{c4} – the wind speed at which stones were first observed to leave the roof by going over the downstream parapet (opposite side from the wind)

In these experiments three nominal stone sizes were used. Each nominal stone size represented a mixture of stone sizes (larger and smaller) similar to the gradation, which would be obtained from a stone quarry. These experiments evaluated the impact of the following variables on the critical wind speeds defined above:

Stone size
Parapet height
Building height
Building geometry
Direction of wind impacting the building
Rooftop wind speed, rooftop gust wind speed, and the shape of the approaching wind velocity profile

In addition to the extensive wind tunnel test program, observed field performance was also a basis for the requirements included in ANSI/SPRI RP-Two of the most critical controlling factors identified through this extensive test program on the various critical wind speeds were stone size and parapet height. A brief summary of the wind tunnel test program, and reports written as part of this program follows.

LTR-LA-142 Estimation of Critical Wind Speeds for Scouring of Gravel or Crushed Stone on Rooftops January 1974

Objectives:

Determine the critical wind speeds and corresponding surface shear stress that cause movement of various stone sizes and shapes by taking direct measurements of these values via wind tunnel testing.
Use this data to determine constants that can be used in equations to calculate critical surface shear stress

Obtain guidance about the effects of parapets and obstacles, which cause strong three-dimensional effects, notably vortices.

Conclusions:

The surface shear stress required to cause stone motion is directly proportional to nominal stone diameter.

The constant of proportionality appears to be essentially independent of stone size and shape and of the detailed shape of the velocity profile near the gravel surface.

Critical wind speeds to initiate stone motion can therefore be easily predicted if the relationship between surface shear stress and wind speed is known for the situation of interest.

The dead air region behind a parapet extended downstream about 15 parapet heights. The turbulence of natural wind will tend to reduce the dead air zone.

LTR-LA-162 Wind Tunnel Tests on Some Building Models to Measure Wind Speeds at Which Gravel is Blown Off Rooftops June 1974

Objectives:

This series of tests was conducted to build upon the data obtained in the January 1974 test series. Specifically to provide data for some typical building geometries and to investigate the effects of building form, building height, parapet height, wind direction, and gravel size on the critical wind speeds required to cause scouring and blow-off of roofing gravel.

In this series 1/10 scale models were evaluated in a 30' x 30' wind tunnel.

Conclusions

The critical wind speeds at which scouring of nominal 0.9", 1.5" and 2.8" diameter gravel (scaled to 1/10 size) occurs and begins to blow-off rooftops were investigated. The nominal sizes represent the average size of a typical mixture.

The critical wind speeds are lowest when the wind direction is at or about 45 to the walls of the building.

For a given building configuration the critical wind speeds are proportional to the square root of the gravel size.

The critical wind speeds increase with increasing parapet height and decrease with increasing building height.

The length:width ratio of the building is unimportant as long as the width and length are large compared to the parapet height.

NRC No. 15544 Design of Rooftops Against Gravel Blow-Off September 1976

Objectives:

This report describes a procedure that can be used to estimate the wind speeds at which gravel of a given nominal size will be blown off rooftops.

The report also describes a procedure for determining design wind speeds at rooftop level.

The gravel blow-off procedure is based on data obtained from previous wind tunnel tests described above.

Conclusions

The results of wind tunnel tests conducted to determine critical wind speeds for scour or blow-off of roofing gravel for a specific low-rise building shape can be generalized to apply to any low-rise rectangular building having a flat rooftop.

Similar generalization is possible for high-rise shapes of any particular length:width ratio.

This permits development of a general, easy to use procedure for estimating critical wind speeds required to cause scour or blow-off of roofing gravel from various building configurations.

LTR-LA-189 Further Wind Tunnel Tests on Building Models to Measure Wind Speeds at Which Gravel is Blown Off Rooftops August 1977

Objectives:

Obtain additional data to permit previously obtained results to be generalized so as to be applicable to any rectangular flat-roofed low-rise building.

Provide data on the effects of substituting solid paving blocks for loose gravel in the most wind sensitive areas of the rooftop.

Conclusions:

The wind speed at rooftop level appears to be the dominant factor in controlling gravel scour and blow-off as opposed to the wind velocity profile.

The measured wind speeds at rooftop level were used to reinterpret the data from previous wind tunnel tests.

Within the boundaries of experimental scatter the critical wind speeds are independent of the rooftop level in the wind boundary layer, allowing for generalization of results to various building heights and geometries.

LTR-LA-234 Model Studies of the Wind Resistance of Two Loose-Laid Roof-Insulation Systems May 1979

Objectives:

Investigate the resistance of protected membrane roof systems to damage from high winds.

Identify wind speeds and failure mechanisms for protected membrane roof systems.

Conclusions:

The results show that wind flows induce pressure distributions underneath the roof-insulation systems as well as on their exterior surfaces. These pressure differences cause uplift and are responsible for system failure.

The wind speed to cause failure for the 2 ft. x 2 ft. paver slabs was found to be proportional to the square root of the system weight per unit area. This relationship should also be true for different geometries.

LTR-LA-269 Further Model Studies of the Wind Resistance of Two Loose-Laid Roof-Insulation Systems (High Rise Buildings) April 1984

Objectives:

This study is an extension of the May 1979 study, to investigate the resistance of various protected membrane roof systems to damage from high winds when they are installed on high-rise buildings.

Conclusions:

The mechanisms for wind damage are the same as those identified in earlier tests, namely gravel scour and uplifting of boards by pressure forces.

The static pressure underneath boards or pavers tend to become equal to the exterior surface because of airflow through the joints between boards or pavers. Complete equalization cannot occur, however, in regions where the exterior pressure distribution is highly non-linear and uplifting pressure differences occur in those regions. System failure therefore tends to occur in these regions.

High parapets are very effective in increasing resistance to wind damage.

Mechanical interconnection of boards or pavers by use of strapping, tongue & groove, etc. is an effective method for increasing wind resistance.

For any particular system configuration, the wind speed to cause failure is proportional to the square root of the system weight per unit area.

Gust speed at rooftop level is the pertinent speed for use in assessing the resistance of the roofing system to wind damage.

LTR-LA-294 Further Wind Tunnel Tests of Loose-Laid Roofing Systems April 1987

Objectives:

Conduct extensive wind tunnel work to further assess the resistance to wind damage of protected membrane roofing system using paver slabs, or similar elements.

Low, intermediate and high-rise buildings were tested, each with several parapet heights.

Conclusions:

When a membrane is loose-laid on a leaky roof deck, ballooning will occur due to air flowing through holes in the deck from the interior of the building. This will normally result in failure at wind speeds well below those required to product failure by other mechanisms.

In the case of immobile membranes, failure results from pressure differences, which develop across elements in some regions of the roof.

Increased parapet height generally resulted in more favorable pressure distributions. That is, maximum suctions were reduced and suction peaks were broadened, so that pressure was less non-uniform and therefore increased failure speeds could be expected.

Element size has a noticeable effect on failure speed, i.e. failure speeds were higher for larger elements.

Pressure non-uniformity is reduced by vortex generators mounted on the parapets near the upwind corner of the roof, thus increasing failure wind speeds.

LTR-LA-295 Pressure Distribution Data Measured During the September 1986 Wind Tunnel Tests on Loose-Laid Roofing Systems September 1987

Objectives:

This report supplements LTR-LA-294 by including contour plots of mean and peak roof surface pressure coefficients and mean and peak coefficients for pressure differential between the upper surface and the underside of the roofing system.

New Section 1504.4.2 and New Table 1504.4 – The new Section 1504.2 provides prescriptive design requirements to avoid blow-off of loose aggregate used on roof systems such as BUR and certain SPF roofs. Based on the Kind-Wardlaw design method, blow-off of loose aggregate is avoided by using minimum parapet heights determined by wind speed, Exposure Category, roof height, Building Category and aggregate size.

The Kind-Wardlaw design method ("K-W" design method) for prevention of scour and blow-off of aggregate from aggregate surfaced roofs has been available since the 1970s (Kind & Wardlaw 1976). It saw limited use until the 1980s when adopted as the basis for the initial 1988 edition of ANSI/SPRI RP-4, "Wind Design Standard for Ballasted Single-Ply Roofing Systems," (RP-4) standard which, in its updated form, continues to be used by the single-ply roofing industry for ballasted roofs (SPRI 2008). SPRI utilized the K-W design method as the basis for aggregate ballasted roof systems to prevent ballast scour.

For aggregate surfaced roof systems, a main wind-related issue of concern is with aggregate blow-off. Scour is not considered important to aggregate surfaced roofs, except to the extent that it might require maintenance (re-distribution of aggregate) after an extreme wind event to maintain fire resistance and the long-term durability of the roof system against degradation caused by solar (U-V) radiation or cause pile-up of aggregate on a downwind parapet.

The technical underpinnings of this proposal are three-fold:

1. The wind tunnel basis of the K-W design method was re-evaluated to confirm or make technically supported improvements with respect to prevention of aggregate blow-off
2. Quantitative field observations of roofing system performance in extreme wind events were compared with the K-W design method
3. Prescriptive design requirements to prevent aggregate blow-off based on items 1 and 2 were developed as a new Table 1504.4 for inclusion in the IBC.

As a result of this study (Crandell, 2009), improvements are recommended for a modified K-W design method for aggregate surfaced roofs. The improvements include a reconfiguration of the design method that allows critical wind velocity for initiation of aggregate blow-off to be predicted as a linear relationship directly with parapet height. This approach greatly simplifies the design method at no loss of accuracy, thereby improving the utility of the method for design practitioners to address a recognized deterrent to broader application of the K-W design method (Smith, 1997). It also avoids inefficiencies (generally overdesign) caused by use of non-dimensional building geometry-based parameters to determine roof design requirements of actual buildings (e.g., wind speed limit, aggregate size, and parapet height).

In addition, the effect of gravel size on critical velocity for blow-off is improved based on the reviewed wind tunnel data which shows a clear relationship between critical velocity for blow-off and the cube-root of the aggregate diameter (not the square-root as used in the K-W design method).

Finally, the modified K-W design method using the above improvements is compared to field observations with sufficient quantitative data (e.g., local wind speed, exposure, aggregate size, parapet height, and building height) available to allow for a meaningful comparison. As a result, a calibration factor is proposed to bring the modified K-W design method in line with observations of successful performance while still maintaining adequate requirements to eradicate clearly problematic observations of aggregate blow-off. Use of such a calibration approach is consistent the RP-4 standard's application of the K-W design method.

REFERENCES:

ASCE (2005). Minimum Design Loads for Buildings and Other Structures. American Society of Civil Engineers, Reston, VA.
Crandell, J. H. (2009) Design of Aggregate Surfaced Roofs to Avoid Aggregate Blow-off: Development of a Modified Kind-Wardlaw Design Method for Buildings of All Heights
International Code Council, Inc. (2006). International Building Code. Falls Church, VA
Kind, R.J. and Wardlaw R.L. (1976). Design of Rooftops Against Gravel Blow-Off. National Aeronautical Establishment, National Research Council, Canada.
Kind, R.J. (1977). Further Wind Tunnel Tests on Building Models to Measure Wind Speeds at Which Gravel is Blown Off Rooftops. LTR-LA-189. National Aeronautical Establishment, National Research Council, Canada.
Minor, J.E. (September 1977). Performance of Roofing Systems in Wind Storms. *Proceedings of the Symposium on Roofing Technology*, National Bureau of Standards and National Roofing Contractors Association.
RICOWI (2007). Hurricane Katrina Investigation Report. Roofing Industry Committee on Weather Issues, Inc., Powder Springs, GA.
Smith, T.L., Kind, R.J., and McDonald, R.J. (1992). Hurricane Hugo: Evaluation of Wind Performance and Wind Design Guidelines for Aggregate Ballasted Single-Ply Membrane Roof Systems. Asociacion Internacional De La Impermeabilizacion, VII Congreso Internacional, Madrid, Spain.
Smith, T.L. (June 1997). Aggregate Blow-Off from BUR and SPF Roofs: Recognizing the Potential Hazards and Avoiding Problems. *Proceedings of The 8th U.S. Conference on Wind Engineering*, AAWE.
SPRI (2008). Wind Design Standard for Ballasted Single-Ply Roofing Systems. ANSI/SPRI RP-4-2008. Single-Ply Roofing Industry, Waltham, MA.

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

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

ICCFILENAME: SMITH-S1-1504.4

S7–09/10
1504.5

Proponent: Mark S. Graham, representing National Roofing Contractors Association (NRCA)

Revise as follows:

1504.5 Edge securement for low-slope roofs. Low-slope ~~membrane~~ built-up, modified bitumen and single-ply roof systems metal edge securement, except gutters, shall be designed and installed for wind loads in accordance with Chapter 16 and tested in accordance with ANSI/SPRI ES-1, except the basic wind speed shall be determined from Figure 1609.

Reason: This proposed code change is intended to add clarity to the code by providing the specific roof membrane types to which Section 1504.5 applies.

The term "…membrane…" is not currently specifically defined in the context of roof systems in Section 1505—Definitions or Chapter 2—Definitions.

The description of roof membranes as "…built-up, modified bitumen and single-ply…" is consistent with other descriptions for membrane-type roof systems already included in Chapter 15.

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

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

ICCFILENAME: GRAHAM-S3-1504.5

S8–09/10
1504.5

Proponent: Mark S. Graham, representing National Roofing Contractors Association (NRCA)

Revise as follows:

1504.5 Edge securement for low-slope roofs. Low-slope membrane roof systems metal edge securement, except gutters, shall be designed and installed for wind loads in accordance with Chapter 16 and tested in accordance with Test Methods RE-1, RE-2 and RE-3 of ANSI/SPRI ES-1, except the basic wind speed shall be determined from Figure 1609.

Reason: This proposed code change is intended to add clarity to the code by providing the specific reference to ANSI/SPRI ES-1's test method requirements (RE-1, RE-2 and RE-3).

ANSI/SPRI ES-1 consists of two primary parts. In the first part the wind loads at a roof edge are determined. In the second part the edge metal flashings' wind resistances are determined according to ANSI/SPRI ES-1's RE-1, RE-2 and RE-3 test methods.

Currently, Section 1504 requires that wind loads be determined according to the code's Chapter 16, not ANSI/SPRI ES-1. Adding specific reference to ANSI/SPRI ES-1's test methods helps clarify that.

This proposed code change is not intended to change the code's current technical requirements; it is only intended to add a specific reference and clarity to which part of ANSI/SPRI ES-1 applies in Section 1504.5.

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

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

ICCFILENAME: GRAHAM-S4-1504.5

S9–09/10
1504.9 (New), Chapter 35

Proponent: Mike Ennis representing Single Ply Roofing Industry (SPRI)

1. Add new text as follows:

1504.9 Roof gardens and landscaped roofs. Roof gardens and landscaped roofs shall comply with Section 1507.16 and shall be installed in accordance with ANSI/SPRI RP14.

2. Add standard to Chapter 35 as follows:

SPRI

RP 14-07   Wind Design Standard for Vegetative Roofing Systems

Reason: Section 1507.16 requires that roof gardens and landscaped roofs comply with the requirements of Chapter 15. Section 1504.1 provides requirements for wind resistance of various roofing assemblies, however no guidance is provided for designing roof gardens and landscaped roofs to withstand wind loads. Roof gardens and landscaped roofs perform in the same manner as ballasted single ply roof assemblies when exposed to wind loads. ANSI/SPRI RP14 is a national consensus standard that has been developed with input from roof membrane manufacturers, component suppliers, contractors, green roofing professionals, testing organizations, and consultants. This design standard is much like the ballast design guide for single-ply roofs currently recognized by the IBC (ANSI/SPRI RP4). It provides the user with a series of tables that define requirements based on design wind speed, building height, parapet height and wind exposure. Three design options are provided. These design options vary in their ability to resist wind loads. Design option 1 uses a 10 lbs/ft2 minimum required load of growth media or trays, Design option 2 also requires minimum 10 lbs/ft2 of growth media or trays in the field of the roof and 13 lbs/ft2 of growth media or interlocking trays or 22 lbs/ft2 of individual trays in the corner and perimeter regions. Design option 3, which is designed for high wind load areas, requires 13 lbs/ft2 of growth media or interlocking trays, or 22 lbs/ft2 of individual trays in the field of the roof and does not allow any loose growth media or trays in the perimeter and corner regions. The perimeter of the building is defined as 40% of the building height. Adjustments are provided to increase the wind resistance of the design based on specific building conditions such as the buildings importance factor, large openings in adjacent walls and rooftop projections to name a few. The standard also provides requirements for newly planted garden roofs that do not have fully developed root systems. Fully developed root systems allow the garden roof assembly to perform very well when exposed to high wind situations, however prior to development of the root system special precautions must be taken.

The basis for the standard includes wind tunnel data generated in support of the ballasted single ply design guide. This wind tunnel testing helped develop an understanding of the impact of particle size and parapet height on the performance of ballasted assemblies. It also provided information regarding the weight of ballast required to keep the roof systems in place at various wind speeds. This data, along with 50-years of garden roof performance data from both the US and Europe were used in the development of this standard.

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, SPRI RP-14 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.

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

ICCFILENAME: ENNIS-S4-1504.9

S10–09/10

Table 1505.1, 1507.16, 1507.16.1 (New)

Proponent: Robert J Davidson, Code Consultant, Alan Shuman, President, representing the National Association of State Fire Marshals (NASFM)

THIS PROPOSAL IS ON THE AGENDA OF THE IBC FIRE SAFETY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THIS COMMITTEE.

Revise as follows:

TABLE 1505.1[a,b, d]
MINIMUM ROOF COVERING CLASSIFICATION
FOR TYPES OF CONSTRUCTION
(No change to table)

(No change to Notes a. through c.)

d. Any exposed portions of roof coverings on roofs containing roof gardens or landscaped roofs shall have their roof covering fire classification increased one level above the level indicated in the table.

1507.16 Roof gardens and landscaped roofs. Roof gardens and landscaped roofs shall comply with the requirements of this chapter and Sections 1607.11.2.2 and 1607.11.2.3 and the International Fire Code.

1507.16.1 Structural fire-resistance. The structural frame and roof construction supporting the load imposed upon the roof by the roof gardens or landscaped roofs shall comply with the requirements of Table 601.

Reason: As rooftop gardens and landscaped roofs gain in acceptance and popularity reasonable requirements need to be added to the codes to address the fuel load that these additions can add to a building or structure.

The addition of a rooftop garden or landscaped roof adds a fuel load to the roof. In recognition of this increased hazard it is proposed that Table 1505.1 be modified by adding a Note d that would require the exposed portions of roof coverings on roofs that contain roof gardens or landscaped roofs have the required classification of the roof covering increased one level. A reference to the International Fire Code is proposed to be added to Section 1507.16.

Table 601 provides for the Fire-Resistance Rating Requirements For Building Elements. Note b of Table 601 states, "

"b. Roof supports: Fire-resistance ratings of structural frame and bearing walls are permitted to be reduced by 1 hour where supporting a roof only."

This reduction is only permitted when the construction elements are "supporting a roof only." The addition of a roof garden or landscaped roof imposes a load and Note b. would not apply. The additional sentence proposed to be added as Section 1507.16.1 is intended to be a pointer to remind the designer and the code official to verify compliance with Table 601 fire-resistance requirements now that a load is planned to be added to the roof structure.

Cost Impact: These requirements will increase the cost of construction for those buildings where a roof garden or landscaped roof is proposed to be installed on the roof.

Analysis: The reference in Section 1507.16 of this proposal to the IFC is dependent on the action on Code Change FXX-09/10 **[Davidson-Shuman-F9-316]** which appears on the hearing order of the IFC Committee and proposes a new IFC *Section 316 - Roof Gardens and Landscaped Roofs*. If that code change is not approved, the reference to the IFC would be deleted from Section 1507.16.

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

ICCFILENAME: DAVIDSON-SHUMAN-S1-1505.1

S11–09/10
Table 1505.1

Proponent: Brian Tollisen, PE, New York State Division of Code Enforcement and Administration, representing the New York State Division of Code Enforcement and Administration

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

Revise as follows:

TABLE 1505.1[a,b]
MINIMUM ROOF COVERING CLASSIFICATION
FOR TYPES OF CONSTRUCTION

IA	IB	IIA	IIB	IIIA	IIIB	IV	VA	VB
B	B	B	C[c]	B	C[c]	B	B[d]	C[c]

(No change to footnotes a through c)

d. Buildings of Type VA construction with a fire separation distance greater than or equal to 30 feet on all sides of the building and equipped throughout with a sprinkler system conforming to Section 903.3.1.1, shall be permitted to have a Class C roof covering classification provided the maximum area does not exceed what is allowed for a Type VB building of the same use and occupancy Classification as determined by Section 506. The total allowable building area shall be determined by multiplying the allowable area per story (A_a), by 3. This area increase shall not be permitted in H-1 and I-2 occupancy classifications.

Reason: Roof covering classifications are used to mitigate the spread of fire from adjoining structures or from wild fires. This proposal requires the fire separation distance to be at a minimum of 30' on all sides. This distance, 30', is the minimum fire separation distance where no fire-resistance ratings for exterior walls are required (IBC Table 602).

This proposal allows buildings of Type VA construction to have a Class C roof covering classification provided the total building area does not exceed what would be allowed for the total building area of a similar use building of Type VB Construction a value of 3 is used to determine the total allowable building area. This allows buildings of constructed of Type 5A construction and with a sprinkler system to have a Class C roof covering classification and the total building area that is at least 33% less than what is typically allowed for the same building with a Class B roof covering classification.

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

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

ICCFILENAME: TOLLISEN-S1-1505.1

S12–09/10
1505.8 (New), Chapter 35

Proponent: Mike Ennis representing the Single Ply Roofing Industry

THIS PROPOSAL IS ON THE AGENDA OF THE IBC FIRE SAFETY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THIS COMMITTEE.

1. Add new text as follows:

<u>**1505.8 Roof gardens and landscaped roofs.** Roof gardens and landscaped roofs shall comply with Section 1507.16 and shall be installed in accordance with ANSI/SPRI VF-1.</u>

2. Add standard to Chapter 35 as follows:

SPRI
<u>VF-1-08</u> <u>Fire Design Standard for Vegetative Roofs</u>

Reason: Section 1507.16 requires that roof gardens and landscaped roofs comply with the requirements of Chapter 15. Section 1505 requires that roofing assemblies be fire classified. The current test procedures used to provide this fire classification are not applicable to garden and landscape roofs due to the many variables (plant types, moisture content, etc.) that exist for these types of systems. ANSI/SPRI VF-1 is a national consensus standard that has been developed with input from roof membrane manufacturers, component suppliers, contractors, green roofing professionals, testing organizations, and consultants. This standard provides a design method to assure an acceptable level of performance of roof gardens and landscaped roofs when exposed to exterior fire sources. The general approach used in this standard is to design in fire breaks for large roof areas, around rooftop equipment and penetrations, and next to adjacent walls. Some of the specific requirements are:

> Exposed membrane areas must conform to the designed fire resistance requirements as determined by the authority having jurisdiction.
> For all vegetated roofing systems abutting combustible vertical surfaces, a Class A (per ASTM E108 or UL790) rated assembly must be achieved for a minimum 6 ft (1.83 m) wide continuous border placed around rooftop structures and all rooftop equipment.
> For large roof areas: Partition the roof area into sections not exceeding 15,625 ft2 (1,450 m^2), with each section having no dimension greater than 125 ft (39 m) by installing a minimum of 3ft. (0.9 m) wide, Class A rated assembly barrier zones.

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, VF-1-08, 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.

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

ICCFILENAME: ENNIS-S5-1505.8

S13–09/10
1505.8 (New)

Proponent: Mark S. Graham, representing National Roofing Contractors Association (NRCA)

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

Add new text as follows:

<u>**1505.8 Photovoltaic systems.** Rooftop installed photovoltaic systems that are adhered or attached to the roof covering shall be labeled to identify their fire classification in accordance with the testing required in Section 1505.1.</u>

Reason: This proposed code change proposal is intended clarify that rooftop photovoltaic systems that are adhered or attached to the roof covering—often referred to as “building integrated photovoltaic (BIPV)”--need to comply with building code requirements for fire classification. The minimum requirement set forth here is intended for the rooftop photovoltaic system to be required to comply with the same minimum fire classification requirements as the underlying roof assembly.

Specific requirements applicable to the electrical portion of rooftop-mounted photovoltaic systems are left to the applicable electrical code.

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

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

ICCFILENAME: GRAHAM-S5-1505.8 NEW

S14–09/10
Table 1507.2.7.1(2); IRC Table R905.2.4.1(2)

Proponent: Mark S. Graham, representing National Roofing Contractors Association (NRCA)

THIS IS A 2 PART PROPOSAL. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

TABLE 1507.2.7.1(2)
CLASSIFICATION OF ASPHALT SHINGLES PER ASTM D3161

MAXIMUM BASIC WIND SPEED FROM FIGURE 1609	CLASSIFICATION REQUIREMENT
85	~~A,~~ D or F
90	~~A,~~D or F
100	~~A, D or~~ F
110	F
120	~~F~~ <u>Not permitted</u>
130	~~F~~ <u>Not permitted</u>
140	~~F~~ <u>Not permitted</u>
150	~~F~~ <u>Not permitted</u>

PART II – IRC BUILDING/ENERGY

Revise as follows:

TABLE R905.2.4.1(2)
CLASSIFICATION OF ASPHALT SHINGLES PER ASTM D3161

MAXIMUM BASIC WIND SPEED FROM FIGURE R301.2(4) (mph)	CLASSIFICATION REQUIREMENT
85	~~A,~~ D or F
90	~~A,~~D or F
100	~~A, D or~~ F
110	F
120	~~F~~ <u>Not permitted</u>
130	~~F~~ <u>Not permitted</u>
140	~~F~~ <u>Not permitted</u>
150	~~F~~ <u>Not permitted</u>

Reason: This proposed code change is intended to make the classification requirements in IBC Table 1507.2.7.1(2) [IRC Table R905.2.4.1(2)] — Classification of Asphalt Shingles per ASTM D3161 consistent with the classifications described in the test method.

In ASTM D3161, asphalt shingles are classified as Class A (60 mph), Class D (90 mph) and Class F (110 mph).

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: GRAHAM-S2- TABLE 1507.2.7.1(2)

S15–09/10

1507.2.8.1, 1507.3.3.3 (New), 1507.4.5 (New), 1507.5.3.1 (New), 1507.6.3.1 (New), 1507.7.3.1 (New), 1507.8.3.1 (New), 1507.9.3.1 (New); IRC R905.2.7.2, R905.3.3.3, R905.4.3.2 (New), R905.5.3.2 (New), R905.6.3.2 (New), R905.7.3.2 (New), R905.8.3.2 (New), R905.10.5.1 (New)

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

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC-STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

1. Revise as follows:

1507.2.8.1 High wind attachment. Underlayment applied in areas subject to high winds [above 110 mph (49 m/s) in accordance with Figure 1609] shall be applied with corrosion-resistant fasteners in accordance with manufacturer's installation instructions. Fasteners are to be applied along the overlap at a maximum spacing of 36 inches (914mm) on center.

Underlayment installed where the basic wind speed equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II, ASTM D 4869 Type IV, or ASTM D 6757. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6 inch (152 mm) spacing at the side laps. Head laps shall be 4 inches (102 mm) and end laps shall be a minimum of 6 inches (152 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

Underlayment installed where the basic wind speed equals or exceeds 140 mph (63 m/s) shall be attached using metal cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

2. Add new text as follows:

1507.3.3.3 High wind attachment. Underlayment applied in areas subject to high wind [over 110 mph (49 m/s) in accordance with Figure 1609] shall be applied with corrosion-resistant fasteners in accordance with manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where the basic wind speed equals or exceeds 120 mph (54 m/s) shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6 inch (152 mm) spacing at the side laps. Head laps shall be 4 inches (102 mm) and end laps shall be a minimum of 6 inches (152 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

Underlayment installed where the basic wind speed equals or exceeds 140 mph (63 m/s) shall be attached using metal cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

1507.4.5 Underlayment and high wind. Underlayment applied in areas subject to high winds [above 110 mph (49 m/s) in accordance with Figure 1609] shall be applied with corrosion-resistant fasteners in accordance with manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where the basic wind speed equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II, ASTM D 4869 Type IV, or ASTM D 1970. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6 inch (152 mm) spacing at the side laps. Head laps shall be 4 inches (102 mm) and end laps shall be a minimum of 6 inches (152 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 5/8 inches (41mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

Underlayment installed where the basic wind speed equals or exceeds 140 mph (63 m/s) shall be attached using metal cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

1507.5.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [above 110 mph (49 m/s) in accordance with Figure 1609] shall be applied with corrosion-resistant fasteners in accordance with manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where the basic wind speed equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6 inch (152 mm) spacing at the side laps. Head laps shall be 4 inches (102 mm) and end laps shall be a minimum of 6 inches (152 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

Underlayment installed where the basic wind speed equals or exceeds 140 mph (63 m/s) shall be attached using metal cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

1507.6.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [above 110 mph (49 m/s) in accordance with Figure 1609] shall be applied with corrosion-resistant fasteners in accordance with manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914mm) on center.

Underlayment installed where the basic wind speed equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6 inch (152 mm) spacing at the side laps. Head laps shall be 4 inches (102 mm) and end laps shall be a minimum of 6 inches (152 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

Underlayment installed where the basic wind speed equals or exceeds 140 mph (63 m/s) shall be attached using metal cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

1507.7.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [above 110 mph (49 m/s) in accordance with Figure 1609] shall be applied with corrosion-resistant fasteners in accordance with manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where the basic wind speed equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6 inch (152 mm) spacing at the side laps. Head laps shall be 4 inches (102 mm) and end laps shall be a minimum of 6 inches (152 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

Underlayment installed where the basic wind speed equals or exceeds 140 mph (63 m/s) shall be attached using metal cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

1507.8.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [above 110 mph (49 m/s) in accordance with Figure 1609] shall be applied with corrosion-resistant fasteners in accordance with manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where the basic wind speed equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches between side laps with a 6 inch (152 mm) spacing at the side laps. Head laps shall be 4 inches (102 mm) and end laps shall be a minimum of 6 inches (152 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

Underlayment installed where the basic wind speed equals or exceeds 140 mph (63 m/s) shall be attached using metal cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

1507.9.3.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [above 110 mph (49 m/s) in accordance with Figure 1609] shall be applied with corrosion-resistant fasteners in accordance with manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where the basic wind speed equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6 inch (152 mm) spacing at the side laps. Head laps shall be 4 inches (102 mm) and end laps shall be a minimum of 6 inches (152 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

Underlayment installed where the basic wind speed equals or exceeds 140 mph (63 m/s) shall be attached using metal cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

PART II – IRC BUILDING/ENERGY

1. Revise as follows:

R905.2.7.2 Underlayment and high wind. Underlayment applied in areas subject to high winds [above 110 mph (49 m/s) per Figure R301.2(4)] shall be applied with corrosion-resistant fasteners in accordance with manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914mm) on center.

Underlayment installed where the basic wind speed equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II, ASTM D 4869 Type IV, or ASTM D 6757. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6 inch (152 mm) spacing at the side laps. Head laps shall be 4 inches (102 mm) and end laps shall be a minimum of 6 inches (152 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

Underlayment installed where the basic wind speed equals or exceeds 140 mph (63 m/s) shall be attached using metal cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

R905.3.3.3 Underlayment and high wind. Underlayment applied in areas subject to high wind [over 110 miles per hour (49 m/s) per R301.2(4)] shall be applied with corrosion-resistant fasteners in accordance with manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where the basic wind speed equals or exceeds 120 mph (54 m/s) shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6 inch (152 mm) spacing at the side laps. Head laps shall be 4 inches (102 mm) and end laps shall be a minimum of 6 inches (152 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

Underlayment installed where the basic wind speed equals or exceeds 140 mph (63 m/s) shall be attached using metal cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

2. Add new text as follows:

R905.4.3.2 Underlayment and high wind. Underlayment applied in areas subject to high winds [above 110 mph (49 m/s) per Figure R301.2(4)] shall be applied with corrosion-resistant fasteners in accordance with manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where the basic wind speed equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II, ASTM D 4869 Type IV, or ASTM D 1970. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6 inch (152 mm) spacing at the side laps. Head laps shall be 4 inches (102 mm) and end laps shall be a minimum of 6 inches (152 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

Underlayment installed where the basic wind speed equals or exceeds 140 mph (63 m/s) shall be attached using metal cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

R905.5.3.2 Underlayment and high wind. Underlayment applied in areas subject to high winds [above 110 mph (49 m/s) per Figure R301.2(4)] shall be applied with corrosion-resistant fasteners in accordance with manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where the basic wind speed equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6 inch (152 mm) spacing at the side laps. Head laps shall be 4 inches (102 mm) and end laps shall be a minimum of 6 inches (152 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

Underlayment installed where the basic wind speed equals or exceeds 140 mph (63 m/s) shall be attached using metal cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

R905.6.3.2 Underlayment and high wind. Underlayment applied in areas subject to high winds [above 110 mph (49 m/s) per Figure R301.2(4)] shall be applied with corrosion-resistant fasteners in accordance with manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where the basic wind speed equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6 inch (152 mm) spacing at the side laps. Head laps shall be 4 inches (102 mm) and end laps shall be a minimum of 6 inches (152 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

Underlayment installed where the basic wind speed equals or exceeds 140 mph (63 m/s) shall be attached using metal cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

R905.7.3.2 Underlayment and high wind. Underlayment applied in areas subject to high winds [above 110 mph (49 m/s) per Figure R301.2(4)] shall be applied with corrosion-resistant fasteners in accordance with manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914mm) on center.

Underlayment installed where the basic wind speed equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6 inch (152 mm) spacing at the side laps. Head laps shall be 4 inches (102 mm) and end laps shall be a minimum of 6 inches (152 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

Underlayment installed where the basic wind speed equals or exceeds 140 mph (63 m/s) shall be attached using metal cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

R905.8.3.2 Underlayment and high wind. Underlayment applied in areas subject to high winds [above 110 mph (49 m/s) per Figure R301.2(4)] shall be applied with corrosion-resistant fasteners in accordance with manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where the basic wind speed equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II or ASTM D 4869 Type IV. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6 inch (152 mm) spacing at the side laps. Head laps shall be 4 inches (102 mm) and end laps shall be a minimum of 6 inches (152 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

Underlayment installed where the basic wind speed equals or exceeds 140 mph (63 m/s) shall be attached using metal cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

R905.10.5.1 Underlayment and high wind. Underlayment applied in areas subject to high winds [above 110 mph (49 m/s) per Figure R301.2(4)] shall be applied with corrosion-resistant fasteners in accordance with manufacturer's installation instructions. Fasteners are to be applied along the overlap not farther apart than 36 inches (914 mm) on center.

Underlayment installed where the basic wind speed equals or exceeds 120 mph (54 m/s) shall comply with ASTM D 226 Type II. The underlayment shall be attached in a grid pattern of 12 inches (305 mm) between side laps with a 6 inch (152 mm) spacing at the side laps. Head laps shall be 4 inches (102 mm) and end laps shall be a minimum of 6 inches (152 mm). Underlayment shall be attached using metal or plastic cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

Underlayment installed where the basic wind speed equals or exceeds 140 mph (63 m/s) shall be attached using metal cap nails with a head diameter of not less than 1 5/8 inches (41 mm) with a shank of at least 32 gauge sheet metal with a length to penetrate through the roof sheathing.

Reason: Observations of roof underlayment performance following Hurricane Ike in Texas and in two sets of tests conducted at the University of Florida and Florida International University demonstrated that relatively new and new ASTM 226 Type I underlayments performed very poorly when subjected to wind over about 110 mph. In the laboratory tests, specimen covered with ASTM 226 Type I and Type II underlayments performed dramatically differently. ASTM Type I felt (15#) material completely blew off some portions of the specimen as winds exceeded 110 mph and pulled over the plastic caps on other parts of the specimen. In contrast, the ASTM 226 Type II (30#) material remained in place and showed very few signs of distress. Plastic caps deformed much more than the metal caps in several installations. Consequently, the use of metal caps is recommended for areas with the highest basic design wind speeds.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: Stafford-S3-1507.2.8.1

S16–09/10
1507.2.9.3; IRC R905.2.8.5 (New)

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

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC-STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

1507.2.9.3 Drip edge. Provide drip edge at eaves and gables of shingle roofs. Overlap to be a minimum of 2 inches (51 mm). Eave drip edges shall extend 1/4 inch (6.4 mm) below sheathing and the flange that rests on top of the sheathing shall extend back on the roof a minimum of 2 inches (51 mm). Drip edge shall be mechanically fastened a maximum of 12 inches (305 mm) o.c. Where the basic wind speed in Figure 1609 is 110 mph (49 m/s) or greater, drip edges shall be mechanically fastened a maximum of 4 inches (102 mm) on center using a stagger pattern such that adjacent fasteners along the length of the drip edge are placed towards opposite sides of the flange. Shingles shall not extend beyond the outer edges of the drip edge by more than ¼ inch (6.4 mm).

PART II – IRC BUILDING/ENERGY

Add new text as follows:

R905.2.8.5 Drip edge. Where a drip edge is provided at eaves and gables of shingle roofs, the installation shall be in accordance with this section. Overlap shall be a minimum of 2 inches (51 mm). Eave drip edges shall extend 1/4 inch (6.4 mm) below sheathing and the flange that rests on the top of the roof sheathing shall extend back on the roof a minimum of 2 inches (51 mm). Drip edge shall be mechanically fastened a maximum of 12 inches (305 mm) o.c. Where the basic wind speed in Figure 1609 is 110 mph (49 m/s) or greater, drip edges shall be mechanically fastened a maximum of 4 inches (102 mm) on center using a stagger pattern such that adjacent fasteners along the length of the drip edge are placed towards opposite sides of the flange. Shingles shall not extend beyond the outer edges of the drip edge or the roof edge by more than ¼ inch (6.4 mm).

Reason: (IBC) The purpose of this proposal is to address two issues related to the wind performance of roofs. Recent hurricanes revealed that even in low wind speeds, the vertical part of the drip edge can rotate upward when subjected to wind, thus triggering the blow off of shingles. Sometimes this resulted in a cascading loss of shingles up or across a roof. By placing the fasteners near the edge of the roof, the potential for rotation of the drip edge is greatly reduced. Shingles that extend significantly beyond the edge of roofs are susceptible to being blown off in high winds. Currently the code does not limit this extension. While some extension is warranted to ensure that water sheds directly into the gutter, too much extension subjects the shingle potentially higher wind loads than what the shingle is capable of resisting.

(IRC)The purpose of this proposal is to address two issues related to the wind performance of roofs. While the IRC doesn't require the installation of a drip edge, improper installation can lead to problems in high wind areas. Recent hurricanes revealed that even in low wind speeds, the vertical part of the drip edge can rotate upward when subjected to wind, thus triggering the blow off of shingles. Sometimes this resulted in a cascading loss of shingles up or across a roof. By placing the fasteners near the edge of the roof, the potential for rotation of the drip edge is greatly reduced. Shingles that extend significantly beyond the edge of roofs are susceptible to being blown off in high winds. Currently the code does not limit this extension. While some extension is warranted to ensure that water sheds directly into the gutter, too much extension subjects the shingle to potentially higher wind loads than what the shingle is capable of resisting.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: Stafford-S2-1507.2.9.3

S17–09/10
1507.5.5.1 (New); IRC R905.4.5.1 (New)

Proponent: Bob Eugene, Underwriters Laboratories Inc

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC-STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Add new text as follows:

1507.5.5.1 Wind resistance. Formed metal roof shingles shall be tested in accordance with procedures adapted from ASTM D 3161. Formed metal roof shingles shall meet the classification requirements of Table 1507.2.7.1(2) for the appropriate maximum basic wind speed. Formed metal roof shingle packaging shall bear a label to indicate compliance with the procedures adapted from ASTM D 3161 and the required classification from Table 1507.2.7.1(2).

PART II – IRC BUILDING/ENERGY

Add new text as follows:

R905.4.5.1 Wind resistance. Formed metal roof shingles shall be tested in accordance with procedures adapted from ASTM D 3161. Metal shingles shall meet the classification requirements of Table R905.2.4.1 (2) for the appropriate maximum basic wind speed. Metal shingle packaging shall bear a label to indicate compliance with the procedures adapted from ASTM D 3161 and the required classification from Table R905.2.4.1 (2).

Reason: (IBC) Metal roof shingles are susceptible to wind. The procedures used in ASTM D 3161 for asphalt shingles are appropriate to use for determining wind resistance, when adapted for testing these types of shingles.
(IRC) This proposal introduces a new section covering wind resistance for metal roof shingles used in residential applications. The text is from IBC Section 1507.5, with an additional requirement for wind resistance (proposed Section R905.4.5.1). Metal roof shingles are susceptible to wind. The procedures used in ASTM D3161 for asphalt shingles are appropriate to use for determining wind resistance, when adapted for testing these types of shingles.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: EUGENE-S4-1507.5.5.1

S18–09/10
1507.10.2, Chapter 35; IRC R905.9.2, Chapter 44

Proponent: Bob Eugene, representing Underwriters Laboratories Inc

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

1. Revise as follows:

1507.10.2 Material standards. Built-up roof covering materials shall comply with the standards in Table 1507.10.2 <u>or UL 55A</u>.

2. Add standard to Chapter 35 as follows:

UL
<u>55A-04 Materials for Built-Up Roof Coverings</u>

PART II – IRC BUILDING/ENERGY

1. Revise as follows:

R905.9.2 Material standards. Built-up roof covering materials shall comply with the standards in Table R905.9.2 <u>or UL 55A</u>.

2. Add standard to Chapter 44 as follows:

UL
<u>55A-04 Materials for Built-Up Roof Coverings</u>

Reason: UL 55A has been in use since 1919, and is still used to evaluate the following materials used in the construction of built-up roof coverings – hot-mopping asphalt, asphalt-saturated and organic felt, coal-tar pitch, coal-tar saturated organic felt, and asphalt-coated glass-fire mat (felt). Several of these materials referenced in listed Roofing Systems evaluated to UL 790 or ASTM E 108 are based on compliance with UL 55A; and therefore UL55A should be referenced as an alternate standard in this code section.

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, UL 55A-04, 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.

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: EUGENE-S5-1507.10.2

S19–09/10
Table 1507.10.2, Chapter 35

Proponent: Michael D. Fischer, The Kellen Company, representing the Asphalt Roofing Manufacturers Association

1. Revise as follows:

TABLE 1507.10.2
BUILT-UP ROOFING MATERIAL STANDARDS

MATERIAL STANDARD	STANDARD
Asphalt coatings used in roofing	ASTM D1227; D 2823 <u>D2824;</u> D4479

(Portions of Table not shown, remain unchanged)

2. Add standard to Chapter 35 as follows:

ASTM International

<u>D 2824-06</u>	<u>Standard Specification for Aluminum-Pigmented Asphalt Roof Coatings, Nonfibered, Asbestos Fibered, and Fibered without Asbestos</u>

Reason: This proposal adds ASTM D2824 to the list of material standards used in BUR systems. The standard is included in the IRC companion table (Table R905.9.2) and should be included in the IBC in order to streamline product approval.

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

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM D2824-06, 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.

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

ICCFILENAME: FISCHER-S1-1507.5.2

S20–09/10

1507.14.3, Table 1507.14.3 (New); IRC R905.14.3, Table R905.14.3 (New)

Proponent: Mark S. Graham, representing National Roofing Contractors Association (NRCA)

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

1507.14.3 Application. Foamed-in-place roof insulation shall be installed in accordance with the manufacturer's instructions. A liquid-applied protective coating that complies with ~~Section 1507.15~~ <u>Table 1507.14.3</u> shall be applied no less than 2 hours nor more than 72 hours following the application of the foam.

<u>TABLE 1507.14.3</u>
<u>PROTECTIVE COATING MATERIAL STANDARDS</u>

<u>MATERIAL</u>	<u>STANDARD</u>
<u>Acrylic coating</u>	<u>ASTM D 6083</u>
<u>Silicone coating</u>	<u>ASTM D 6694</u>
<u>Moisture cured polyurethane coating</u>	<u>ASTM D 6947</u>

PART II – IRC BUILDING/ENERGY

Revise as follows:

R905.14.3 Application. Foamed-in-place roof insulation shall be installed in accordance with this chapter and the manufacturer's instructions. A liquid-applied protective coating that complies with ~~Section R905.15~~ <u>Table R905.14.3</u> shall be applied no less than 2 hours nor more than 72 hours following the application of the foam.

<u>TABLE R905.14.3</u>
<u>PROTECTIVE COATING MATERIAL STANDARDS</u>

<u>MATERIAL</u>	<u>STANDARD</u>
<u>Acrylic coating</u>	<u>ASTM D 6083</u>
<u>Silicone coating</u>	<u>ASTM D 6694</u>
<u>Moisture cured polyurethane coating</u>	<u>ASTM D 6947</u>

Reason: This proposed code change is intended to clarify the code by adding a table within Section 1507.14.3 that identifies the specific protective coating materials that are applicable to sprayed polyurethane foam roof systems.

The specific coating materials' ASTM standards that are included in this new table are already included in the code in Sec. 1507.15.2. Inclusion of all of the standards from Section 1507.15.2 is not appropriate here because some of these materials are not suitable for spray polyurethane foam roof systems.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: GRAHAM-S6-1507.14.3

S21–09/10

1507.15, 1507.15.1, 1507.15.2; IRC R905.15, R905.15.1, R905.15.2, R905.15.3

Proponent: Mark S. Graham, representing National Roofing Contractors Association (NRCA)

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDIING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDER FOR THESE COMMITTEES.

PART I- IBC STRUCTURAL

Revise as follows:

1507.15 Liquid-applied ~~coatings~~ <u>roofing.</u> The installation of liquid-applied ~~coatings~~ <u>roofing</u> shall comply with the provisions of this section.

1507.15.1 Slope. Liquid-applied r~~oofs~~ <u>roofing</u> shall have a design slope of a minimum of one-fourth unit vertical in 12 units horizontal (2-percent slope).

1507.15.2 Material standards. Liquid-applied roof<u>ing</u> ~~coatings~~ shall comply with ASTM C 836, ASTM C 957, ASTM D 1227 or ASTM D 3468, ASTM D 6083, ASTM D 6694 or ASTM D 6947.

PART II- IRC BUILDING/ENERGY

Revise as follows:

R905.15 Liquid-applied ~~coatings~~ <u>roofing.</u> The installation of liquid-applied ~~coatings~~ <u>roofing</u> shall comply with the provisions of this section.

R905.15.1 Slope. Liquid-applied ~~roofs~~ <u>roofing</u> shall have a design slope of a minimum of one-fourth unit vertical in 12 units horizontal (2-percent slope).

R905.15.2 Material standards. Liquid-applied ~~roof~~ <u>roofing</u> ~~coatings~~ shall comply with ASTM C 836, ASTM C 957, ASTM D 1227 or ASTM D 3468, ASTM D 6083, ASTM D 6694 or ASTM D 6947.

R905.15.3 Application. Liquid-applied ~~roof~~ <u>roofing</u> ~~coatings~~ shall be installed according to this chapter and the manufacturer's installation instructions

Reason: This proposed code change is intended to clarify the code by adding terminology that is appropriate for this section and applicable to the referenced material standards.

This section addresses those materials liquid-applied products that form a waterproof barrier that can serve as a roof covering. When serving the function as a roof covering, these products are not necessarily a "coating."

This proposed code change is not intended to change the code's current technical requirements for the products descried in this section.

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

PART I- IBC STRUCTURAL

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

PART II- IRC BUILDING/ENERGY

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

ICCFILENAME: GRAHAM-S7-1507.15 –RB-3-R905.15

S22–09/10

1502.1, 1507.17 (New), 1509.6 (New), Chapter 35; IRC R202, R905.16 (New), Chapter 44

Proponent: Bob Eugene, representing Underwriters Laboratories Inc

THIS IS A 3 PART CODE CHANGE. PARTS I & II WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART III WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

1. Add definition as follows:

1502.1 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.

PHOTOVOLTAIC MODULES/SHINGLES. A roof covering composed of flat-plate photovoltaic modules fabricated in sheets that resemble three-tab composite shingles.

2. Add new text as follows:

1507.17 Photovoltaic modules/shingles. The installation of photovoltaic modules/shingles shall comply with the provisions of this section.

1507.17.1 Material standards. Photovoltaic modules/shingles shall be listed and labeled in accordance with UL1703.

1507.17.2 Attachment. Photovoltaic modules/shingles shall be attached in accordance with the manufacturer's installation instructions.

1507.17.3 Wind resistance. Photovoltaic modules/shingles shall be tested in accordance with procedures adapted from ASTM D 3161. Photovoltaic modules/shingles shall comply with the classification requirements of Table 1507.2.7.1(2) for the appropriate maximum basic wind speed. Photovoltaic modules/shingle packaging shall bear a label to indicate compliance with the procedures adapted from ASTM D 3161 and the required classification from Table 1507.2.7.1(2).

3. Add standard to Chapter 35 as follows:

UL
1703-02 Flat-Plate Photovoltaic Modules and Panels – with revisions through April 2008

PART II – IBC STRUCTURAL

Add new text as follows:

1509.6 Photovoltaic panels and modules. Photovoltaic panels and modules mounted on top of a roof shall be listed and labeled in accordance with UL 1703 and shall be installed in accordance with the manufacturer's installation instructions.

PART III – IRC BUILDING/ENERGY

1. Add definition as follows:

SECTION R202
DEFINITIONS

PHOTOVOLTAIC MODULES/SHINGLES. A roof covering composed of flat-plate photovoltaic modules fabricated in sheets that resemble three-tab composite shingles.

2. Add new text as follows:

R905.16 Photovoltaic modules/shingles. The installation of photovoltaic modules/shingles shall comply with the provisions of this section.

R905.16.1 Material standards. Photovoltaic modules/shingles shall be listed and labeled in accordance with UL 1703.

R905.16.2 Attachment. Photovoltaic modules/shingles shall be attached in accordance with the manufacturer's installation instructions.

R905.16.3 Wind resistance. Photovoltaic modules/shingles shall be tested in accordance with procedures adapted from ASTM D 3161. Photovoltaic modules/shingles shall comply with the classification requirements of Table R905.2.4.1(2) for the appropriate maximum basic wind speed. Photovoltaic modules/shingle packaging shall bear a label to indicate compliance with the procedures adapted from ASTM D 3161 and the required classification from Table R905.2.4.1(2).

3. Add standard to Chapter 44 as follows:

UL
1703-02 Flat-Plate Photovoltaic Modules and Panels – with revisions through April 2008

Reason: (Parts I & III)The proposal provides guidance for installers and code officials regarding the installation of photovoltaic modules/shingles. These shingles are integrated with the building, and provide both a roof covering and source of electrical power. UL 1703 is the standard used to investigate and list photovoltaic modules. The appropriate design slope and fastening of the shingles are different for each manufacturer's product. For wind resistance, the procedures used in ASTM D 3161 for asphalt shingles are appropriate to use, when adapted for these types of shingles. Several companies currently have listings for these products.
(Part II) The ever increasing number of installations of photovoltaic panels and modules raises concerns about the safety of these installations. This proposal requires these products to be listed and installed in accordance with the manufacturer's instructions. UL 1703 is the standard used to investigate photovoltaic modules and panels, and includes construction and performance requirements to address potential safety hazards. Over 60 companies currently have UL 1703 listings for photovoltaic modules and panels.

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, UL 1703-02, 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.

PARTS I & II – IBC STRUCTURAL

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

PART III – IRC BUILDING/ENERGY

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

ICCFILENAME: EUGENE-S1-1507.17

S23–09/10
1507.17 (New); IRC R905.16 (New)

Proponent: Bob Eugene, representing Underwriters Laboratories Inc.

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Add new text as follows:

1507.17 Formed plastic shingles. The installation of formed plastic shingles shall comply with the provisions of this section.

1507.17.1 Attachment. Plastic shingles shall be attached as required by the manufacturer.

1507.1.1 Wind resistance. Plastic shingles shall be tested in accordance with procedures adapted from ASTM D 3161.Plastic shingles shall comply with the classification requirements of Table 1507.2.7.1(1) for the appropriate maximum basic wind speed. Plastic shingle packaging shall bear a label to indicate compliance with the procedures adapted from ASTM D 3161 and the required classification from Table 1507.2.7.1(2).

PART II – IRC BUILDING/ENERGY

Add new text as follows:

R905.16 Formed plastic shingles. The installation of formed plastic shingles shall comply with the provisions of this section.

R905.16.1 Attachment. Plastic shingles shall be attached as required by the manufacturer.

R905.16.1.1 Wind resistance. Plastic shingles shall be tested in accordance with procedures adapted from ASTM D 3161. Plastic shingles shall meet the classification requirements of Table R905.2.4.1 (2) for the appropriate maximum basic wind speed. Plastic shingle packaging shall bear a label to indicate compliance with the procedures adapted from ASTM D 3161 and the required classification from Table R905.2.4.1 (2).

Reason: The proposal provides guidance for installers and code officials regarding the installation of formed plastic shingles. The appropriate design slope and fastening of the shingles are different for each manufacturer's product. For wind resistance, the procedures used in ASTM D 3161 for asphalt shingles are appropriate to use, when adapted for these types of shingles.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: EUGENE-S2-1507.17

S24–09/10
1508.1

Proponent: Mike Ennis representing Single Ply Roofing Industry (SPRI)

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

Revise as follows:

1508.1 General. The use of above-deck thermal insulation shall be permitted provided such insulation is covered with an approved roof covering and either passes the tests of FM 4450 or UL 1256 when tested as an assembly or is separated from the interior of the building by an approved thermal barrier in accordance with Section 2603.4.

Exceptions~~s~~:

~~1. Foam plastic roof insulation shall conform to the material and installation requirements of Chapter 26.~~
~~2~~ Where a concrete roof deck is used and the above-deck thermal insulation is covered with an approved roof covering.

Reason: The proposed wording clarifies requirements for the use of above deck insulation by providing testing options (FM4450 or UL1256) and the installation option (thermal barrier) within the text of Section 1508.1. Chapter 26 of the IBC currently recognizes that thermal barriers provide adequate protection for the use of foam plastic insulation (Section 2603.4). Thermal barriers will also provide adequate protection for other insulation types used in this application. Other commonly used types of insulation for this application are fiberglass, cellular fiber, mineral fiber, perlite and wood fiberboard. By making this change options for including above deck insulation are clearly spelled out, an assembly that has passed UL1256 or FM 4450 must be used, or a thermal barrier must be installed.

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

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

ICCFILENAME: ENNIS-S3-1508.1

S25–09/10

Table 1508.2, Chapter 35

THIS PROPOSAL IS ON THE AGENDA OF THE IBC FIRE SAFETY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THIS COMMITTEE.

Proponent: Tony Crimi, AC Consulting Solutions Inc., representing North American Insulation Manufacturers Association

1. Revise as follows:

TABLE 1508.2
MATERIAL STANDARDS FOR ROOF INSULATION

Cellular glass board	ASTM C 552
Composite boards	ASTM C 1289, Type III, IV, V or VI
Expanded polystyrene	ASTM C 578
Extruded polystyrene board	ASTM C 578
Perlite board	ASTM C 728
Polyisocyanurate board	ASTM C 1289, Type I or Type II
Wood fiberboard	ASTM C 208
Mineral Fiber Insulation board	ASTM C 726

2. Add standard to Chapter 35 as follows:

ASTM International
C 726-05e1 Standard Specification for Mineral Fiber Roof Insulation Board

Reason: To add the current ASTM C 726 specification for the composition and physical properties of mineral fiber insulation board used above structural roof decks as a base for built-up roofing and single ply membrane systems in building construction.
This specification covers the composition and physical properties of mineral fiber insulation board used above structural roof decks as a base for built-up roofing and single ply membrane systems in building construction.

The current table in section 1508.2 does not include reference to the appropriate ASTM Standard specification for the composition and physical properties of mineral fiber insulation board used above structural roof decks as a base for built-up roofing and single ply membrane systems in materials in the Table. This Standard covers testing and conformance to the following physical properties: compressive resistance, tensile strength, breaking load strength, water absorption, response to thermal and humid aging, linear dimensional change, thermal resistance, and dimensions.

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

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM C726-05e1, 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.

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

ICCFILENAME: CRIMI-S2-TABLE 1508.2

S26–09/10
1509.1, 1509.2.4

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

THIS PROPOSAL IS ON THE AGENDA OF THE IBC GENERAL COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THIS COMMITTEE.

Revise as follows:

1509.1 General. The provisions of this section shall govern the construction of rooftop structures <u>and of rooftop mounted enclosures such as mechanical equipment screens.</u>

1509.2.4 Type of construction. Penthouses <u>and other rooftop enclosures</u> shall be constructed with walls, floors and roof as required for the building.

Exceptions:

1. On buildings of Type I construction, the exterior walls and roofs of penthouses with *a fire separation distance* of more than 5 feet (1524 mm) and less than 20 feet (6096 mm) shall be of at least 1-hour fire resistance rated noncombustible construction. Walls and roofs with a *fire separation distance* of 20 feet (6096 mm) or greater shall be of noncombustible construction. Interior framing and walls shall be of noncombustible construction.
2. On buildings of Type I construction two stories above *grade plane* or less in height and Type II construction, the exterior walls and roofs of penthouses with a *fire separation distance* of more than 5 feet (1524 mm) and less than 20 feet (6096 mm) shall be of at least 1-hour fire-resistance-rated noncombustible *or fire-retardant-treated wood* construction. Walls and roofs with *a fire separation distance* of 20 feet (6096 mm) or greater shall be of noncombustible or *fire-retardant-treated wood* construction. Interior framing and walls shall be of noncombustible or fire retardant- treated wood construction.
3. On buildings of Type III, IV and V construction, the exterior walls of penthouses with a *fire separation distance* of more than 5 feet (1524 mm) and less than 20 feet (6096 mm) shall be at least 1-hour fire resistance- rated construction. Walls with a *fire separation distance* of 20 feet (6096 mm) or greater from a common property line shall be of Type IV construction or noncombustible, *or fire-retardant-treated wood* construction. Roofs shall be constructed of materials and fire-resistance rated as required in Table 601 and Section 603 Item 25.3. Interior framing and walls shall be Type IV construction or noncombustible or *fire-retardant-treated wood* construction.
4. On buildings of Type I construction, unprotected noncombustible enclosures <u>including screens</u> housing only mechanical equipment and located with a minimum *fire separation distance* of 20 feet (6096 mm) shall be permitted.
5. On buildings of Type I construction two stories or less above *grade plane* in height, or Type II, III, IV, and V construction, unprotected noncombustible or *fire-retardant-treated wood* enclosures <u>including screens</u> housing only mechanical equipment and located with a *minimum fire separation distance* of 20 feet (6096 mm) shall be permitted.
6. On one-story buildings, combustible unroofed mechanical equipment screens, fences or similar enclosures are permitted where located with a *fire separation distance* of at least 20 feet (6096 mm) from adjacent property lines and where not exceeding 4 feet (1219 mm) in height above the roof surface.
7. Dormers shall be of the same type of construction as the roof on which they are placed, or of the exterior walls of the building.

Reason: The provisions of Section 1509 include more than just "*rooftop structures*" that are defined in Section 1502 as "*an enclosed structure*", such as a penthouse. The section currently contains provisions for unroofed mechanical equipment screens and towers that may be unenclosed. As a result, there have been disagreements between code enforcers and designers regarding the application of fire resistance rules specified within the exceptions to Section 1509.2.4, to *unenclosed* rooftop structures such as mechanical equipment screens. To address this, the scope statement in Section 1509.1 is revised to specifically add rooftop mounted enclosures such as mechanical equipment screens.

Current exceptions 4, 5 and 6 to Section 1509.2.4 address more than just penthouses. According to written interpretations from ICC staff (Paul Wong), these exceptions are intended to address the necessary fire resistance of mechanical equipment screens that are unenclosed rooftop structures. To clarify this intent, the wording in Exceptions 4 and 5 is revised to state "*enclosures including screens*". The wording in exception 6 is not revised because it is clear that it applies to roof screens as currently written. Exception 6 however is limited to screens having a maximum height of 4'-0", and many jurisdictions require taller screens to hide roof mounted HVAC equipment. As a result exceptions 4 and 5 provide rules that can be applied to those taller screens.

Each of these changes in intended to clarify that unenclosed roof screens are specifically included in the provisions.

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

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

ICCFILENAME: MAIEL-S2-1509.1

S27–09/10

1509.2-1509.5.2, 1509.6-1509.7.5 (New)

Proponent: Rick Thornberry, PE, The Code Consortium, Inc., representing Alcan Composites USA, Inc. and Jesse J. Beitel, Hughes Associates, Inc., representing, Trespa North America, Ltd.

THIS PROPOSAL IS ON THE AGENDA OF THE IBC GENERAL COMMITTEE. SEE THE TENTATIVE HEARING ORDER FOR THIS COMMITTEE.

1. Revise as follows:

1509.1 General. The provisions of this section shall govern the construction of rooftop structures.

1509.2 Penthouses. ~~A penthouse or~~ Penthouses in compliance with Sections 1509.2.1 through ~~1509.2.4~~ 1509.2.5 shall be considered as a portion of the story directly below the roof deck on which such penthouses are located. All other penthouses shall be considered as an additional story of the building.

1509.2.1 Height above roof deck. ~~A penthouse~~ Penthouses ~~or other projection above the roof in structures~~ constructed on buildings of other than Type I construction ~~shall not exceed 28 feet (8534 mm) above the roof where used as an enclosure for tanks or for elevators that run to the roof and in all other cases~~ shall not exceed ~~extend more than~~ 18 feet (5486 mm) in height above the roof deck as measured to the average height of the roof of the penthouse.

Exceptions:

1. Where used to enclose tanks or elevators that travel to the roof level, penthouses shall be permitted to have a maximum height of 28 feet (8534 mm) above the roof deck.
2. Penthouses located on the roof of buildings of Type I construction shall not be limited in height.

1509.2.2 Area limitation. The aggregate area of penthouses and other enclosed rooftop structures shall not exceed one-third the area of the supporting roof deck. Such penthouses and other enclosed rooftop structures shall not be required to be included in determining ~~contribute to either~~ the building area or number of stories as regulated by Section 503.1. The area of ~~the penthouse~~ such penthouses shall not be included in determining the fire area ~~defined~~ specified in Section 901.7 ~~902~~.

1509.2.3 Use limitations. ~~A penthouse~~ Penthouses~~, bulkhead or any other similar projection above the roof~~ shall not be used for purposes other than the shelter of mechanical or electrical equipment or ~~shelter of~~ vertical shaft openings in the roof assembly.

1509.2.4 Weather protection. Provisions such as louvers, louver blades or flashing shall be made to protect the mechanical and electrical equipment and the building interior from the elements. ~~Penthouses or bulkheads used for purposes other than permitted by this section shall conform to the requirements of this code for an additional story. The restrictions of this section shall not prohibit the placing of wood flagpoles or similar structures on the roof of any building.~~

~~1509.2.4~~ **1509.2.5 Type of construction.** Penthouses shall be constructed with walls, floors and roof as required for the type of construction of the building on which such penthouses are built.

Exceptions:

1. On buildings of Type I construction, the exterior walls and roofs of penthouses with a fire separation distance ~~of more~~ greater than 5 feet (1524 mm) and less than 20 feet (6096 mm) shall be permitted to have not less than a ~~of at least~~ 1-hour fire resistance-rating ~~rated noncombustible construction~~. The exterior walls and roofs of penthouses with a fire separation distance of 20 feet (6096 mm) or greater shall ~~be of noncombustible~~ ~~construction~~ not be required to have a fire-resistance rating. ~~Interior framing and walls shall be of noncombustible construction.~~
2. On buildings of Type I construction two stories or less in height above grade plane or ~~less in height and~~ Type II construction, the exterior walls and roofs of penthouses with a fire separation distance ~~of more~~ greater than 5 feet (1524 mm) and less than 20 feet (6096 mm) shall be permitted to have not less than a ~~of at least~~ 1-hour fire-resistance-rating ~~rated noncombustible or~~ and be constructed of fire-retardant-treated wood ~~construction~~. The exterior walls and roofs of penthouses with a fire separation distance of 20 feet (6096 mm) or greater shall be permitted to be constructed of ~~noncombustible~~ ~~or~~ fire-retardant-treated wood ~~construction~~ and shall not be required to have a fire-resistance rating. Interior framing and walls shall be permitted to be constructed of ~~noncombustible or~~ fire retardant treated wood ~~construction~~.
3. On buildings of Type III, IV or ~~and~~ V construction, the exterior walls of penthouses with a fire separation distance ~~of more~~ greater than 5 feet (1524 mm) and less than 20 feet (6096 mm) shall be permitted to have not less than a ~~at least~~ 1-hour fire-resistance-rating or a lesser fire-resistance rating as required by Table 602 ~~rated construction~~. The exterior walls of penthouses with a fire separation distance of 20 feet (6096 mm) or greater ~~from a common property line~~ shall be of Type IV construction, or shall be constructed of noncombustible~~,~~ or fire-retardant-treated wood, ~~construction~~ and shall not be required to have a fire-resistance rating. ~~Roofs shall be constructed of materials and fire-resistance rated as required in Table 601 and Section 603, Item 25.3. Interior framing and walls shall be Type IV construction or noncombustible or fire-retardant-treated wood construction.~~
~~4. On buildings of Type I construction, unprotected noncombustible enclosures housing only mechanical equipment and located with a minimum fire separation distance of 20 feet (6096 mm) shall be permitted.~~

4 ~~5~~. On buildings of ~~Type I construction two stories or less above grade plane in height, or~~ Type ~~II,~~ III, or IV ~~and V~~ construction, ~~unprotected~~ penthouses constructed of noncombustible materials or fire-retardant-treated wood, ~~enclosures~~ housing only mechanical equipment, and located with a ~~minimum~~ fire separation distance ~~of~~ not less than 20 feet (6096 mm) shall be permitted and shall not be required to be fire-resistance-rated.

~~6. On one-story buildings, combustible unroofed mechanical equipment screens, fences or similar enclosures are permitted where located with a fire separation distance of at least 20 feet (6096 mm) from adjacent property lines and where not exceeding 4 feet (1219 mm) in height above the roof surface.~~

~~7. Dormers shall be of the same type of construction as the roof on which they are placed, or of the exterior walls of the building.~~

1509.3 Tanks. Tanks having a capacity of more than 500 gallons (2 m^3) ~~placed in or~~ located on the roof deck of a building shall be supported on masonry, reinforced concrete, steel or Type IV construction provided that, where such supports are located in the building above the lowest story, the support shall be fire-resistance rated as required for Type IA construction.

1509.3.1 Valve and drain. ~~Such tanks shall have~~ In the bottom or on the side near the bottom of the tank, a pipe or outlet~~,~~ fitted with a suitable quick opening valve for discharging the contents in an emergency into a ~~through an adequate~~ drain shall be provided.

1509.3.2 Location. ~~Such~~ Tanks shall not be placed over or near a ~~line of stairs~~ stairway or an elevator shaft, unless there is a solid roof or floor underneath the tank.

1509.3.3 Tank cover. Unenclosed ~~roof~~ tanks shall have covers sloping toward the ~~outer edges~~ perimeter of the tanks.

1509.4 Cooling towers. Cooling towers located on the roof deck of a building and greater than ~~in excess of~~ 250 square feet (23.2 m^2) in base area or greater than ~~in excess of~~ 15 feet (4572 mm) ~~high~~ in height above the roof deck, as measured to the highest point on the cooling tower, where ~~located on building~~ the roof~~s more~~ is greater than 50 feet (15 240 mm) ~~high~~ in height above grade plane shall be constructed of noncombustible materials ~~construction~~. The base area of cooling towers shall not exceed one-third the area of the supporting roof deck ~~area~~.

Exception: Drip boards and the enclosing construction shall be permitted to be ~~of~~ wood not less than 1 inch (25 mm) nominal thickness, provided the wood is covered on the exterior of the tower with noncombustible material.

1509.5 Towers, spires, domes and cupolas. ~~Any tower, spire, dome or cupula~~ Towers, spires, domes and cupolas shall be of a type of construction ~~not less in~~ having fire-resistance ~~rating~~ ratings not less than required for the building ~~to~~ on top of which ~~it~~ such tower, spire, dome or cupola is built. ~~attached, except that any such tower, spire, dome or cupula~~ Towers, spires, domes and cupolas greater than ~~that exceeds~~ 85 feet (25 908 mm) in height above grade plane as measured to the highest point on such structures, and either greater than ~~exceeds~~ 200 square feet (18.6 m2) in horizontal area or ~~is~~ used for any purpose other than a belfry or an architectural embellishment, shall be constructed of and supported on Type I or II construction.

1509.5.1 Noncombustible construction required. ~~Any tower, spire, dome or cupula~~ Towers, spires, domes and cupolas ~~that exceeds~~ greater than 60 feet (18 288 mm) in height above the highest point at which ~~it comes in~~ such structure contacts ~~contact with~~ the roof as measured to the highest point on such structure, or that exceeds 200 square feet (18.6 m^2) in area at any horizontal section, or which is intended to be used for any purpose other than a belfry or architectural embellishment, or is located on the top of a building greater than 50 feet (1524 mm) in building height shall be ~~entirely~~ constructed of and supported by noncombustible materials~~. Such structures~~ and shall be separated from the building below by construction having a fire-resistance rating of not less than 1.5 hours with openings protected in accordance with Section 712. ~~with a minimum 1.5-hour fire protection rating. Structures, except aerial supports 12 feet (3658 mm) high or less, flagpoles, water tanks and cooling towers, placed above~~ Such structures located on the top ~~roof~~ of a ~~any~~ building ~~more~~ greater than 50 feet (15 240 mm) in building height~~, shall be of noncombustible material and~~ shall be supported by noncombustible construction ~~of noncombustible material~~.

1509.5.2 Towers and spires. Enclosed towers and spires ~~where enclosed~~ shall have exterior walls constructed as required for the building ~~to~~ on top of which ~~they~~ such towers and spires are built ~~attached~~. The roof covering of spires shall not be ~~of a~~ less than the same class of roof covering ~~as~~ required for the ~~main roof of the rest of the structure~~ building on top of which the spire is located.

2. Add new text as follows:

1509.6 Mechanical equipment screens. Mechanical equipment screens shall be constructed of the materials specified for the exterior walls in accordance with the type of construction of the building without being required to comply with the fire-resistance rating requirements.

1509.6.1 Height limitations. Mechanical equipment screens shall not exceed 18 feet (5486 mm) in height above the roof deck, as measured to the highest point on the mechanical equipment screen, and the highest point on the mechanical equipment screen, as measured to grade plane, shall not exceed the maximum building height allowed for the building by other provisions of this code.

> **Exception:** Where located on buildings of Type IA construction, the height of mechanical equipment screens shall not be limited.

1509.6.2 Types I, II, III, and IV construction. Regardless of the requirements in Section 1509.6, mechanical equipment screens shall be permitted to be constructed of combustible materials where located on the roof decks of building of Type I, II, III, or IV construction in accordance with any of the following limitations:

1. The fire separation distance shall not be less than 20 feet (6096 mm) and the height of the mechanical equipment screen above the roof deck shall not exceed 4 feet (1219 mm) as measured to the highest point on the mechanical equipment screen.
2. The fire separation distance shall not be less than 20 feet (6096 mm) and the mechanical equipment screen shall be constructed of fire-retardant-treated wood complying with Section 2302.2 for exterior installation.
3. The materials shall have a flame spread index of 25 or less when tested in the minimum and maximum thicknesses intended for use with each face tested independently in accordance with ASTM E 84 or UL 723, the facings shall be tested in the minimum and maximum thicknesses intended for use in accordance with, and shall comply with the acceptance criteria of, NFPA 285, and the facings shall be installed as tested but without any substrates or wall assemblies.

1509.6.3 Type V construction. The height of mechanical equipment screens located on the roof decks of buildings of Type V construction, as measured from grade plane to the highest point on the mechanical equipment screen, shall be permitted to exceed the maximum building height allowed for the building by other provisions of this code where complying with any one of the following limitations, provided the fire separation distance is greater than 5 feet (1524 mm):

1. Where the fire separation distance is not less than 20 feet (6096 mm), the height above grade plane of the mechanical equipment screen shall not exceed 4 feet (1219 mm) more than the maximum building height allowed,
2. The mechanical equipment screen shall be constructed of noncombustible materials,
3. The mechanical equipment screen shall be constructed of fire-retardant-treated wood complying with Section 2303.2 for exterior installation, or
4. Where fire separation distance is not less than 20 feet (6096 mm), the mechanical equipment screen shall be constructed of materials having a flame spread index of 25 or less when tested in the minimum and maximum thicknesses intended for use in accordance with ASTM E 84 or UL 723.

1509.7 Other rooftop structures. Rooftop structures not regulated by Sections 1509.2 through 1509.6 shall comply with Section 1509.7.1 through 1509.7.5 as applicable.

1509.7.1 Aerial supports. Aerial supports shall be constructed of noncombustible materials.

Exception: Aerial supports not greater than 12 feet (3658 mm) in height as measured from the roof deck to the highest point on the aerial supports shall be permitted to be constructed of combustible materials.

1509.7.2 Bulkheads. Bulkheads used for the shelter of mechanical or electrical equipment or vertical shaft openings in the roof assembly shall comply with Section 1509.2 as penthouses. Bulkheads used for any other purpose shall be considered as an additional story of the building.

1509.7.3 Dormers. Dormers shall be of the same type of construction as required for the roof in which such dormers are located or the exterior walls of the building.

1509.7.4 Fences. Fences and similar structures shall comply with Section 1509.6 as mechanical equipment screens.

1509.7.5 Flagpoles. Flagpoles and similar structures shall not be required to be constructed of noncombustible materials and shall not be limited in height or number.

3. Revise as follows:

1502.1 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.

MECHANICAL EQUIPMENT SCREEN. A ~~partially enclosed~~ rooftop structure, not covered by a roof, used to aesthetically conceal ~~heating, ventilating and air conditioning (HVAC)~~ plumbing, electrical or mechanical equipment from view.

PENTHOUSE. An enclosed, unoccupied rooftop structure ~~above the roof of a building, other than a tank, tower, spire, dome, cupola or bulkhead.~~ used for sheltering mechanical and electrical equipment, tanks, elevators and related machinery, and vertical shaft openings.

ROOF DECK. The flat or sloped surface constructed on top of the exterior walls of a building or other supports for the purpose of enclosing the story below, or sheltering an area, to protect it from the elements, not including its supporting members or vertical supports.

ROOFTOP STRUCTURE. ~~An enclosed~~ A structure erected on ~~or above~~ top of the roof deck or on top of any part of a building.

Reason: The vast majority of the revisions proposed in this code change are editorial in nature but there are also a few technical changes, some of which are significant. The editorial changes are provided for:

clarification
elimination of redundant language including redundant Exceptions
consistency of terminology and application of requirements to specific types of rooftop structures
reformatting regarding the use of the Exception format
reformatting into subsections that deal with different requirements contained in the same section
determination of the height of rooftop structures

We have also provided phraseology to address how to determine the height of a rooftop structure since that height is not defined, unlike “Building Height” which is defined in Section 502.1. We have also substituted the term “roof deck” for the word “roof” since it is a defined term found in Section 1502.1. We have also proposed to revise the definition for “Roof Deck” to clarify its application and make it more specific.

The specific revisions proposed to each Subsection of Section 1509 are discussed in the following.

1509.2 Penthouses. The revision to the first sentence clarifies that the story below, of which the penthouse would be considered a portion if complying with these provisions, is the story that is located directly below the roof deck on which the penthouse is located. Since there are many stories below the penthouse, although it may be obvious, this clearly indicates that it is the story directly below the roof deck on which the penthouse is located. The second sentence merely makes it clear that any other penthouse not compliant with these provisions would actually be considered as an additional story of the building.

1509.2.1 Height above roof deck. The reference to other projections above the roof has been deleted since the focus of this section is penthouses which are defined in Section 1502.1. In fact, the definition for "Penthouse" is proposed to be revised by this code change to further clarify it based on the intent of the section addressing penthouses. Also, we have added new Sections 1509.6 and 1509.7 which address other projections above the roof. This section has also been reformatted into Exception format to make its application more clear and the wording has been revised to be consistent with other code text used throughout the code. These revisions should not result in any technical changes.

1509.2.2 Area limitation. The word "enclosed" is proposed to be added to modify the term "rooftop structures" to be consistent with the proposed revision to the definition for "rooftop structure" contained in this code change. It is our opinion that a rooftop structure encompasses all of the types of enclosures and other projections that could be located or constructed on the top of a roof deck of a building, so it would include both enclosed and unenclosed structures. So the definition of rooftop structure has been broadened to delete the limitation on enclosed structures. In this way both mechanical equipment screens, for which the definition is also being revised by this code change proposal, and penthouses become subsets of "rooftop enclosures". Mechanical equipment screens are not covered by a roof so they are not enclosed and "penthouses" are truly enclosed structures. The definition for "Mechanical Equipment Screen," as noted, is proposed to be revised to indicate that it is a rooftop structure that is not covered by a roof and is, thus, not enclosed. We believe this is a better definition than relying on the term "partially enclosed" which is proposed to be deleted in the definition for "Mechanical Equipment Screen." In the last sentence rather than referring to the definition for "Fire Area" in Section 902, we believe it is more appropriate to refer to where fire areas are used in Section 901.7 to clarify the intent of this sentence.

1509.2.3 Use limitations. The reference to "bulkhead or any other similar projection above the roof" has been deleted since it is being addressed in a proposed new Section 1509.7.2 as it is out of context in this section on penthouses. This section has also been revised to include electrical equipment as a part of the sheltering function of a penthouse since most mechanical equipment installations will also be associated with electrical equipment. The word "assembly" has been added to the word "roof" since "roof assembly" is a defined term in Section 1502.1.

New Section 1509.2.4 Weather protection. This is a reformatting of the section since this sentence addresses a separate requirement from the use limitations provisions in Section 1509.2.3. Again, "electrical equipment" has also been added for the same reasons as noted above. The second sentence has been deleted since it is redundant as it has already been addressed in Section 1509.2. The last sentence has been deleted since it is out of context as it addresses wood flagpoles or similar structures on the roofs of buildings. It has been relocated to a new Section 1509.7.5.

1509.2.5 Type of construction. The revision to the charging paragraph is basically a clarification for specifying compliance with the type of construction of the building on which the penthouses are built.

Exception 1. These revisions are editorial to be consistent with terminology used throughout the code. The revisions to the second sentence are to implement the intent of these Exceptions which address the fire-resistance ratings not being required. By default, this results in noncombustible construction in a Type I building. The last sentence has been deleted because it is unnecessary as this is a basic requirement for buildings of Type I construction.

Exception 2. Again, these are basically editorial revisions to use terminology consistent with the rest of the code and to be consistent with the revisions to Exception 1 as noted above. The revisions to the last sentence are made for the same reasons as noted in Exception 1 above where noncombustible construction is the requirement for such partitions, but the intent of the section is to allow for the use of fire-retardant-treated wood in lieu of noncombustible construction.

Exception 3. Since this is an Exception, the word "and" has been changed to "or" in the list of the types of construction to which this Exception is applicable. Additional editorial revisions have been made to be consistent with those made to Exceptions 1 and 2 above. The phrase "from a common property line" has been deleted because it is unnecessary since the term "fire separation distance" is a defined term in Section 702.1. The next to the last sentence has been deleted as it is unnecessary based on the charging sentence in Section 1509.2.4 to which this is an Exception. This appears to be a code requirement within an Exception that is not necessary. Similarly, the last sentence has also been deleted since other provisions of the code already allow such construction.

Exception 4. This Exception has been deleted since it is redundant. It is covered by Exception 1 above and is actually more limiting than Exception 1.

Exception 5. The reference to Types I and II construction have been deleted as they are already covered by Exception 2 above, whereas this Exception as noted in Exception 4 above is somewhat more restrictive than Exception 2. The rest of the revisions are editorial by utilizing consistent terminology to that used throughout the rest of the code and to be consistent with the revisions to Exceptions 1 and 2 above.

Exception 6. This Exception is being deleted since it does not address penthouses and is, thus, out of context. The provisions of this Exception, however, have been utilized in the new Sections 1509.6 and 1509.7 being added by this code change which will be discussed later.

Exception 7. This section is also being deleted since it is out of context as it does not address penthouses. It has been editorially revised and relocated as new Section 1509.7.3.

1509.3 Tanks. This entire subsection including sub-subsections .1, .2, and .3 have been editorially revised with no technical changes.

1509.4 Cooling towers. This section has been revised to make it clear that it is only applicable to cooling towers located on the roof deck of a building. It also provides a clarification on how the height of the cooling tower is to be measured for applying the limitations in this section. The rest of the changes are editorial without technical change.

1509.5 Towers, spires, domes, and cupolas. These are editorial revisions to make the section consistent with the previous sections in terms of format and terminology and also to incorporate the method for measuring the height of these structures.

1509.5.1 Noncombustible construction required. The first sentence has been revised to be consistent with Section 1509.5 including how to make the measurements for the height of these structures. The last sentence has been revised and broken up into separate parts with the one part referencing the noncombustible construction limitation incorporated into the first sentence which deals with the construction of these structures, whereas the last sentence deals with the support of these structures under certain conditions. The second sentence has been revised to be included in the first sentence since it is conditional to the application of the first sentence. The reference to the minimum 1.5 hour fire protection rating for protection of openings in the 1.5 hour separation of the structures from the building below has been deleted with a reference to Section 712 provided. Section 712 addresses how to protect openings in horizontal assemblies. Generally speaking, opening protectives with fire protection ratings are not used to protect openings in horizontal assemblies unless those openings are protected with shaft enclosures with openings. However, there are floor fire door assemblies that can be used which have a fire-resistance rating, as opposed to a fire protection rating.

1509.5.2 Towers and spires. These are basically editorial clarifications without any technical change.

1509.6 Mechanical equipment screens. This is a new section being proposed to specifically address mechanical equipment screens which are defined in Section 1502.1. They are not otherwise addressed in Section 1509 with the exception of the out of context Exception 6 to Section 1509.2.4 which only addresses the type of construction of penthouses as previously noted. Also, as previously noted, we are proposing to revise the definition for "Mechanical Equipment Screen" in Section 1502.1 to make it clear that it is a rooftop structure that is not covered by a roof, rather than a "partially enclosed" rooftop structure.

This new section takes what we believe to be a conservative approach to the construction of mechanical equipment screens on roofs by specifying that they must be constructed of the same materials as required by the code for exterior walls based on the type of construction of the building on which they are located. However, it is proposed that they be exempt from the fire-resistance rating requirements since they do not fully enclose a space as they are without a roof and they represent a different exposure hazard than a penthouse, for example. Basically, the exposure hazard of a mechanical equipment screen is the combustibility of the screen itself and the amount of combustible materials it contains.

1509.6.1 Height limitations. The height limits specified in this section are also conservative, in our opinion, as they are based on those required for penthouses in Section 1509.2.1. The height limit is also based on the assumption that the overall height of the mechanical equipment screen should not exceed that allowed for the maximum building height for the type of construction of the building on which it is constructed. Thus, the need for the Exception for mechanical equipment screens located on buildings of Type IA construction which are not limited in height by Table 503.

1509.6.2 Types I, II, III, and IV construction. This new section is, in essence, an Exception to the requirements in Section 1509.6 for these types of construction which require the exterior walls to be constructed of noncombustible materials. The three itemized limitations in this section allow for combustible materials to be used for the construction of mechanical equipment screens based upon the provisions in those three items as discussed in the following.

Item 1. This item is based on Exception 6 to Section 1509.2.4 as previously noted for penthouses which has been deleted. The 1-story building height limitation has not been included since we believe it is not necessary. In our opinion, the hazard of a combustible mechanical equipment screen located on the roof of a Type I, II, III, or IV building with a fire separation distance of not less than 20 feet and with the height of the mechanical equipment screen limited to 4 feet above the roof deck is not a significant fire hazard. It is interesting to note that Table 705.8 Maximum Area of Exterior Wall Openings Based on Fire Separation Distance and Degree of Opening Protection would allow up to 45% of the exterior wall area of a nonsprinklered building to have unprotected openings and would allow unlimited unprotected openings in sprinklered buildings. Thus, for a building having floor-to-floor heights of at least 10 feet, which is very minimal, unprotected window openings around the entire perimeter could be as tall as 4.5 feet. This would represent a greater fire exposure, once the story flashes over and the windows break out, than a burning 4 foot high mechanical equipment screen which will normally be set back some distance from the face of the exterior wall.

Item 2. The provisions of this Item are based on Exceptions 2 and 3 to Section 1509.2.4 for penthouses with the 2-story limit on Type I buildings omitted. We believe this to be a reasonable approach since the hazard doesn't justify limiting the Type I buildings to two stories in height where fire-retardant-treated wood is used to construct these unenclosed mechanical equipment screens. The main difference between Item 1 above and this Item 2 is that Item 2 does not place a 4 foot height limit on the height of the mechanical equipment screen above the roof deck. That is because it must be constructed of fire-retardant-treated wood as compared to any combustible material allowed by the code being permitted in Item 1. Of course, the height of the mechanical equipment screen is still limited to a maximum of 18 feet above the roof based on Section 1509.6.1. It is also limited to the maximum building height that would be allowed by the type of construction of the building in accordance with Section 1509.6.1 as well.

Item 3. These limitations are based on a totally new concept where the combustible materials used to build the mechanical equipment screen are limited to a maximum flame spread index of 25 (which is also required for fire-retardant-treated wood) and the materials are required to be successfully tested in accordance with NFPA 285 Standard Method of Test for the Evaluation of Flammability Characteristics of Exterior Nonload-Bearing Wall Assemblies Containing Combustible Components. This is the same test method that is used to validate the use of foam-plastic insulations in exterior walls of Types I, II, III, and IV construction, as well as for the use of metal composite materials (MCM) in accordance with Section 1407.10. Although the material would be tested as the outer face (or skin) of the exterior wall in the NFPA 285 test as part of an exterior wall assembly, the test clearly assesses the surface flame spread resistance of the materials constituting the outer face, as well as to a certain degree, the inner face where it is exposed to any open cavities in the wall assembly. The NFPA 285 test is conducted for a full 30 minutes under severe fire exposure conditions to both the inside of the wall assembly and the outside of the wall assembly with an exterior window burner replicating a fire that has gone to post-flashover and has broken out a window, exposing the outside face of the exterior wall finish. Since the materials used to construct the mechanical equipment screen do not comprise a completely enclosed wall assembly, the maximum flame spread index of 25 has been proposed as a conservative limitation for the backside face of the material which may not have been directly exposed to the exterior window burner flame in the NFPA 285 test. Since the NFPA 285 test is used to qualify combustible materials for use where noncombustible exterior walls are required, it seems reasonable to allow its use for this application for mechanical equipment screens without the need to have the entire wall assembly constructed as tested for the mechanical equipment screen, instead utilizing the materials tested on the exterior face of the wall system in accordance with NFPA 285.

1509.6.3 Type V construction. This new section is basically an Exception to the requirements in Section 1509.6.1 Height Limitations for Type V construction where the mechanical equipment screens are allowed to be constructed of combustible materials. The one condition that must be met for all four options in this section is that the minimum fire separation distance must be greater than 5 feet which is consistent with Section 1406 Combustible Materials on the Exterior Side of Exterior Walls.

Item 1. This item is based on Exception 6 to Section 1509.2.4 for penthouses which has been deleted. It was limited to one story buildings. However, we don't see the hazard represented by a 4 foot high increase in the overall height of the mechanical equipment screen on buildings of Type V construction (which are allowed to be constructed entirely of combustible materials) as justifying that one story limitation. This is especially true where the fire separation distance specified is not less than 20 feet. Please refer to the discussion on Item 1 of Section 1509.6.2 above.

Item 2. This seems intuitively obvious to allow these mechanical equipment screens to be taller when they are constructed of noncombustible materials where combustible materials would otherwise be permitted. Noncombustible materials pose no additional fire load or fire exposure to the building.

Item 3. This allows the use of fire-retardant-treated wood which, although combustible, does not pose a significant fire hazard, in our opinion, when constructed as a mechanical equipment screen where there is a minimum 5 foot fire separation distance.

Item 4. This is somewhat similar to Item 3 in that fire-retardant-treated wood is required to have a maximum flame spread index of 25 as proposed for the combustible materials allowed in this item. However, there is an additional requirement that the fire separation distance be not less than 20 feet as compared to the base requirement of 5 feet for these provisions to be allowed to be used. We believe that the fire hazard associated with this type of installation would not be significant so as to allow the greater heights for the mechanical equipment screens installed on these Type V buildings. Again, please refer to the discussion on Item 1 of Section 1509.6.2 above regarding the minimum 20 foot fire separation distance limitation.

1509.7 Other rooftop structures. This new section becomes a catchall section to address other rooftop structures that are not specifically regulated by Sections 1509.2 through 1509.6. In reviewing Section 1509 we found references to such other rooftop structures as aerial supports covered in proposed new Section 1509.7.1, bulkheads covered in proposed new Section 1509.7.2, dormers covered in proposed new Section 1509.7.3, fences covered in proposed new Section 1509.7.4, and flagpoles covered in proposed new Section 1509.7.5. So we believe we have addressed all of the rooftop structures the code currently addresses.

1509.7.1 Aerial supports. The requirements for this section are taken from Section 1509.5.1 which were deleted because they were out of context in regard to the provisions of the section which addressed towers, spires, domes, and cupolas.

1509.7.2 Bulkheads. It is proposed to treat bulkheads like penthouses as we believe was intended by Section 1509.2.3 where bulkheads are currently referred to. We eliminated bulkheads from that section since it specifically addresses penthouses. In this way we keep the section clean and then simply reference it and require bulkheads to be constructed to meet those requirements as if they were penthouses. Looking at the definition for "Bulkhead" in Webster's dictionary, the most likely meaning within the context of Section 1509 is "a projecting framework with a sloping door giving access to a cellar stairway or a shaft."

1509.7.3 Dormers. This is where Exception 7 of Section 1509.2.4 was relocated after it was deleted from that section addressing penthouses since it was clearly out of context.

1509.7.4 Fences. It is our best judgment that fences should be treated as mechanical equipment screens since they are similar structures and can be considered similar in fire hazard. It should be noted that they are mentioned in Exception 6 to Section 1509.2.4 for penthouses which was deleted, again, as being out of context.

1509.7.5 Flagpoles. These requirements are taken from the last sentence of Section 1509.2.3 for penthouses which was deleted as being out of context.

In summary, it is readily obvious after delving into Section 1509 that it is very disjointed and inconsistent and utilizes terminology and language that is not consistent with the rest of the code. It appears to be, as it most likely is, an amalgam of the three legacy code requirements for rooftop structures which was put together without a lot of detailed evaluation or review. Since Trespa North America, Ltd. manufactures products that are used in many of these applications, we have come across many projects where Section 1509 has been attempted to be applied but without much success. It is extremely difficult to determine what the true intent is of many of the requirements, especially those that do not specifically address penthouses. We have tried our best to clarify and reformat this section to make it clearer to understand, easier to read and interpret, and, hopefully, more effectively enforced and applied. We hope that the Committee will give this comprehensive revision to Section 1509 serious and in-depth consideration so that we can fix it for the 2012 IBC, remembering that we only have one chance to get it corrected before the next edition is published.

Cost Impact: The proponent shall indicate one of the following regarding the cost impact of the code change proposal: The code change proposal will not increase the cost of construction.

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

ICCFILENAME: Thornberry-S1-1509

S28–09/10
1509.6 (New)

Proponent: Mark S. Graham, National Roofing Contractors Association (NRCA)

THIS PROPOSAL IS ON THE AGENDA OF THE IBC GENERAL CODE DEVELOPMENT COMMITTEE. SEE THE TENTATIVE HEARING ORDER FOR THIS COMMITTEE.

Add new text as follows:

<u>**1509.6 Photovoltaic systems.** Rooftop mounted photovoltaic systems shall be designed in accordance with this section.</u>

<u>**1509.6.1 Wind resistance.** Rooftop mounted photovoltaic systems shall be designed for wind loads in accordance with Chapter 16.</u>

<u>**1509.6.2 Fire classification.** Rooftop mounted photovoltaic systems shall be shall have the same fire classification as the roof assembly as defined Section 1505.</u>

<u>**1509.6.3 Installation.** Rooftop mounted photovoltaic systems shall be installed in accordance with the manufacturer's installation instructions.</u>

Reason: Rooftop-mounted photovoltaic systems are becoming more common. This proposed code change proposal is intended clarify that rooftop-mounted photovoltaic systems need to comply with building code requirements. The minimum requirements set forth here are intended for the rooftop-mounted photovoltaic system to be required to comply with the same minimum requirements of the underlying roof assembly.

Specific requirements applicable to the electrical portion of rooftop-mounted photovoltaic systems are left to the applicable electrical code.

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

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

ICCFILENAME: GRAHAM-S8-1509.6 NEW

S29–09/10
1510.3

Proponent: Mike Ennis representing Single Ply Roofing Industry (SPRI, Inc.)

Revise as follows:

1510.3 Recovering versus replacement. New roof coverings shall not be installed without first removing all existing layers of roof coverings down to the roof deck where any of the following conditions occur:

1. Where the existing roof or roof covering is water soaked or has deteriorated to the point that the existing roof or roof covering is not adequate as a base for additional roofing.
2. Where the existing roof covering is wood shake, slate, clay, cement or asbestos-cement tile.
3. Where the existing roof has two or more applications of any type of roof covering.

Exceptions:

1. Complete and separate roofing systems, such as standing-seam metal roof systems, that are designed to transmit the roof loads directly to the building's structural system and that do not rely on existing roofs and roof coverings for support, shall not require the removal of existing roof coverings.
2. Metal panel, metal shingle and concrete and clay tile roof coverings shall be permitted to be installed over existing wood shake roofs when applied in accordance with Section 1510.4.
3. The application of a new protective coating over an existing spray polyurethane foam roofing system shall be permitted without tear-off of existing roof coverings.
4. <u>The application of a new single-ply membrane directly over an existing roofing system shall be permitted without tear-off of the existing roof coverings except where the existing roof or roof covering is water soaked or has deteriorated to the point that the existing roof or roof covering is not adequate as a base for additional roofing..</u>

Reason: A layer of single-ply membrane is very lightweight, adding approximately 1/3 of a pound per square foot to the existing structure. The single-ply membrane can be used as a reflective layer to reduce rooftop temperatures, thus providing a cooling benefit for the building. The cooling benefits of reflective roof systems are recognized by the energy codes. This exception will allow for a cost effective method for increasing the energy efficiency of the building while providing excellent waterproofing protection. A single layer of membrane will also provide the same function and benefit as a new protective coating over an existing spray polyurethane foam roofing system, which is currently allowed as an exception under Section 1510.3, Exception 3.

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

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

ICCFILENAME: ENNIS-S2-1510.3

S30–09/10
1510.3; IRC R907.3

Proponent: Mark S. Graham, representing National Roofing Contractors Association (NRCA)

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

1510.3 Recovering versus replacement. New roof coverings shall not be installed without first removing all existing layers of roof coverings down to the roof deck where any of the following conditions occur:

1. Where the existing roof or roof covering is water soaked or has deteriorated to the point that the existing roof or roof covering is not adequate as a base for additional roofing.
2. Where the existing roof covering is wood shake, slate, clay, cement or asbestos-cement tile.
3. Where the existing roof has two or more applications of any type of roof covering.

Exceptions:

1. Complete and separate roofing systems, such as standing-seam metal roof systems, that are designed to transmit the roof loads directly to the building's structural system and that do not rely on existing roofs and roof coverings for support, shall not require the removal of existing roof coverings.
2. Metal panel, metal shingle and concrete and clay tile roof coverings shall be permitted to be installed over existing wood shake roofs when applied in accordance with Section 1510.4.
3. The application of a new protective coating over an existing spray polyurethane foam roofing system shall be permitted without tear-off of existing roof coverings.

4. Where the existing roof assembly includes an ice barrier membrane that is adhered to the roof deck, the existing ice barrier membrane shall be permitted to remain in place and covered with an additional layer of ice barrier membrane in accordance with Section 1507.

PART II – IRC BUILDING/ENERGY

Revise as follows:

R907.3 Recovering versus replacement. New roof coverings shall not be installed without first removing all existing layers of roof coverings where any of the following conditions exist:

1. Where the existing roof or roof covering is water-soaked or has deteriorated to the point that the existing roof or roof covering is not adequate as a base for additional roofing.
2. Where the existing roof covering is wood shake, slate, clay, cement or asbestos-cement tile.
3. Where the existing roof has two or more applications of any type of roof covering.
4. For asphalt shingles, when the building is located in an area subject to moderate or severe hail exposure according to Figure R903.5.

Exceptions:

1. Complete and separate roofing systems, such as standing-seam metal roof systems, that are designed to transmit the roof loads directly to the building's structural system and that do not rely on existing roofs and roof coverings for support, shall not require the removal of existing roof coverings.
2. Installation of metal panel, metal shingle and concrete and clay tile roof coverings over existing wood shake roofs shall be permitted when the application is in accordance with Section R907.4.
3. The application of new protective coating over existing spray polyurethane foam roofing systems shall be permitted without tear-off of existing roof coverings.
4. Where the existing roof assembly includes an ice barrier membrane that is adhered to the roof deck, the existing ice barrier membrane shall be permitted to remain in place and covered with an additional layer of ice barrier membrane in accordance with Section R905.

Reason: In roof removal situations where an existing ice barrier membrane is adhered to the existing roof deck, it is oftentimes difficult, if not impossible, to remove the existing layer of adhered ice barrier membrane without damaging or replacing the roof deck.

The proposed addition of a new exception to Section 1510.3 (IRC Section R907.3)—Recovering Versus Replacement is intended to account for this situation by allowing existing adhered ice barrier membrane to remain in place and be covered with a new ice barrier membrane as required in Section 1507 (IRC Section R905), followed by the installation of the new primary roof covering material.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: GRAHAM-S9-1510.3

S31–09/10
1602.1

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

Revise as follows:

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

VEHICLE BARRIER SYSTEM. A system of building components near open sides or walls of ~~a~~ garage floors or ramps ~~or building walls~~ that acts as a restraint~~s~~ for vehicles.

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. The need for correlation is due to ASCE 7 Proposal LLSC-LL9, which was approved by the Live Load Subcommittee and is being balloted by the Main Committee (Item #5 of the Second Main Committee Ballot on Live/Dead Load Provisions). It is expected that the Main Committee will approve the proposal.

The revision to the definition of "vehicle barrier system" will more clearly convey the intent that the entire perimeter of a garage floor or ramp, not merely where there are open sides, is subject o the loading requirements for vehicle barrier systems. It will also avoid the interpretation that building walls, regardless of their proximity to garage floors or ramps, are subject to the same loading requirements.

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

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

ICCFILENAME: Brazil-S20-1602.1

S32–09/10
1602.1

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

Revise as follows:

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

LIVE LOAD~~S~~. ~~Those loads~~ A load produced by the use and occupancy of the building or other structure ~~and do~~ that does not include construction or environmental loads such as wind load, snow load, rain load, earthquake load, flood load or dead load.

LIVE LOAD~~S,~~ ~~(~~ROOF~~)~~. ~~Those loads~~ A load on a roof produced (1) during maintenance by workers, equipment and materials; and ~~(2)~~ during the life of the structure by movable objects such as planters ~~and by people~~ or other similar small decorative appurtances that are not occupancy related; or (2) by the use and occupancy of the roof such as for roof gardens or assembly areas.

NOTATIONS.

D = Dead load.
E = Combined effect of horizontal and vertical earthquake induced forces as defined in Section 12.4.2 of ASCE 7.
F = Load due to fluids with well-defined pressures and maximum heights.
F_a = Flood load in accordance with Chapter 5 of ASCE 7.
H = Load due to lateral earth pressures, ground water pressure or pressure of bulk materials.
L = Roof live load greater than 20 psf (0.96 kN/m^2) and floor live load~~, except roof live load, including any permitted live load reduction~~.
L_r = Roof live load ~~including any permitted live load reduction~~ of 20 psf (0.96 kN/m^2) or less.
R = Rain load.
S = Snow load.
T = Self-straining force arising from contraction or expansion resulting from temperature change, shrinkage, moisture change, creep in component materials, movement due to differential settlement or combinations thereof.
W = Load due to wind pressure.

Reason: The purpose for this proposal is to revise the IBC for consistency with an ASCE 7 proposal being considered by the Live Load Subcommittee. The IBC currently treats floor and roof live loads synonymously. Table 1607.1 lists live loads as great as 100 psf for occupied areas of floors and roofs. Roof live load, however, also accounts for loads from maintenance workers. Item #30 of Table 1607.1 separately specifies a uniform load of 20 psf and a concentrated load of 300 lb. for this purpose.

The load combinations in Sections 1605.2 and 1605.3 should account for roof live loads at occupied areas in the same manner as for floor live loads. For strength design in Sec. 1605.2 (neglecting snow load and rain load), Equations 16-3 and 16-4 consider dead load, floor live load, roof live load and wind load. Earthquake load is not considered in combination with roof live load. For allowable stress design in Section 1605.3.1, however (neglecting earth/water pressure, fluid load, snow load and rain load), Equation 16-13 considers dead load, floor live load, roof live load, and wind or earthquake load. Thus, earthquake load is required to be considered in combination with roof live load for allowable stress design but not for strength design.

The proposal changes the notation for floor live load and roof live load by (1) assigning "L" to roof live load greater than 20 psf as well as to floor live load, and (2) assigning "L_r" to roof live load of 20 psf or less. This new boundary between "L" and "L_r" will distinguish between roof live loads intended for the general public ("L") and only for maintenance workers ("L_r").

A separate proposal splits ASD Equation 16-13 into two equations: 16-13 for wind load and 16-14 for earthquake load. This change in the ASD load combinations will make them equivalent to the LRFD load combinations with respect to their treatment of roof live load.

The proposal revises the definitions of "live load" and "live load, roof" for consistency with the corresponding definitions in ASCE 7 as modified by the proposal being considered by the ASCE 7 Live Load Subcommittee. Note that neither the IBC nor ASCE 7 define "floor live load" or utilize the term in their respective provisions. "Live load" in both documents is defined as a load "produced by the use and occupancy of the building or other structure," which can occur on roofs as readily as on floors.

The proposal deletes "including any permitted live load reduction" from the notation "L" and "L_r" in Section 1602.1 for consistency with the same notation in Section 2.2 of ASCE 7, which corresponds to IBC Section 1602.1. This has the added benefit of avoiding potential conflicts with the same notation defined differently in IBC Sections 1607.9.1 ("L") and 1607.11.2.1 ("L_r"), which is reproduced below. These sections define the notation as reduced live load and distinguish them from unreduced live load, L_o, which is tabulated in Table 1607.1. Similar definitions are found in Sections 4.7.1 and 4.8.1 of ASCE 7-10. Note that the addition of "roof" before "live load" in L_r below is in a separate proposal.

IBC Section 1607.9.1 (and Section 4.7.1 of ASCE 7-10):

L = Reduced design live load per square foot (meter) of area supported by the member.

L_o = Unreduced design live load per square foot (meter) of area supported by the member (see Table 1607.1).

IBC Section 1607.11.2.1 (and Section 4.8.1 of ASCE 7-10):

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

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

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

ICCFILENAME: Brazil-S34-1602.1

S33–09/10

1603.1.3, 1603.1.4, 1603.1.5, 1705.3.4, 2304.6.1

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

Revise as follows:

1603.1.3 Roof snow load data. The ground snow load, P_g, shall be indicated. In areas where the ground snow load, P_g, exceeds 10 pounds per square foot (psf) (0.479 kN/m^2), the following additional information shall also be provided, regardless of whether snow loads govern the design of the roof:

1. Flat-roof snow load, P_f.
2. Snow exposure factor, C_e.
3. Snow load importance factor, I_s.
4. Thermal factor, C_t.

1603.1.4 Wind design data. The following information related to wind loads shall be shown, regardless of whether wind loads govern the design of the lateral-force-resisting system of the building:

1. Basic wind speed (3-second gust), miles per hour (km/hr).
2. ~~Wind importance factor, I, and~~ Occupancy category.
3. Wind exposure~~,~~; ~~if more than one wind exposure is utilized, the wind exposure and~~ applicable wind direction ~~shall be indicated~~ if more than one wind exposure is utilized.
4. ~~The~~ Applicable internal pressure coefficient.
5. ~~Components and cladding. The~~ Design wind pressures ~~in terms of psf (kN/m²)~~ to be used for ~~the design of~~ exterior component and cladding materials not specifically designed by the registered design professional responsible for the design of the structure, psf (kN/m^2).

1603.1.5 Earthquake design data. The following information related to seismic loads shall be shown, regardless of whether seismic loads govern the design of the lateral-force-resisting system of the building:

1. ~~Seismic importance factor, I, and~~ Occupancy category.
2. Seismic importance factor, I_e.
~~2~~3. Mapped spectral response accelerations, S_S and S_1.
~~3~~4. Site class.
~~4~~5. Spectral response coefficients, S_{DS} and S_{D1}.
~~5~~6. Seismic design category.
~~6~~7. Basic seismic-force-resisting system(s).
~~7~~8. Design base shear(s).

~~8~~9. Seismic response coefficient(s), C_S.
~~9~~10. Response modification factor(s), *R*.
11. Location of base(s) as defined in Section 11.2 of ASCE 7.
~~10~~12. Analysis procedure used.

1705.3.4 Seismic Design Category D. The following additional systems and components in structures assigned to Seismic Design Category D:

1. Systems required for Seismic Design Category C.
2. Exterior wall panels and their anchorage.
3. Suspended ceiling systems and their anchorage.
4. Access floors and their anchorage.
5. Steel storage racks and their anchorage, where the importance factor, I_e~~,~~ is equal to 1.5 in accordance with Section 15.5.3 of ASCE 7.

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

Reason: The purpose for the proposal is to update the required documentation of snow, wind and seismic design data with respect to the corresponding provisions in ASCE 7. The deletion of wind importance factor is due to ASCE 7 Proposal WSC-WL8-14, which has been approved by the Wind Subcommittee is being balloted by the Main Committee (Item #20 of the First Main Committee Ballot on Wind Load Provisions). It is expected that the Main Committee will approve the proposal. Adding the locations of the seismic base will add a design parameter that is critical to understanding the basis of the structural design. The remaining changes are editorial.

Changes are also being made to the symbols for importance factor in conjunction with ASCE 7 Proposal GPSC-3BR2, which was approved by the General Subcommittee on March 1, 2009 and is being balloted by the Main Committee (Item #14 of the Second Main Committee Ballot on General Requirements). It is expected that the Main Committee will approve the proposal. All instances of “importance factor” in the 2009 IBC are included in this proposal.

A separate proposal revises all instances of “occupancy category” in the IBC and IEBC to “risk category.” These revisions are not repeated in this proposal but, should both proposals be approved by the membership, the proponent intends that “occupancy category” in Item #2 of Section 1603.1.4 and Item #1 of Section 1603.1.5 be changed to “risk category.”

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

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

ICCFILENAME: Brazil-S39-1603.1.3

S34–09/10
1603.1.5

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

Revise as follows:

1603.1.5 Earthquake design data. The following information related to seismic design loads shall be shown, regardless of whether seismic loads govern the design of the lateral-force-resisting system of the building:

1. Seismic importance factor, *I*, and *occupancy category.*
2. Mapped spectral response accelerations, S_S and S_1.
3. *Site class.*
4. Spectral response coefficients, S_{DS} and S_{D1}.
5. *Seismic design category.*
6. Basic seismic-force-resisting system(s).
7. Design base shear.
8. Seismic response coefficient(s), C_S.
9. Response modification factor(s), *R*.
10. Analysis procedure used.

11. Applicable horizontal structural irregularities.
12. Applicable vertical structural irregularities.

Reason: Structural irregularities (defined in ASCE-7 section 12.3) can result in restrictions on building height, prohibition of certain configurations, increased design forces, additional analytical requirements, restriction of permissible analytical procedures, greater building separations, or additional detailing requirements for certain structural elements. It is often not evident whether one or more irregularities are applicable to a structure, because many of them require structural analysis to determine their applicability. This information is useful for building officials, plan checkers, peer reviewers, and for structural engineers in future building additions and/or alterations.

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

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

ICCFILENAME: MOORE-HUSTON-S2-1603.1.5

S35–09/10
Table 1604.3

Proponent: Stephen Kerr, PE, SE, representing self

Revise as follows:

TABLE 1604.3
DEFLECTION LIMITS [a, b, c, h, i]

Construction	L	S or W [f]	D + L [d, g]
Roof members: [e]			
Supporting plaster or stucco ceiling	*l* / 360	*l* / 360	*l* / 240
Supporting nonplaster ceiling	*l* / 240	*l* / 240	*l* / 180
Not supporting ceiling	*l* / 180	*l* / 180	*l* / 120
Floor members	*l* / 360		*l* / 240
Exterior walls and interior partitions:			
With plaster or stucco finishes	-	*l* / 360	-
With other brittle finishes	-	*l* / 240	-
With flexible finishes	-	*l* / 120	-
Farm Buildings	-	-	*l* / 180
Greenhouses	-	-	*l* / 120

(Footnotes not shown, remain unchanged)

Reason: The proposal adds a new line item for the deflection limit on plaster or stucco finishes. The intent is to bring the Deflection Limits table into conformance with the IRC deflection table (Table R301.7) and the referenced ASTM standard. ASTM C926-98a Standard Specification for Application of Portland Cement-Based Plaster section Annex A2.1.6 states "Maximum allowable deflection for vertical or horizontal framing for plaster, not including cladding, shall be L/360."

Cost Impact: The code change proposal will not increase the cost of construction, because the defection limit is already required.

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

ICCFILENAME: KERR-S1-TABLE 1604.3

S36–09/10
Table 1604.3

Proponent: Robert McCluer, RMC Code Consulting, representing the Metal Construction Association (MCA)

Revise as follows:

TABLE 1604.3
DEFLECTION LIMITS a, b, c, h, i, j
(No change to table entries.)

(No change to footnotes a through h)

i. Metal composite material panels shall be designed to have not more than *l*/180 permanent set deflection after the application of the design load.

~~i.~~ j. For cantilever members, *l* shall be taken as twice the length of the cantilever.

Reason: The purpose of this proposed code change is to provide a deflection limitation for metal composite material (MCM) panels.
Objectives of Deflection Limitation:

1. To assure structural integrity so that the panel system will remain on the building during design wind load conditions.
2. To assure that sealant joints do not fail due to movement of panels during design wind load conditions.
3. To assure that "permanent set" of panels is not visually objectionable after occurrence of design wind load conditions.

Rationale for L/180 Permanent Set Limitation:

1. The structural adequacy of the panel system to meet the design load and remain on the building will be verified by structural testing. Therefore, deflection limitation will not be a controlling factor in safety considerations (staying on the building).
2. The adequacy of the sealant joints will be verified by the appropriate Static Air Infiltration (ASTM E283) and Static Water Infiltration (ASTM E331) tests. Therefore, deflection limitation will not be a controlling factor in the ability of the panels to provide appropriate protection for wind and water penetration of the building.
3. Generally, the widest available Metal Composite Material panel is 60". Therefore, if L/180 permanent set limitation is maintained the largest deviation from absolute flatness will be 1/3 inch. On a 60" wide panel this is a very acceptable visual deviation after a very severe climatic wind condition. During more normal wind events, permanent set would not occur. The ability of the panel system to accommodate expansion and contraction due to thermal changes is a much more critical visual concern.

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

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

ICCFILENAME: MCCLUER-S1-T1604.3

S37–09/10
1604.3.6

Proponent: Stephen Kerr, PE, SE, representing self

Delete and substitute as follows:

~~**1604.3.6 Limits.** Deflection of structural members over span, *l*, shall not exceed that permitted by Table 1604.3.~~

<u>**1604.3.6 Limits.** The deflection limits of Section 1604.3.1 shall be used unless more restrictive deflection limits are required in order to ensure adequate serviceability of the structural members and finish material.</u>

Reason: The intent of this proposal is to address possible increased deflection requirements that may be necessary for certain finishes not specifically addressed in Table 1604.3. Where the manufacturer or association of a specific finish material has deflection requirements more restrictive than Table 1604.3 the designer should take into account possibly changing the deflection limit. While this proposal does not require the designer to adhere to industry standards not specifically referenced in IBC Chapter 35, it is intended to put the designer on notice of possible other criteria for determining deflection limits.

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

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

ICCFILENAME: KERR-S2-1604.3.6

S38–09/10
1604.4

Proponent: Randy Lee Dube, Proprietor, representing Tor-Eggs-Tor Design Solutions

Revise as follows:

1604.4 Analysis. *Load effects* on structural members and their connections shall be determined by methods of structural analysis that take into account equilibrium, general stability, geometric compatibility and both short- and long-term material properties.

Members that tend to accumulate residual deformations under repeated service loads shall have included in their analysis the added eccentricities expected to occur during their service life.

Any system or method of construction to be used shall be based on a rational analysis in accordance with well-established principles of mechanics. Such analysis shall result in a system <u>or combined systems</u> that provides a complete load path <u>or combination of load paths</u> capable of transferring loads from their point <u>or multiple points</u> of origin to the load-resisting elements <u>or load-resisting and load-redistributing elements</u>.

The total lateral force shall be distributed to the various vertical elements of the lateral-force-resisting system in proportion to their rigidities, considering the rigidity of the horizontal bracing system or diaphragm. Rigid elements assumed not to be a part of the lateral-force-resisting system are permitted to be incorporated into buildings provided their effect on the action of the system is considered and provided for in the design. Except where diaphragms are flexible, or are permitted to be analyzed as flexible, provisions shall be made for the increased forces induced on resisting elements of the structural system resulting from torsion due to eccentricity between the center of application of the lateral forces and the center of rigidity of the lateral-force-resisting system.

Every structure shall be designed to resist the overturning effects caused by the lateral forces specified in this chapter. See Section 1609 for wind loads, Section 1610 for lateral soil loads and Section 1613 for earthquake loads.

Reason: ICC recognition, authority, enforcement of recently successfully developed (2005-2007) structural body-frame continuous reinforcing method and related structural design, assembly, and performance elements; and design & assembly criteria. (New reinforcing & structural body-frame systems currently not authorized to construction industry by IBC section 1604.4, Par 3 , Sentence 2.)

Cost Impact: Reduces new building/structure reinforcing methods costs; various cost impacts in other applications.

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

ICCFILENAME: DUBE-S1-1604.4

S39–09/10

Table 1604.5

Proponent: David R. Badger PE, CBO, Virginia Tech, representing self.

Revise as follows:

TABLE 1604.5
OCCUPANCY CATEGORY OF BUILDINGS AND OTHER STRUCTURES

OCCUPANCY CATEGORY	NATURE OF OCCUPANCY
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: • Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300. • Buildings and other structures containing elementary school, secondary school or day care facilities with an occupant load greater than 250. ~~• Buildings and other structures containing adult education facilities, such as colleges and universities, with an occupant load greater than 500.~~ • <u>Buildings and other structures used for the education of adults who are either above the 12th grade or not in a formal educational system; where the teaching is done in classroom settings with an occupant load density equal to or greater than that required for educational classroom areas per Table 1004.1.1; and the aggregate occupant load of all classrooms exceeds 500.</u> • Group I-2 occupancies with an occupant load of 50 or more resident patients but not having surgery or emergency treatment facilities. • Group I-3 occupancies. • Any other occupancy with an occupant load greater than 5,000[a]. • Power-generating stations, water treatment facilities for potable water, waste water treatment facilities and other public utility facilities not included in Occupancy Category IV. • Buildings and other structures not included in Occupancy Category IV containing sufficient quantities of toxic or ex-plosive substances to be dangerous to the public if released.

(Portions of table not shown do not change)
(No change to footnote)

Reason: The general language of 1604.5 is ambiguous and misleading. This problem is well recognized, and there have been several proposals to rewrite major portions of this in recent code change cycles. Although a general overhaul of this section is needed, there is a specific problem that needs to be addressed immediately. There is one phrase in Table 1604.5 that is routinely being misinterpreted, resulting in hidden costs which are extremely high, and completely unnecessary. It is a problem primarily for colleges and universities, has been a problem for many years, and must be corrected.

Under 1604.5 the phrase "*Buildings and other structures containing adult education facilities, such as colleges and universities*" is very easily interpreted to require <u>any</u> building on a college or university campus with an occupant load over 500 to be classified as Occupancy Category III, regardless of use. The phrase is so poorly written, it is difficult to not read it this way. The perceived connection to "adult education facilities" occurs simply because the building is located on a college or university campus; not because there is an educational function occurring within the building.

For example, a research laboratory building with 600 occupants located in an industrial research park clearly would be classified under Occupancy Category II. If the exact same building was placed on a university campus, the Occupancy Category should not change. But in fact, many design professionals and code officials would classify the building as Category III since it now sits on campus. A check of several local structural engineering firms confirmed that every one of them interprets 1604.5 as requiring an Occupancy Category III for any building on campus with an occupant load greater than 500. This is very likely occurring on a national level as well. But this is not the intent of 1604.5.

Occupancy Category III addresses the extra risk associated with the presence of a large number of occupants concentrated in small areas, such as classrooms or lecture halls. There is nothing special about the act of teaching that warrants a Category III classification. The only reason it is referenced in 1604.5 is that teaching is usually done in groups, and it is the people in those groups to be protected. University laboratory and office buildings, with no classrooms, should not be subject to the 500 occupant threshold. The occupant load threshold for a Category III classification for a lab building is 5,000 occupants, not 500. Classification is a function of the building occupancy and not the property upon which the building sits.

The proposed new language clarifies the intent of the current regulation in three ways.

It emphasizes that it is the specific use of the building to be evaluated. Reference is made to both higher education, and a catch-all for any other adult educational building, to ensure that a broad scope of coverage is established.

Since the IBC does not define a “classroom,” the proposed change uses the basis for occupant load calculations as a handle to indentify spaces to be included in the analysis. Classrooms are calculated at 20 SF per person and this sets the benchmark for “high” occupant densities. Educational spaces with “low” densities such as teaching labs and vocational areas, at 50 SF per person, would not be included in the analysis. There are non-educational uses identified in Table 1004.1.1 which are also at 50 SF per person, and these are not subject to the 500 occupant threshold. Therefore, if the principle is to be applied consistently, this threshold should not apply to low density educational occupancies. Spaces with densities higher than a classroom will normally be classified as assembly space, but it’s possible that a classroom could have a density greater than 20, so this potential is also addressed with the phrase “or greater than.”

Since the specific risk being addressed occurs only in the classrooms, it is appropriate to use the summation of the occupant loads of just these rooms as the basis for the analysis, and not the total building occupant load. The limit should apply to those people associated with the higher risk, and should not include other general occupants of the building.

Cost Impact: The change would result in a major savings by minimizing the probability of future misapplication of this section. The cost from this problem isn’t immediately apparent; it is buried in the structural engineer’s calculations and the resulting overdesign. A recent project cost analysis for a 54,000 square foot laboratory building identified a **$1.2 million savings** ($22.71 / gsf) by changing the Occupancy Category from III to II, as shown in the
following.

VBI III Conceptual Estimate on Cost Premium for Category III Construction vs Category II

Description	Qty		Unit Cost		Total	Comments
Increase concrete reinforcing by 40%	147	ton	$ 2,500	/ton	$ 367,000	
Increase perimeter beam size by 20%	28	cyd	$ 1,200	/cyd	$ 33,333	
Increase for pre-cast concrete panel	10,056	sf	$ 4.00	/sf	$ 40,224	
Increase CMU wall reinforcing by 30%	2.93	ton	$ 2,500	/ton	$ 7,313	
Increase grout fill by 30%	88	cyd	$ 175	/cyd	$ 15,381	
Increase for Hokie Stone connections	13,600	sf	$ 10.00	/sf	$ 136,000	
Impact on roof screen	150.00	ton	$ 4,000	/ton	$ 400,000	
Increase for metal panel connections	1,103	sf	$ 3.00	/sf	$ 3,309	
Impact on curtain wall due to wind loading increase	15,935	sf	$ 5.00	/sf	$ 79,675	
Impact on acoustical ceiling supports	23,970	sf	$ 1.50	/sf	$ 35,955	
Impact on elevators and shaft						Possibility?
Impact on MEP for seismic supports	54,000	sf	$ 2.00	/sf	$ 108,000	Equipment associated changes need further discussion
					$1,226,190	
					$ 22.71	/gsf

$1.2 million was saved on just one building. Given how prevalent this misinterpretation is likely being made on colleges and universities nationwide, the net potential savings by correcting the problem is enormous.

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

ICCFILENAME: BADGER-S1-1604.5

S40-09/10
Table 1604.5, 1705.3.3, 1707.7

Proponent: Philip Brazil, PE, SE, representing self

Revise as follows:

TABLE 1604.5
OCCUPANCY CATEGORY OF BUILDINGS AND OTHER STRUCTURES

OCCUPANCY CATEGORY	NATURE OF OCCUPANCY
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300 Buildings and other structures containing elementary school, secondary school or day care facilities with an occupant load greater than 250 Buildings and other structures containing adult education facilities, such as colleges and universities, with an occupant load greater than 500 Group I-2 occupancies with an occupant load of 50 or more resident patients but not having surgery or emergency treatment facilities Group I-3 occupancies Any other occupancy with an occupant load greater than 5,000 [a] Power-generating stations, water treatment for potable water, waste water treatment facilities and other public utility facilities not included in Occupancy Category IV Buildings and other structures not included in Occupancy Category IV containing ~~sufficient~~ quantities of toxic or explosive ~~substances~~ materials that: Exceed maximum allowable quantities per control area as given in Table 307.1(1) or 307.1(2) or per outdoor control area in accordance with the *International Fire Code*; and Are sufficient to ~~be dangerous~~ pose a threat to the public if released [b]
IV	Buildings and other structures designated as essential facilities, including but not limited to: Group I-2 occupancies having surgery or emergency treatment facilities Fire, rescue, ambulance and police stations and emergency vehicle garages Designated earthquake, hurricane or other emergency shelters Designated emergency preparedness, communications, and operations centers and other facilities required for emergency response Power-generating stations and other public utility facilities required as emergency backup facilities for Occupancy Category IV structures Buildings and other structures containing quantities of highly toxic materials ~~as defined by Section 307 where the quantity of the material~~ that: Exceed~~s~~ ~~the~~ maximum allowable quantities ~~of~~ per control area as given in Table 307.1(2) or per outdoor control area in accordance with the *International Fire Code*; and Are sufficient to pose a threat to the public if released [b] Aviation control towers, air traffic control centers and emergency aircraft hangars Buildings and other structures having critical national defense functions Water storage facilities and pump structures required to maintain water pressure for fire suppression

(Portions of table not shown remain unchanged))

(No change to footnote a)

b. Where approved the building official, the classification of buildings and other structures as Occupancy Category III or IV based on their quantities of toxic, highly toxic or explosive materials is permitted to be reduced to Occupancy Category II, provided it can be demonstrated by a hazard assessment in accordance with Section 1.5.2 of ASCE 7 that a release of the toxic, highly toxic or explosive materials is not sufficient to pose a threat to the public.

1705.3.3 Seismic Design Category C. The following additional systems and components in structures assigned to Seismic Design Category C:

1. ~~Heating, ventilating and air-conditioning (HVAC)~~ Ductwork ~~containing~~ designed to carry hazardous materials and anchorage of such ductwork.
2. Piping systems ~~and mechanical units containing flammable, combustible or highly toxic~~ designed to carry hazardous materials and their associated mechanical units.
3. Anchorage of electrical equipment used for emergency or standby power systems.

1707.7 Mechanical and electrical components. Special inspection for mechanical and electrical equipment shall be as follows:

1. Periodic special inspection is required during the anchorage of electrical equipment for emergency or standby power systems in structures assigned to Seismic Design Category C, D, E or F;
2. Periodic special inspection is required during the ~~installation of~~ anchorage of other electrical equipment in structures assigned to Seismic Design Category E or F;
3. Periodic special inspection is required during the installation and anchorage of piping systems ~~intended~~ designed to carry ~~flammable, combustible or highly toxic contents~~ hazardous materials and their associated mechanical units in structures assigned to Seismic Design Category C, D, E or F;
4. Periodic special inspection is required during the installation and anchorage of ~~HVAC~~ ductwork ~~that will contain~~ designed to carry hazardous materials in structures assigned to Seismic Design Category C, D, E or F; and
5. Periodic special inspection is required during the installation and anchorage of vibration isolation systems in structures assigned to Seismic Design Category C, D, E or F where the construction documents require a nominal clearance of 1/4 inch (6.4 mm) or less between the equipment support frame and restraint.

Reason: The purpose for this proposal is to clarify the determination of occupancy category and the requirements for special inspection where hazardous materials are present. It was prepared in conjunction with ASCE 7 Proposal GPSC-5R2, which was approved by the General Subcommittee on March 1, 2009 and is being balloted by the Main Committee (Second Main Committee Ballot on General Requirements). It is expected that the Main Committee will approve the proposal.

Table 1604.5 currently classifies buildings and other structures containing certain quantities of toxic, highly toxic or explosive materials as Occupancy Category III or IV. The Category III classification applies to toxic and explosive materials and the threshold for the classification is subjective: quantities sufficient to be dangerous to the public if released. The Category IV classification applies to highly toxic materials and the threshold is objective: quantities exceeding the maximum allowable quantities of Table 307.1(2). Table 307.1(2) specifies maximum allowable quantities per control area for hazardous materials posing a health hazard.

Explosive materials are classified as posing a "physical hazard." Toxic and highly toxic materials are classified as posing a "health hazard." Materials that pose a physical hazard or a health hazard are classified as "hazardous materials." Refer to IBC Section 307.2 and IFC Section 2702.1 for definitions of these terms. The maximum quantities per control area are given in IBC Table 307.1(1) and IFC Table 2703.1.1(1) for hazardous materials posing a physical hazard and IBC Table 307.1(2) and IFC Table 2703.1.1(2) for hazardous materials posing a health hazard. The maximum quantities per outdoor control area are given in IFC Table 2703.1.1(3) for hazardous materials posing a physical hazard and IFC Table 2703.1.1(4) for hazardous materials posing a health hazard,.

A "control area" is defined in Section 307.2 as a space "within a building where quantities of hazardous materials not exceeding the maximum allowable quantities per control area are stored, used or handled." The effect of this definition on a Category IV classification is that it is limited to quantities of highly toxic materials within buildings. Not considered in the classification are quantities per "outdoor control area," which is defined in Section 2702.1 of the *International Fire Code* (IFC) as "an outdoor area that contains hazardous materials in amounts not exceeding the maximum allowable quantities of (IFC) Table 2703.1.1(3) (e.g., explosive materials) or Table 2703.1.1(4) (e.g., toxic and highly toxic materials)."

The intent in classifying buildings and other structures containing certain quantities of toxic, highly toxic or explosive materials as Occupancy Category III or IV is to reduce the potential for catastrophic release of these hazardous materials resulting from the failure of a building or structure (or a component conveying or supporting the materials and supported by a building or structure) to resist the structural demands of a design event, such as an earthquake. The required classification is limited to toxic, highly toxic and explosive materials because they pose the most serious threat to the general public if released. The threat being addressed is related to large-scale impacts on the general public, which can be characterized as global (e.g., beyond the boundaries of the site where the building or structure is located) rather than local (e.g., within those same boundaries).

Table 1604.5 currently classifies the building or structure as Occupancy Category III based on a subjective threshold but as Occupancy Category IV based on an objective threshold. This proposal revises the thresholds for both classifications so that two conditions are met for classification as Occupancy Category III or IV and they are summarized below. These revised thresholds are more consistent with the global threat discussed above. Similar thresholds are found in Table 1-1 of ASCE 7-10.

1.The quantities exceed maximum allowable quantities per control area within buildings or structures or per outdoor control area for outdoor areas; and

2. The quantities are sufficient to pose a threat to the public if released.

The first condition has the effect of exempting buildings or portions thereof from being classified as Occupancy Category III or IV except where they are classified as Group H. Where the quantities of hazardous materials in the control areas of a building or portion thereof are less than the maximum allowable quantities per control area, the occupancy classifications without considering the presence of hazardous materials are not affected by their presence. The first condition has the effect of exempting such buildings or portions thereof because the small quantities of hazardous materials permitted in occupancies other than Group H do not generally pose a global threat.

The second condition is subjective but the global threat posed by toxic, highly toxic and explosive materials is not easily quantified. Footnote (b) is added to Table 1604.5 permitting classification as Occupancy Category II for a building or structure otherwise classified as Occupancy Category III or IV, provided a hazard assessment in accordance with Section 1.5.2 of ASCE 7 is performed and it is demonstrated that a release of the toxic, highly toxic or explosive materials is not sufficient to pose a threat to the public. Refer to Section 1.5.2 and Commentary Section C1.5.2 of ASCE 7 for further information.

In Table 1604.5, a reference to IBC Table 307.1(1) is specified as well as Table 307.1(2). This is because explosive materials pose a physical hazard as discussed above but toxic and highly toxic materials pose a health hazard.

Also in Table 1604.5, maximum allowable quantities per outdoor control area are specified as well as maximum allowable quantities per control area. Hazardous materials pose physical or health hazards not only from being located in buildings but also from being located in structures not generally considered as buildings, such as tanks, towers, bins, hoppers, silos and similar structures. ASCE 7-10 distinguishes between "building structures" (e.g., buildings) and "nonbuilding structures." Refer to Chapter 11 of ASCE 7-10 for definitions of these terms.

In conjunction with Chapter 13 of ASCE 7-10 and the revisions to Table 1604.5 in this proposal, Sections 1705.3.3 and 1707.7 are also revised. All instances of terms related to hazardous materials in the structural chapters (Chapter 16 through 23) in the 2009 IBC are found in these sections. In Item #1 of Section 1705.3.3 and Item #4 of Section 1707.7, "HVAC" is deleted because HVAC ducts typically convey environmental air, not hazardous materials. At items in both sections, the change to "designed to carry" is for consistency with Chapter 13 of ASCE 7-10. The other revisions are either editorial or are intended to make the intent more clear.

Although Section 1705.3.3 is being modified in this proposal, the deletion of this section is the subject of a separate proposal. Should both proposals be approved by the ICC membership, it is not the intent of the proponent to retain Section 1705.3.3 in the 2012 IBC for the purpose of modifying the section in accordance with this proposal.

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

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

FILENAME: Brazil-S1-1604.5

S41–09/10

202, 403.2.3, 1602.1, 1603.1.4, 1603.1.5, 1604.5, Table 1604.5, 1604.5.1, 1609.1.2, 1613.2, 1613.5.6, Table 1613.5.6(1), Table 1613.5.6(2), 1614.1, 1704.5-1704.5.3, 1710.2, 1710.3, 1809.5, 2109.1.1, 2308.2, 3408.4; IEBC Table 101.5.4.1, 101.5.4.2, Table 101.5.4.2, 307.4, 907.3.1, 907.3.2, A102.2

THIS IS A 2 PART CODE CHANGE. BOTH PARTS WILL BE HEARD BY THE IBC STRUCTURAL CODE COMMITTEE AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDER FOR THE STRUCTURAL CODE COMMITTEE.

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

PART I- IBC STRUCTURAL

Revise as follows:

SECTION 202
DEFINITIONS

~~OCCUPANCY~~ RISK CATEGORY. See Section 1602.1.

403.2.3 Structural integrity of exit enclosures and elevator hoistway enclosures. For high-rise buildings of ~~occupancy~~ Risk Category III or IV in accordance with Section 1604.5, and for all buildings that are more than 420 feet (128 000 mm) in building height, exit enclosures and elevator hoistway enclosures shall comply with Sections 403.2.3.1 through 403.2.3.4.

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

~~OCCUPANCY~~ RISK CATEGORY. A category used to determine structural requirements based on occupancy.

1603.1.4 Wind design data. The following information related to wind loads shall be shown, regardless of whether wind loads govern the design of the lateral-force-resisting system of the building:

1. Basic wind speed (3-second gust), miles per hour (km/hr).
2. Wind importance factor, *I*, and ~~occupancy~~ risk category.
3. Wind exposure. Where more than one wind exposure is utilized, the wind exposure and applicable wind direction shall be indicated.
4. The applicable internal pressure coefficient.
5. Components and cladding. The design wind pressures in terms of psf (kN/m^2) to be used for the design of exterior component and cladding materials not specifically designed by the registered design professional.

1603.1.5 Earthquake design data. The following information related to seismic loads shall be shown, regardless of whether seismic loads govern the design of the lateral-force-resisting system of the building:

1. Seismic importance factor, I, and ~~occupancy~~ risk category.
2. Mapped spectral response accelerations, S_S and S_1.
3. Site class.
4. Spectral response coefficients, S_{DS} and S_{D1}.
5. Seismic design category.
6. Basic seismic-force-resisting system(s).
7. Design base shear.
8. Seismic response coefficient(s), C_S.
9. Response modification factor(s), R.
10. Analysis procedure used.

1604.5 ~~Occupancy~~ Risk Category. Each building and structure shall be assigned ~~an occupancy~~ a risk category in accordance with Table 1604.5. Where a referenced standard specifies an occupancy category, the risk category shall not be taken as lower than the occupancy category specified therein.

TABLE 1604.5
~~OCCUPANCY~~ RISK CATEGORY OF BUILDINGS AND OTHER STRUCTURES

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

(Portions of table not shown, remain unchanged)

1604.5.1 Multiple occupancies. Where a building or structure is occupied by two or more occupancies not included in the same ~~occupancy~~ risk category, it shall be assigned the classification of the highest ~~occupancy~~ risk category corresponding to the various occupancies. Where buildings or structures have two or more portions that are structurally separated, each portion shall be separately classified. Where a separated portion of a building or structure provides required access to, required egress from or shares life safety components with another portion having a higher ~~occupancy~~ risk category, both portions shall be assigned to the higher ~~occupancy~~ risk category.

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

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

Exceptions:

1. Wood structural panels with a minimum thickness of 7/16 inch (11.1 mm) and maximum panel span of 8 feet (2438 mm) shall be permitted for opening protection in one- and two-story buildings classified as Group R-3 or R-4 occupancy. Panels shall be precut so that they shall be attached to the framing surrounding the opening containing the product with the glazed opening. Panels shall be predrilled as required for the anchorage method and shall be secured with the attachment hardware provided. Attachments shall be designed to resist the components and cladding loads determined in accordance with the provisions of ASCE 7, with corrosion-resistant attachment hardware provided anchors permanently installed on the building. Attachment in accordance with Table 1609.1.2 with corrosion-resistant attachment hardware provided and anchors permanently installed on the building is permitted for buildings with a mean roof height of 45 feet (13 716 mm) or less where wind speeds do not exceed 140 mph (63 m/s).
2. Glazing in ~~Occupancy~~ Risk Category I buildings as defined in Table 1604.5, including greenhouses that are occupied for growing plants on a production or research basis, without public access shall be permitted to be unprotected.
3. Glazing in ~~Occupancy~~ Risk Category II, III or IV buildings located over 60 feet (18,288 mm) above the ground and over 30 feet (9144 mm) above aggregate surface roofs located within 1,500 feet (458 m) of the building shall be permitted to be unprotected.

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

SEISMIC DESIGN CATEGORY. A classification assigned to a structure based on its ~~occupancy~~ risk category and the severity of the design earthquake ground motion at the site.

1613.5.6 Determination of seismic design category. Structures classified as ~~Occupancy~~ Risk Category I, II or III that are located where the mapped spectral response acceleration parameter at 1-second period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category E. Structures classified as ~~Occupancy~~ Risk Category IV that are located where the mapped spectral response acceleration parameter at 1-second period, S_1, is greater than or equal to 0.75 shall be assigned to Seismic Design Category F. All other structures shall be assigned to a seismic design category based on their ~~occupancy~~ risk category and the design spectral response acceleration coefficients, S_{DS} and S_{D1}, determined in accordance with Section 1613.5.4 or the site-specific procedures of ASCE 7. Each building and structure shall be assigned to the more severe seismic design category in accordance with Table 1613.5.6(1) or 1613.5.6(2), irrespective of the fundamental period of vibration of the structure, *T*.

TABLE 1613.5.6(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD RESPONSE ACCELERATIONS

	~~OCCUPANCY~~ RISK CATEGORY		
VALUE OF S_{DS}	**I or II**	**III**	**IV**
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

TABLE 1613.5.6(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

VALUE OF S_{D1}	~~OCCUPANCY~~ RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$0.20g \leq S_{D1}$	D	D	D

1614.1 General. Buildings classified as high-rise buildings in accordance with Section 403 and assigned to ~~Occupancy~~ Risk Category III, or IV shall comply with the requirements of this section. Frame structures shall comply with the requirements of Section 1614.3. Bearing wall structures shall comply with the requirements of Section 1614.4.

1704.5 Masonry construction. Masonry construction shall be inspected and verified in accordance with the requirements of Sections 1704.5.1 through 1704.5.3, depending on the ~~occupancy~~ risk of the building or structure.

Exception: Special inspections shall not be required for:

1. Empirically designed masonry, glass unit masonry or masonry veneer designed by Section 2109, 2110 or Chapter 14, respectively, or by Chapter 5, 6 or 7 of TMS 402/ACI 530/ASCE 5, respectively, when they are part of structures classified as ~~Occupancy~~ Risk Category I, II or III in accordance with Section 1604.5.
2. Masonry foundation walls constructed in accordance with Table 1807.1.6.3(1), 1807.1.6.3(2), 1807.1.6.3(3) or 1807.1.6.3(4).
3. Masonry fireplaces, masonry heaters or masonry chimneys installed or constructed in accordance with Section 2111, 2112 or 2113, respectively.

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

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

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

1710.2 Structural observations for seismic resistance. Structural observations shall be provided for those structures assigned to Seismic Design Category D, E or F, as determined in Section 1613, where one or more of the following conditions exist:

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

1710.3 Structural observations for wind requirements. Structural observations shall be provided for those structures sited where the basic wind speed exceeds 110 mph (49 m/sec), determined from Figure 1609, where one or more of the following conditions exist:

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

4. When such observation is specifically required by the building official.

1809.5 Frost protection. Except where otherwise protected from frost, foundations and other permanent supports of buildings and structures shall be protected from frost by one or more of the following methods:

1. Extending below the frost line of the locality;
2. Constructing in accordance with ASCE-32; or
3. Erecting on solid rock.

   Exception: Free-standing buildings meeting all of the following conditions shall not be required to be protected:

   1. Assigned to ~~Occupancy~~ Risk Category I, in accordance with Section 1604.5;
   2. Area of 600 square feet (56 m^2) or less for light-frame construction or 400 square feet (37 m^2) or less for other than light-frame construction; and
   3. Eave height of 10 feet (3048 mm) or less.

Shallow foundations shall not bear on frozen soil unless such frozen condition is of a permanent character.

2109.1.1 Limitations. The use of empirical design of masonry walls shall be limited as noted in Section 5.1.2 of TMS 402/ACI 530/ASCE 5. The use of dry-stacked surface-bonded masonry shall be prohibited in ~~Occupancy~~ Risk 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 cripple walls not exceeding 14 inches (356 mm) in height need not be considered a story.

2. Maximum floor-to-floor height shall not exceed 11 feet, 7 inches (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/m^2) for combined roof and ceiling, exterior walls, floors and partitions.

   Exceptions:

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

   3.2. Live loads shall not exceed 40 psf (1916 N/m^2) for floors.
   3.3. Ground snow loads shall not exceed 50 psf (2395 N/m^2).
4. Wind speeds shall not exceed 100 miles per hour (mph) (44 m/s) (3-second gust).

   Exception: Wind speeds shall not exceed 110 mph (48.4 m/s) 3-second gust for buildings in Exposure Category B 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~~ Risk 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.

3408.4 Change of occupancy. When a change of occupancy results in a structure being reclassified to a higher ~~occupancy~~ risk category, the structure shall conform to the seismic requirements for a new structure of the higher ~~occupancy~~ risk category. Where the existing seismic force-resisting system is a type that can be designated ordinary, values of R, Ω_0 and C_d for the existing seismic force-resisting system shall be those specified by this code for an ordinary system unless it is demonstrated that the existing system will provide performance equivalent to that of a detailed, intermediate or special system.

Exceptions:

1. Specific seismic detailing requirements of this code or Section 1613 for a new structure shall not be required to be met where it can be shown that the level of performance and seismic safety is equivalent to that of a new structure. Such analysis shall consider the regularity, overstrength, redundancy and ductility of the structure within the context of the existing and retrofit (if any) detailing provided.
2. When a change of use results in a structure being reclassified from ~~Occupancy~~ Risk Category I or II to ~~Occupancy~~ Risk Category III and the structure is located in a seismic map area where $S_{DS} < 0.33$, compliance with the seismic requirements of this code and Section 1613 are not required.

PART II- IEBC

TABLE 101.5.4.1
PERFORMANCE CRITERIA FOR IBC LEVEL SEISMIC FORCES

~~OCCUPANCY~~ RISK CATEGORY (BASED ON IBC TABLE 1604.5)	PERFORMANCE LEVEL FOR USE WITH ASCE 41 BSE-1 EARTHQUAKE HAZARD LEVEL	PERFORMANCE LEVEL FOR USE WITH ASCE 41 BSE-2 EARTHQUAKE HAZARD LEVEL
I	Life safety (LS)	Collapse Prevention (CP)
II	Life safety (LS)	Collapse Prevention (CP)
III	Note a	Note a
IV	Immediate Occupancy (IO)	Life safety (LS)

a. Acceptance criteria for ~~Occupancy~~ Risk Category III shall be taken as 80 percent of the acceptance criteria specified for ~~Occupancy~~ Risk Category II performance levels, but need not be less than the acceptance criteria specified for ~~Occupancy~~ Risk Category IV performance levels.

101.5.4.2 Compliance with reduced IBC level seismic forces. Where seismic evaluation and design is permitted to meet reduced *International Building Code* seismic force levels, the procedures used shall be in accordance with one of the following:

1. The *International Building Code* using 75 percent of the prescribed forces. Values of R, Ω_0 and C_d used for analysis shall be as specified in Section 101.5.4.1 of this code.
2. Structures or portions of structures that comply with the requirements of the applicable chapter in Appendix A as specified in Items 2.1 through 2.5 shall be deemed to comply with this section.
   - 2.1. The seismic evaluation and design of unreinforced masonry bearing wall buildings in ~~Occupancy~~ Risk Category I or II are permitted to be based on the procedures specified in Appendix Chapter A1.
   - 2.2. Seismic evaluation and design of the wall anchorage system in reinforced concrete and reinforced masonry wall buildings with flexible diaphragms in ~~Occupancy~~ Risk Category I or II are permitted to be based on the procedures specified in Appendix Chapter A2.
   - 2.3. Seismic evaluation and design of cripple walls and sill plate anchorage in residential buildings of light-frame wood construction in ~~Occupancy~~ Risk Category I or II are permitted to be based on the procedures specified in Chapter A3.
   - 2.4. Seismic evaluation and design of soft, weak, or open-front wall conditions in multiunit residential buildings of wood construction in ~~Occupancy~~ Risk Category I or II are permitted to be based on the procedures specified in Chapter A4.
   - 2.5. Seismic evaluation and design of concrete buildings in all ~~occupancy~~ risk categories are permitted to be based on the procedures specified in Chapter A5.
3. Compliance with ASCE 31 based on the applicable performance level as shown in Table 101.5.4.2. It shall be permitted to use the BSE-1 earthquake hazard level as defined in ASCE 41 and subject to the limitations in Item 4 below.
4. Compliance with ASCE 41 using the BSE-1 Earthquake Hazard Level and the performance level shown in Table 101.5.4.2. The design spectral response acceleration parameters S_{XS} and S_{X1} specified in ASCE 41

shall not be taken less than seventy-five percent of the respective design spectral response acceleration parameters S_{DS} and S_{D1} defined by the *International Building Code*.

TABLE 101.5.4.2
PERFORMANCE CRITERIA FOR REDUCED IBC LEVEL SEISMIC FORCES

~~OCCUPANCY~~ RISK CATEGORY (BASED ON IBC TABLE 1604.5)	PERFORMANCE LEVEL FOR USE WITH ASCE 31	PERFORMANCE LEVEL FOR USE WITH ASCE 41 BSE-1 EARTHQUAKE HAZARD LEVEL
I	Life safety (LS)	Life safety (LS)
II	Life safety (LS)	Life safety (LS)
III	Note a	Note a
IV	Immediate Occupancy (IO)	Immediate Occupancy (IO)

a. Acceptance criteria for ~~Occupancy~~ Risk Category III shall be taken as 80 percent of the acceptance criteria specified for ~~Occupancy~~ Risk Category II performance levels, but need not be less than the acceptance criteria specified for ~~Occupancy~~ Risk Category IV performance levels.

b. For ~~Occupancy~~ Risk Category III, the ASCE 31 screening phase checklists shall be based on the life safety performance level.

[B] 307.4 Structural. When a *change of occupancy* results in a structure being reclassified to a higher ~~occupancy~~ risk category, the structure shall conform to the seismic requirements for a new structure of the higher ~~occupancy~~ risk category. Where the existing seismic force-resisting system is a type that can be designated ordinary, values of R, Ω_0 and C_d for the existing seismic force-resisting system shall be those specified by this code for an ordinary system unless it is demonstrated that the existing system will provide performance equivalent to that of a detailed, intermediate or special system.

Exceptions:

1. Specific seismic detailing requirements of this code or ASCE 7 for a new structure shall not be required to be met where it can be shown that the level of performance and seismic safety is equivalent to that of a new structure. Such analysis shall consider the regularity, overstrength, redundancy and ductility of the structure within the context of the existing and retrofit (if any) detailing provided.
2. When a change of use results in a structure being reclassified from ~~Occupancy~~ Risk Category I or II to ~~Occupancy~~ Risk Category III and the structure is located in a seismic map area where S_{DS} < 0.33, compliance with the seismic requirements of this code and Section 1613 of the *International Building Code* are not required.

907.3.1 Compliance with the *International Building Code* level seismic forces. Where a building or portion thereof is subject to a change of occupancy that results in the building being assigned to a higher ~~occupancy~~ risk category based on Table 1604.5 of the *International Building Code*; or where such change of occupancy results in a reclassification of a building to a higher hazard category as shown in Table 912.4; or where a change of a Group M occupancy to a Group A, E, I-1 R-1, R-2 or R-4 occupancy with two-thirds or more of the floors involved in Level 3 alteration work, the building shall comply with the requirements for *International Building Code* level seismic forces as specified in Section 101.5.4.1 for the new ~~occupancy~~ risk category.

Exceptions:

1. Group M occupancies being changed to Group A, E, I-1, R-1, R-2 or R-4 occupancies for buildings less than six stories in height and in Seismic Design Category A, B or C.
2. Where approved by the code official, specific detailing provisions required for a new structure are not required to be met where it can be shown that an equivalent level of performance and seismic safety is obtained for the applicable ~~occupancy~~ risk category based on the provision for reduced *International Building Code* level seismic forces as specified in Section 101.5.4.2.
3. Where the area of the new occupancy with a higher hazard category is less than or equal to 10 percent of the total building floor area and the new occupancy is not classified as ~~Occupancy~~ Risk Category IV. For the purposes of this exception, buildings occupied by two or more occupancies not included in the same ~~occupancy~~ risk category shall be subject to the provisions of Section 1604.5.1 of the *International Building Code*. The cumulative effect of the area of occupancy changes shall be considered for the purposes of this exception.
4. Unreinforced masonry bearing wall buildings in ~~Occupancy~~ Risk Category III when assigned to Seismic Design Category A or B shall be allowed to be strengthened to meet the requirements of Appendix Chapter A1 of this code [*Guidelines for the Seismic Retrofit of Existing Buildings* (GSREB)].

907.3.2 Access to ~~Occupancy~~ Risk Category IV. Where a change of occupancy is such that compliance with Section 907.3.1 is required and the building is assigned to ~~Occupancy~~ Risk Category IV, the operational access to the building shall not be through an adjacent structure unless the adjacent structure conforms to the requirements for ~~Occupancy~~ Risk Category IV structures. Where operational access is less than 10 feet (3048 mm) from either an interior lot line or from another structure, access protection from potential falling debris shall be provided by the owner of the ~~Occupancy~~ Risk Category IV structure.

A102.2 Essential and hazardous facilities. The provisions of this chapter shall not apply to the strengthening of buildings or structures in ~~Occupancy~~ Risk Category III when assigned to Seismic Design Category C, D, or E or buildings or structures in ~~Occupancy~~ Risk Category IV. Such buildings or structures shall be strengthened to meet the requirements of the *International Building Code* for new buildings of the same ~~occupancy~~ risk category or other such criteria that have been established by the jurisdiction.

Reason: The purpose for this proposal is to correlate the IBC and IEBC with the 2010 edition of ASCE 7. The need for correlation is due to ASCE 7 Proposal GPSC-3AR2, which was approved by the General Subcommittee on March 1, 2009 and is being balloted by the Main Committee (Item #10 of the Second Main Committee Ballot on General Requirements). It is expected that the Main Committee will approve the proposal.
The proposed changes will reduce confusion between occupancies and their classifications utilized by the codes promulgated by the International Code Council in their fire- and life-safety regulations, and occupancy considerations associated with structural safety. In addition, this proposal recognizes that the factor determining whether a building or other structure falls into a specific Category is not specifically its occupancy, but rather, the risk that failure would pose to persons and society at large. Indeed, some of the structures that fall under the provenance of the IBC and IEBC are unoccupied, such as water treatment plants, but still fall into Category III because their failure would pose a substantial risk to the public.
In Section 1604.5, a statement is added to assign a risk category to an occupancy category specified in a referenced standard. A similar statement is being added to Section 1.5.1 of ASCE 7-10 by Proposal GPSC-3AR1. This is being done in the event that a standard referenced by the 2012 IBC has not updated to be consistent with the change from “occupancy category” to “risk category.”
All instances of “occupancy category” in the 2009 IBC and IEBC are included in this proposal.

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

Part I-IBC

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

Part II-IEBC

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

ICCFILENAME: Brazil-S46-1604.5

S42–09/10
Table 1604.5

Proponent: Michael Mahoney, Federal Emergency Management Agency and Ron Lynn, Clark County, Nevada representing Western States Seismic Policy Council Code Committee

Revise as follows:

TABLE 1604.5
OCCUPANCY CATEGORY OF BUILDINGS AND OTHER STRUCTURES

OCCUPANCY CATEGORY	NATURE OF OCCUPANCY
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure, including but not limited to: • Buildings and other structures whose primary occupancy is public assembly with an occupant load greater than 300. ~~• Buildings and other structures containing elementary school, secondary school or day care facilities with an occupant load greater than 250.~~ • Buildings and other structures containing adult education facilities, such as colleges and universities, with an occupant load greater than 500. • Group I-2 occupancies with an occupant load of 50 or more resident patients but not having surgery or emergency treatment facilities. • Group I-3 occupancies. • Any other occupancy with an occupant load greater than 5,000[a]. • Power-generating stations, water treatment facilities for potable water, waste water treatment facilities and other public utility facilities not included in Occupancy Category IV. • Buildings and other structures not included in Occupancy Category IV containing sufficient quantities of toxic or ex-plosive substances to be dangerous to the public if released.
IV	Buildings and other structures designated as essential facilities, including but not limited to: • Group I-2 occupancies having surgery or emergency treatment facilities. • Fire, rescue, ambulance and police stations and emergency vehicle garages. • Designated ~~earthquake, hurricane or other~~ emergency shelters, including elementary school, secondary school or day care facilities with an occupant load greater than 100. • Designated emergency preparedness, communications and operations centers and other facilities required for emergency response. • Power-generating stations and other public utility facilities required as emergency backup facilities for Occupancy Category IV structures. • Structures containing highly toxic materials as defined by Section 307 where the quantity of the material exceeds the maximum allowable quantities of Table 307.1(2). • Aviation control towers, air traffic control centers and emergency aircraft hangars. • Buildings and other structures having critical national defense functions. • Water storage facilities and pump structures required to maintain water pressure for fire suppression.

(Portions of table not shown remain unchanged)
(No change to footnote)

Reason: This proposal makes two changes. The first is to eliminate the list of hazards, since emergency shelters are normally designed to withstand all applicable natural and man-made hazards and are very rarely designated as hazard specific. The second and more significant change is to specifically include elementary, secondary and day care school buildings. The reason for this second change is that 1) most communities have designated their school buildings as community shelters in their emergency operations plans, and 2) almost every school district in the country has instituted a policy of keeping students sheltered in place if disaster strikes during school hours until parents can pick up their children, effectively making school buildings shelters for their students. Further, the impact of losses of school buildings in natural disaster was made painfully aware to the entire world in the China earthquake last year. This country also has its school-related disaster, which was the 1933 Long Beach earthquake, which collapsed a significant percentage of school buildings. Fortunately, the buildings were not occupied at the time, but the disaster led the State of California to adopt the Field Act, which to this day mandates special construction requirements for school buildings.

Cost Impact: The only potential cost impact would potentially be a slight increase to the cost of new schools. However, past similar studies on seismic design by FEMA have shown that the increase would be less than one percent of the total cost of the structure.

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

ICCFILENAME: MAHONEY-S3-TABLE 1604

S43–09/10

1604.7, 1702.1, 1703.4, Table 1704.5.1, Table 1704.5.3, 1714.2, 1714.3.1, 1715.1, 1715.2, 1715.3, 2303.4.7, 2407.1, 2408.2.1, 2408.3

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

Revise as follows:

1604.7 Preconstruction load tests. Materials and methods of construction that are not capable of being designed by approved engineering analysis or that do not comply with the applicable ~~material design~~ referenced standards ~~listed in Chapter 35~~, or alternative test procedures in accordance with Section 1712, shall be load tested in accordance with Section 1715.

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.

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

1703.4 Performance. Specific information consisting of test reports conducted by an approved testing agency in accordance with the appropriate referenced standards ~~referenced in Chapter 35~~, or other such information as necessary, shall be provided for the building official to determine that the material meets the applicable code requirements.

TABLE 1704.5.1
LEVEL 1 REQUIRED VERIFICATION AND INSPECTION OF MASONRY CONSTRUCTION
(No changes to Table)

For SI: °C = (°F - 32)/1.8.
~~a. The specific standards referenced are those listed in Chapter 35.~~

TABLE 1704.5.3
LEVEL 2 REQUIRED VERIFICATION AND INSPECTION OF MASONRY CONSTRUCTION
(No changes to Table)

For SI: °C = (°F - 32)/1.8.
~~a. The specific standards referenced are those listed in Chapter 35.~~

1714.2 Test standards. Structural components and assemblies shall be tested in accordance with the appropriate ~~material~~ referenced standards ~~listed in Chapter 35~~. In the absence of a standard that contains an applicable load test procedure, the test procedure shall be developed by a registered design professional and approved. The test procedure shall simulate loads and conditions of application that the completed structure or portion thereof will be subjected to in normal use.

1714.3.1 Load test procedure specified. Where a referenced standard ~~listed in Chapter 35~~ contains an applicable load test procedure and acceptance criteria, the test procedure and acceptance criteria in the standard shall apply. In the absence of specific load factors or acceptance criteria, the load factors and acceptance criteria in Section 1714.3.2 shall apply.

1715.1 General. In evaluating the physical properties of materials and methods of construction that are not capable of being designed by approved engineering analysis or do not comply with the applicable ~~material design~~ referenced standards ~~listed in Chapter 35~~, the structural adequacy shall be predetermined based on the load test criteria established in this section.

1715.2 Load test procedures specified. Where specific load test procedures, load factors and acceptance criteria are included in the applicable ~~design~~ referenced standards ~~listed in Chapter 35~~, such test procedures, load factors and acceptance criteria shall apply. In the absence of specific test procedures, load factors or acceptance criteria, the corresponding provisions in Section 1715.3 shall apply.

1715.3 Load test procedures not specified. Where load test procedures are not specified in the applicable ~~design~~ referenced standards ~~listed in Chapter 35~~, the load-bearing and deformation capacity of structural components and assemblies shall be determined on the basis of a test procedure developed by a registered design professional that simulates applicable loading and deformation conditions. For components and assemblies that are not a part of the seismic-load-resisting system, the test shall be as specified in Section 1715.3.1. Load tests shall simulate the applicable loading conditions specified in Chapter 16.

2303.4.7 Truss quality assurance. Trusses not part of a manufacturing process in accordance with either Section 2303.4.6 or a referenced standard ~~listed in Chapter 35~~, which provides requirements for quality control done under the supervision of a third-party quality control agency, shall be manufactured in compliance with Sections 1704.2 and 1704.6, as applicable.

2407.1 Materials. Glass used as a handrail assembly or a guard section shall be constructed of either single fully tempered glass, laminated fully tempered glass or laminated heat-strengthened glass. Glazing in railing in-fill panels shall be of an approved safety glazing material that conforms to the provisions of Section 2406.1.1. For all glazing types, the minimum nominal thickness shall be 1/4 inch (6.4 mm). Fully tempered glass and laminated glass shall comply with Category II of CPSC 16 CFR 1201 or Class A of ANSI Z97.1~~, listed in Chapter 35~~.

2408.2.1 Testing. Test methods and loads for individual glazed areas in racquetball and squash courts subject to impact loads shall conform to those of CPSC 16 CFR 1201 or ANSI Z97.1~~, listed in Chapter 35,~~ with impacts being applied at a height of 59 inches (1499 mm) above the playing surface to an actual or simulated glass wall installation with fixtures, fittings and methods of assembly identical to those used in practice.

Glass walls shall comply with the following conditions:

1. A glass wall in a racquetball or squash court, or similar use subject to impact loads, shall remain intact following a test impact.
2. The deflection of such walls shall not be greater than 1-1/2 inches (38 mm) at the point of impact for a drop height of 48 inches (1219 mm).

Glass doors shall comply with the following conditions:

1. Glass doors shall remain intact following a test impact at the prescribed height in the center of the door.
2. The relative deflection between the edge of a glass door and the adjacent wall shall not exceed the thickness of the wall plus 1/2 inch (12.7 mm) for a drop height of 48 inches (1219 mm).

2408.3 Gymnasiums and basketball courts. Glazing in multipurpose gymnasiums, basketball courts and similar athletic facilities subject to human impact loads shall comply with Category II of CPSC 16 CFR 1201 or Class A of ANSI Z97.1~~, listed in Chapter 35~~.

Reason: The purpose for this proposal is to correlate the references to referenced standards with their charging text in Section 102.4 and to eliminate superfluous text. A "referenced standard" is understood to be a standard listed in Chapter 35, the title of which is "Referenced Standards," and statements that they are "listed in Chapter 35" are not needed in the IBC. All such references to Chapter 35 in the IBC are included in this proposal.

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

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

ICCFILENAME: Brazil-S36-1604.7

S44–09/10
1604.8.2, 1613.7

Proponent: Philip Brazil, PE, SE, Reid Middleton, Inc., representing self
Jim Rossberg, SEI of ASCE, representing self

1. Revise as follows:

1604.8.2 Structural walls. Walls that provide vertical load bearing resistance or lateral shear resistance for a portion of the structure shall be anchored to the roof and to all floors~~, roofs~~ and ~~other structural elements~~ members that provide lateral support for the wall or that are supported by the wall. ~~Such anchorage shall provide a positive direct connection~~ The connections shall be capable of resisting the horizontal forces specified in ~~this chapter but not less than the minimum strength design horizontal force specified in~~ Section ~~11.7.3~~ 1.4.4 of ASCE 7~~, substituted for "*E*" in the load combinations of Section 1605.2 or 1605.3~~ for walls of structures assigned to Seismic Design Category A and to Section 12.11 of ASCE 7 for walls of all other structures. Concrete and masonry walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet (1219 mm). Required anchors in masonry walls of hollow units or cavity walls shall be embedded in a reinforced grouted structural element of the wall. See Section 1609 for wind design requirements and see Section 1613 for earthquake design requirements.

2. Delete without substitution:

~~**1613.7 ASCE 7, Section 11.7.5.** Modify ASCE 7, Section 11.7.5 to read as follows:~~

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

.

Reason:
(BRAZIL)-The purpose for this proposal is to delete a revision to ASCE 7-05 that will no longer be needed because a similar revision will have been incorporated into the 2010 edition of ASCE 7. Section 1604.8.2 is also revised for consistency with this and other related revisions to ASCE 7. These are being accomplished by ASCE 7 Proposal GPSC-2R2, which was approved by the General Subcommittee on March 1, 2009 and is being balloted by the Main Committee (Item #2 of the Second Main Committee Ballot on General Requirements); and by ASCE 7 Proposal SSC TC-4-CH14-07-R1, which was approved by the Seismic Subcommittee on May 15, 2009 and is being balloted by the Main Committee (Item #1 of the Seventh Main Committee Ballot on Seismic Provisions). It is expected that the Main Committee will approve both proposals.
(ROSSBERG)- This provision has been considered and approved by the Seismic Subcommittee of ASCE 7 for inclusion into the 2010 edition of ASCE 7 hence with the adoption of ASCE 7-10 by reference this provision becomes duplicative. As of the submission date of this code change, the ASCE 7 Standards Committee is completing the committee balloting portion of the 2010 edition of ASCE/SEI 7. The document is designated ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* and it is expected that it will be completed and available for purchase prior to the ICC Final Action Hearings in May of 2010 . Any person interested in obtaining a public comment copy of ASCE/SEI 7-10 may do so by contacting the proponent at jrossberg@asce.org.

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

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

ICCFILENAME: Brazil-Rossberg-S23-1604.8.2

S45–09/10
1604.9

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

Revise as follows:

1604.9 Counteracting structural actions. Structural members, systems, components and cladding shall be designed to resist forces due to earthquake and wind, with consideration of overturning, sliding and uplift. Continuous load paths shall be provided for transmitting these forces to the foundation. Where sliding is used to isolate the elements, the effects of friction between sliding elements shall be included as a force. Where all or a portion of the resistance to these forces is provided by dead load, the dead load shall be taken as the minimum dead load likely to be in place during the event causing the considered forces. Consideration shall be given to the effects of vertical and horizontal deflections resulting from such forces.

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. The added text will align IBC Section 1604.9 with Section 1.3.5 of ASCE 7.

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

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

ICCFILENAME: Brazil-S40-1604.9

S46–09/10

1604.11 (New)

Proponent: Daniel J. Walker, PE, Thomas Associates, Inc. representing the National Sunroom Association

Add new text as follows:

<u>**1604.11 Patio cover design loads**. *Patio covers* shall be designed and constructed to sustain, within the stress limits of this code, all dead loads plus a minimum vertical live load of 10 pounds per square foot (0.48 kN/m^2) except that snow loads shall be used where such snow loads exceed this minimum. Such *patio covers* shall be designed to resist the minimum wind and seismic loads set forth in this code.</u>

Reason: This language has long been included in Appendix I of the code. The requirements are specific to this section and therefore should be included here.

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

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

ICCFILENAME: Walker-S1-1604.11

S47–09/10

1605.1

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

Revise as follows:

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

1. The load combinations specified in Section 1605.2, 1605.3.1 or 1605.3.2,
2. The load combinations specified in Chapters 18 through 23, and
3. The load combinations with overstrength factor specified in Section 12.4.3.2 of ASCE 7 where required by Section 12.2.5.2, 12.3.3.3 or 12.10.2.1 of ASCE 7. With the simplified procedure of ASCE 7 Section 12.14, the load combinations with overstrength factor of Section 12.14.3.2 of ASCE 7 shall be used.

Applicable loads shall be considered, including both earthquake and wind, in accordance with the specified load combinations. Each load combination shall also be investigated with one or more of the variable loads set to zero. <u>Where members, elements or components of a structure are designed in accordance with a particular material chapter of this code or a reference standard, they shall be designed to resist all applicable load combinations of Section 1605.2 or of Section 1605.3.</u>

Where the load combinations with overstrength factor in Section 12.4.3.2 of ASCE 7 apply, they shall be used as follows:

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

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. The need for correlation is due to ASCE 7 Proposal LCSC LC-12-R1, which has been approved by the Load Combinations Subcommittee and is being balloted by the Main Committee (Item #22 of the Second Main Committee Ballot on Load Combinations). It is expected that the Main Committee will approve the proposal.

Where proportioning members, elements or components of a particular construction material, Section 2.1 of ASCE-05 requires, throughout the structure, the exclusive use of the load combinations for strength/LRFD design or for allowable stress design (ASD). This requirement does not exist in the 2009 IBC but the modification to Section 2.1 in ASCE 7-10 warrants its inclusion in the 2012 IBC.

ASCE 7 has contained the requirement in Section 2.1 since the introduction of load combinations for strength/LRFD design in the 1982 edition of the standard. The primary reason was to avoid an indiscriminate mixture of strength and allowable stress design methods, which may lead to unpredictable structural system performance. The reliability analyses and code calibrations leading to the strength/LRFD load combinations were based on member limit states rather than system limit states.

The change to Section 2.1 of ASCE 7 acknowledges current industry practice while continuing to prohibit indiscriminate mixing of strength/LRFD and ASD load combinations where it is not appropriate. For example, the design of cold-formed steel framing and open web steel joists is typically done using ASD load combinations but the design of the structural steel is typically done using strength/LRFD load combinations. The AISC Code of Standard Practice indicates that cold-formed steel and steel joists are not considered structural steel. Foundations are also commonly designed at the soil-structure interface using ASD load combinations but the design of the concrete is typically done using strength/LRFD load combinations.

The change permits the mixing of strength/LRFD and ASD load combinations for the design of a structure, provided mixing does not occur for design of the individual classes of members, elements or components (e.g., cold-formed steel, steel joists and structural steel) in accordance with their respective material standards.

The addition to Section 1605.1 will make the 2012 IBC consistent with ASCE 7-10 with respect to this aspect of structural design but the text in this proposal does not match the corresponding text in ASCE 7-10. It has been adapted for inclusion in the IBC. For comparison purposes, the text from ASCE 7 Proposal LCSC LC-12-R1 is: "Where elements of a structure are designed by a particular material standard or specification, they shall be designed exclusively by either Section 2.3 or 2.4."

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

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

ICCFILENAME: Brazil-S31-1605. 1

S48–09/10

1605.1, 1810.3.6.1, 1810.3.9.4, 1810.3.11.2, 1810.3.12

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

Revise as follows:

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

1. The load combinations specified in Section 1605.2, 1605.3.1 or 1605.3.2,
2. The load combinations specified in Chapters 18 through 23, and
3. The ~~load combinations with~~ seismic load effects including overstrength factor ~~specified~~ in accordance with Section ~~12.4.3.2~~ 12.4.3 of ASCE 7 where required by Section 12.2.5.2, 12.3.3.3 or 12.10.2.1 of ASCE 7. With the simplified procedure of ASCE 7 Section 12.14, the ~~load combinations with~~ seismic load effects including overstrength factor ~~of~~ in accordance with Section 12.14.3.2 of ASCE 7 shall be used.

Applicable loads shall be considered, including both earthquake and wind, in accordance with the specified load combinations. Each load combination shall also be investigated with one or more of the variable loads set to zero.

Where the load combinations with overstrength factor in Section 12.4.3.2 of ASCE 7 apply, they shall be used as follows:

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

1810.3.6.1 Seismic Design Categories C through F. For structures assigned to Seismic Design Category C, D, E, or F, splices of deep foundation elements shall develop the lesser of the following:

1. The ~~full~~ nominal strength of the deep foundation element; and
2. The axial and shear forces and moments from the ~~load combinations with~~ seismic load effects including overstrength factor in accordance with Section 12.4.3.2 or 12.14.3.2 of ASCE 7.

1810.3.9.4 Seismic reinforcement. Where a structure is assigned to Seismic Design Category C reinforcement shall be provided in accordance with Section 1810.3.9.4.1. Where a structure is assigned to Seismic Design Category D, E, or F reinforcement shall be provided in accordance with Section 1810.3.9.4.2.

Exceptions:

1. Isolated deep foundation elements supporting posts of Group R-3 and U occupancies not exceeding two stories of light-frame construction shall be permitted to be reinforced as required by rational analysis but with not less than one No. 4 bar, without ties or spirals, where detailed so the element is not subject to lateral loads and the soil provides adequate lateral support in accordance with Section 1810.2.1.
2. Isolated deep foundation elements supporting posts and bracing from decks and patios appurtenant to Group R-3 and U occupancies not exceeding two stories of light-frame construction shall be permitted to be reinforced as required by rational analysis but with not less than one No. 4 bar, without ties or spirals, where the lateral load, *E*, to the top of the element does not exceed 200 pounds (890 N) and the soil provides adequate lateral support in accordance with Section 1810.2.1.
3. Deep foundation elements supporting the concrete foundation wall of Group R-3 and U occupancies not exceeding two stories of light-frame construction shall be permitted to be reinforced as required by rational analysis but with not less than two No. 4 bars, without ties or spirals, where the design cracking moment determined in accordance with Section 1810.3.9.1 exceeds the required moment strength determined using the load combinations with overstrength factor in Section 12.4.3.2 or 12.14.3.2 of ASCE 7 and the soil provides adequate lateral support in accordance with Section 1810.2.1.
4. Closed ties or spirals where required by Section 1810.3.9.4.2 shall be permitted to be limited to the top 3 feet (914 mm) of deep foundation elements 10 feet (3048 mm) or less in depth supporting Group R-3 and U occupancies of Seismic Design Category D, not exceeding two stories of light-frame construction.

1810.3.11.2 Seismic Design Categories D through F. For structures assigned to Seismic Design Category D, E, or F in accordance with Section 1613, deep foundation element resistance to uplift forces or rotational restraint shall be provided by anchorage into the pile cap, designed considering the combined effect of axial forces due to uplift and bending moments due to fixity to the pile cap. Anchorage shall develop a minimum of 25 percent of the strength of the element in tension. Anchorage into the pile cap shall ~~be capable of developing~~ comply with the following:

1. In the case of uplift, the anchorage shall be capable of developing the least of the following:
   1.1. The nominal tensile strength of the longitudinal reinforcement in a concrete element;
   1.2. The nominal tensile strength of a steel element; and
   1.3. The frictional force developed between the element and the soil multiplied by 1.3~~; and~~.

   Exception: The anchorage is permitted to be designed to resist the axial tension force resulting from the ~~load combinations with~~ seismic load effects including overstrength factor in accordance with Section 12.4.3.2 or 12.14.3.2 of ASCE 7.

2. In the case of rotational restraint, the ~~lesser of the following:~~ anchorage shall be designed to resist the axial ~~force,~~ and shear forces, and ~~bending~~ moments resulting from the ~~load combinations with~~ seismic load effects including overstrength factor in accordance with Section 12.4.3.2 or 12.14.3.2 of ASCE 7; ~~or development of~~ or shall be capable of developing the full axial, bending and shear nominal strength of the element.

Where the vertical lateral-force-resisting elements are columns, the pile cap flexural strengths shall exceed the column flexural strength. The connection between batter piles and pile caps shall be designed to resist the nominal strength of the pile acting as a short column. Batter piles and their connection shall be ~~capable of resisting forces and moments from the load combinations with~~ designed to resist the seismic load effects including overstrength factor in accordance with Section 12.4.3.2 or 12.14.3.2 of ASCE 7.

1810.3.12 Grade beams. For structures assigned to Seismic Design Category D, E, or F in accordance with Section 1613, grade beams shall comply with the provisions in Section 21.12.3 of ACI 318 for grade beams, except where they ~~have the capacity~~ are designed to resist the ~~forces from the load combinations with~~ seismic load effects including overstrength factor in accordance with Section 12.4.3.2 or 12.14.3.2 of ASCE 7.

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. The need for correlation is due to ASCE 7 Proposal SSC TC-2-CH12-63-R1, which was approved by the Seismic Subcommittee on 12/10/08 and is being balloted by the Main Committee (Item #23 of the Fourth Main Committee Ballot on Seismic Provisions). It is expected that the Main Committee will approve the proposal.
Proposal SSC-TC-2-CH12-63-R1 changes several references in ASCE 7 from Section 12.4.3.2 to Section 12.4.3 (and from Section 12.14.3.2.2 to Section 12.14.3.2 for the simplified procedure) so that the Exception to Section 12.4.3.1 (and to Section 12.14.3.2.1 for the simplified procedure)

permitting the maximum force that can be developed in the element to substitute for the horizontal seismic effects with overstrength is included. In the process of doing this, “load combinations with overstrength factor” was changed to “seismic load effects including overstrength factor” throughout ASCE 7 for consistency with the subject (and title) of Section 12.4.3 (and Section 12.14.3.2 for the simplified procedure).
This proposal revises the IBC for consistency with the revisions to ASCE 7 in Proposal SSC-TC-2-CH12-63-R1. In Item 3 of Paragraph #1 in Section 1605.1, the references to Sections 12.2.5.2, 12.3.3.3 and 12.10.2.1 of ASCE 7-05 will not be changing in ASCE 7-10. The references to Sections 12.4.3 and 12.14.3.2 of ASCE 7-05 throughout this proposal will also not be changing in ASCE 7-10. The IBC sections in this proposal correspond to the following sections of ASCE 7:

1. IBC Section 1810.3.6.1 corresponds to Section 12.13.6.6 of ASCE 7.
2. IBC Section 1810.3.11.2 corresponds to Section 12.13.6.5 of ASCE 7.
3. IBC Section 1810.3.12 corresponds to Section 14.2.3.2.2 of ASCE 7.

All instances of “load combinations with overstrenght factor” in the 2009 IBC are included in this proposal.

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

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

ICCFILENAME: Brazil-S47-1605.1

S49–09/10
1605.2.1, 1605.3.1

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

Revise as follows:

1605.2.1 Basic load combinations. Where strength design or load and resistance factor design is used, structures and portions thereof shall resist the most critical effects from the following combinations of factored loads:

$1.4\ (D + F)$ **(Equation 16-1)**

$1.2\ (D + F + T) + 1.6\ (L + H) +$
$0.5\ (L_r \text{ or } S \text{ or } R)$ **(Equation 16-2)**

$1.2\ \underline{(}D\ \underline{+ F)} + 1.6\ (L_r \text{ or } S \text{ or } R)\ \underline{+ 1.6\ H} + (f_1 L \text{ or } 0.8\ W)$ **(Equation 16-3)**

$1.2\ \underline{(}D\ \underline{+ F)} + 1.6\ W + f_1 L\ \underline{+ 1.6\ H} + 0.5\ (L_r \text{ or } S \text{ or } R)$ **(Equation 16-4)**

$1.2\ \underline{(}D\ \underline{+ F)} + 1.0\ E + f_1 L\ \underline{+ 1.6\ H} + f_2 S$ **(Equation 16-5)**

$0.9\ D + 1.6\ W + 1.6\ H$ **(Equation 16-6)**

$0.9\ D + 1.0\ E + 1.6\ H$ **(Equation 16-7)**

where:

f_1 = 1 for floors in places of public assembly, for live loads in excess of 100 pounds per square foot (4.79 kN/m^2), and for parking garage live load, and
= 0.5 for other live loads.
f_2 = 0.7 for roof configurations (such as saw tooth) that do not shed snow off the structure, and
= 0.2 for other roof configurations.

Exceptions:

1. Where other factored load combinations are specifically required by ~~the~~ other provisions of this code, such combinations shall take precedence.
2. Where the effect of H resists the primary variable load effect, a load factor of 0.9 shall be included with H where H is permanent and H shall be set to zero for all other conditions.

1605.3.1 Basic load combinations. Where allowable stress design (working stress design), as permitted by this code, is used, structures and portions thereof shall resist the most critical effects resulting from the following combinations of loads:

$D + F$ **(Equation 16-8)**

$D + H + F + L + T$ **(Equation 16-9)**

$D + H + F + (L_r \text{ or } S \text{ or } R)$ **(Equation 16-10)**

$D + H + F + 0.75\ (L + T) +$
$0.75\ (L_r \text{ or } S \text{ or } R)$ **(Equation 16-11)**

$D + H + F + (W \text{ or } 0.7\ E)$ **(Equation 16-12)**

$D + H + F + 0.75$ ~~(~~*W* ~~or 0.7 E)~~ +
0.75 *L* + 0.75 (L_r *or S or R*) **(Equation 16-13)**

$D + H + F + 0.75\ (0.7\ E) + 0.75\ L + 0.75\ S$ **(Equation 16-14)**

$0.6\ D + W + H$ **(Equation 16-~~14~~15)**

$0.6\ D + 0.7\ E + H$ **(Equation 16-~~15~~16)**

(Renumber remaining equations)

Exceptions:

1. Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m^2) or less and roof live loads of 30 psf or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.
3. Where the effect of H resists the primary variable load effect, a load factor of 0.6 shall be included with H where H is permanent and H shall be set to zero for all other conditions.

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. The need for correlation is due to ASCE 7 Proposal LCSC LC-9a, which has been approved by the Load Combinations Subcommittee (LCSC) and is being balloted by the Main Committee (Item #2 of the Second Main Committee Ballot on Load Combinations). It is expected that the Main Committee will approve the proposal. This proposal is being submitted in conjunction with a similar proposal that revises IBC Sections 1605.2 and 1605.3 to closely match Sections 2.3 and 2.4 of ASCE 7-10 as revised by the ASCE 7 proposal. This proposal is equivalent to that proposal but relies more on the actual load combinations and less on text modifying the load combinations.

The changes will improve the equivalency of the Strength/LRFD and ASD load combinations with respect to the inclusion of loads due to fluids, *F*, and loads due to lateral earth pressures, ground water pressures or the pressure of bulk materials, *H.* The ASCE 7 proposal noted above was prompted, in part, by a proposal that added *F*, *H*, and loads due to self-straining forces, *T*, to the basic strength design load combinations of ASCE 7 necessary to achieve this equivalency. The removal of *T* from the Strength/LRFD and ASD load combinations is the subject of a separate proposal. The removal of earthquake load, *E*, from Equation 16-13 and the addition of Equation 16-14 has the effect of retaining roof live load, L_r, and rain load, *R*, in combination with wind load, *W* (Equation 16-13) but removing these loads in combination with earthquake load, *E* (Equation 16-14). This is being done to improve the equivalency of these load combinations with Strength/LRFD Equations 16-4 and 16-5. A similar deletion from the ASD load combinations in Section 2.4.1 of ASCE 7 is made by Proposal LCSC LC-9a but L_r and *R* are deleted in combination with *W* and *E*.

Correlation of LRFD and Basic ASD load combinations:

1. 1.4 (D+F)	8. D + F
2. 1.2 (D+F+T) + 1.6 (L+H) + 0.5S	9. D + H + F + L + T [not S]
2. 1.2 (D+F+T) + 1.6 (L+H) + 0.5S	10. D + H + F + S [not T, L]
2. 1.2 (D+F+T) + 1.6 (L+H) + 0.5S	11. D + H + F + 0.75L + 0.75T + 0.75S
3a. 1.2D + 1.6S + 1.0L [not H, F, T]	9. D + H + F + L + T [not S]
3a. 1.2D + 1.6S + 1.0L [not H, F]	10. D + H + F + S [not L]
3a. 1.2D + 1.6S + 1.0L [not H, F, T]	11. D + H + F + 0.75L + 0.75T + 0.75S
3b. 1.2D + 1.6S + 0.8W [not H, F]	12a. D + H + F + W [not S]
3b. 1.2D + 1.6S + 0.8W [not H, F, L]	13a. D + H + F + 0.75W + 0.75L + 0.75S
4. 1.2D + 1.6W + 1.0L + 0.5S [not H, F]	12a. D + H + F + W [not L, S]
4. 1.2D + 1.6W + 1.0L + 0.5S [not H, F]	13a. D + H + F + 0.75W + 0.75L + 0.75S
5. 1.2D + 1.0E + 1.0L + 0.2S [not H, F]	12b. D + H + F + 0.7E [not L, S]
5. 1.2D + 1.0E + 1.0L + 0.2S [not H, F]	13b. D + H + F + 0.75 (0.7E) + 0.75L + 0.75S
6. 0.9D + 1.6W + 1.6H	14. 0.6D + W + H
7. 0.9D + 1.0E + 1.6H	15. 0.6D + 0.7 E + H

Correlation of LRFD and Basic ASD load combinations with loads rearranged to facilitate review:

1. 1.4 (D+F)	8. D + F
2. 1.2 (D+F+T) + 1.6H + 1.6L + 0.5S	9. D + H + F + T + L [not S]
2. 1.2 (D+F+T) + 1.6H + 1.6L + 0.5S	10. D + H + F + S [not T, L]
2. 1.2 (D+F+T) + 1.6H + 1.6L + 0.5S	11. D + H + F + 0.75T + 0.75L + 0.75S
3a. 1.2D + 1.0L + 1.6S [not H, F, T]	9. D + H + F + T + L [not S]
3a. 1.2D + 1.0L + 1.6S [not H, F]	10. D + H + F + S [not L]
3a. 1.2D + 1.0L + 1.6S [not H, F, T]	11. D + H + F + 0.75T + 0.75L + 0.75S
3b. 1.2D + 1.6S + 0.8W [not H, F]	12a. D + H + F + W [not S]
3b. 1.2D + 1.6S + 0.8W [not H, F, L]	13a. D + H + F + 0.75L + 0.75S + 0.75W
4. 1.2D + 1.0L + 0.5S + 1.6W [not H, F]	12a. D + H + F + W [not L, S]
4. 1.2D + 1.0L + 0.5S + 1.6W [not H, F]	13a. D + H + F + 0.75L + 0.75S + 0.75W
5. 1.2D + 1.0L + 0.2S + 1.0E [not H, F]	12b. D + H + F + 0.7E [not L, S]
5. 1.2D + 1.0L + 0.2S + 1.0E [not H, F]	13b. D + H + F + 0.75L + 0.75S + 0.75 (0.7E)
6. 0.9D + 1.6H + 1.6W	14. 0.6D + H + W
7. 0.9D + 1.6H + 1.0E	15. 0.6D + H + 0.7 E

Correlation of LRFD and Basic ASD load combinations with proposed changes from proposal originally submitted by the proponent to the ASCE 7 Committee ("T" shown for reference only):

1. 1.4 (D+F)	8. D + F
2. 1.2 (D+F+T) + 1.6H + 1.6L + 0.5S	9. D + H + F + T + L [not S]
2. 1.2 (D+F+T) + 1.6H + 1.6L + 0.5S	10. D + H + F + S [not T, L]
2. 1.2 (D+F+T) + 1.6H + 1.6L + 0.5S	11. D + H + F + 0.75T + 0.75L + 0.75S
3a. 1.2 (D+F+T) + 1.0 (L+H) + 1.6S [not H, F, T]	9. D + H + F + T + L [not S]
3a. 1.2 (D+F+T) + 1.0 (L+H) + 1.6S [not H, F]	10. D + H + F + S [not T, L]
3a. 1.2 (D+F+T) + 1.0 (L+H) + 1.6S [not H, F, T]	11. D + H + F + 0.75T + 0.75L + 0.75S
3b. 1.2 (D+F+T) + 1.6S + 0.8W [not H, F]	12a. D + H + F + W [not T, S]
3b. 1.2 (D+F+T) + 1.6S + 0.8W [not H, F, T]	13a. D + H + F + 0.75L + 0.75S + 0.75W
4. 1.2 (D+F) + 1.0 (L+H) + 0.5S + 1.6W [not H, F]	12a. D + H + F + W [not L, S]
4. 1.2 (D+F) + 1.0 (L+H) + 0.5S + 1.6W [not H, F]	13a. D + H + F + 0.75L + 0.75S + 0.75W
5. 1.2 (D+F) + 1.0 (L+H) + 0.2S + 1.0E [not H, F]	12b. D + H + F + 0.7E [not L, S]
5. 1.2 (D+F) + 1.0 (L+H) + 0.2S + 1.0E [not H, F]	13b. D + H + F + 0.75L + 0.75S + 0.75 (0.7E)
6. 0.9D + 1.6H + 1.6W	14. 0.6D + H + W
7. 0.9D + 1.6H + 1.0E	15. 0.6D + H + 0.7 E

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

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

ICCFILENAME: Brazil-S30-1605.2.1

S50–09/10
1605.2.1, 1605.3.1, 1605.3.2

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

Revise as follows:

1605.2.1 Basic load combinations. Where strength design or load and resistance factor design is used, structures and portions thereof shall resist the most critical effects from the following combinations of factored loads:

$1.4 (D + F)$ **(Equation 16-1)**

$1.2 (D + F + T) + 1.6 (L + H)$
$+ 0.5 (L_r \text{ or } S \text{ or } R)$ **(Equation 16-2)**

$1.2D + 1.6 (L_r \text{ or } S \text{ or } R) + (f_1L \text{ or } 0.8W)$ **(Equation 16-3)**

$1.2D + 1.6W + f_1L + 0.5 (L_r \text{ or } S \text{ or } R)$ **(Equation 16-4)**

$1.2D + 1.0E + f_1L + \cancel{f_2}\ \underline{0.2}\ S$ **(Equation 16-5)**

$0.9D + 1.6W + 1.6H$ **(Equation 16-6)**

$0.9D + 1.0E + 1.6H$ **(Equation 16-7)**

where:

f_1 = 1 for floors in places of public assembly, for live loads in excess of 100 pounds per square foot (4.79 kN/m^2), and for parking garage live load, and
= 0.5 for other live loads.
~~f_2 = 0.7 for roof configurations (such as saw tooth) that do not shed snow off the structure, and~~
~~= 0.2 for other roof configurations.~~

~~**Exception:** Where other factored load combinations are specifically required by the provisions of this code, such combinations shall take precedence.~~

1605.3.1 Basic load combinations. Where allowable stress design (working stress design), as permitted by this code, is used, structures and portions thereof shall resist the most critical effects resulting from the following combinations of loads:

$D + F$ **(Equation 16-8)**

$D + H + F + L + T$ **(Equation 16-9)**

$D + H + F + (L_r \text{ or } S \text{ or } R)$ **(Equation 16-10)**

$D + H + F + 0.75 (L + T) +$
$0.75 (L_r \text{ or } S \text{ or } R)$ **(Equation 16-11)**

$D + H + F + (W \text{ or } 0.7E)$ **(Equation 16-12)**

$D + H + F + 0.75 (W \text{ or } 0.7E)$
$+ 0.75L + 0.75 (L_r \text{ or } S \text{ or } R)$ **(Equation 16-13)**

$0.6D + W + H$ **(Equation 16-14)**

$0.6D + 0.7E + H$ **(Equation 16-15)**

Exception~~s~~:

~~1.~~ Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.

~~2. Flat roof snow loads of 30 psf (1.44 kN/m^2) or less and roof live loads of 30 psf or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.~~

1605.3.2 Alternative basic load combinations. In lieu of the basic load combinations specified in Section 1605.3.1, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following combinations. When using these alternate basic load combinations that include wind or seismic loads, allowable stresses are permitted to be increased or load combinations reduced, where permitted by the material chapter of this code or the referenced standards. For load combinations that include the counteracting effects of dead and wind loads, only two-thirds of the minimum dead load likely to be in place during a design wind event shall be used. Where wind loads are calculated in accordance with Chapter 6 of ASCE 7, the coefficient ω in the following equations shall be taken as 1.3. For other wind loads, ω shall be taken as 1.0. When using these alternative load combinations to evaluate sliding, overturning and soil bearing at the soil-structure interface, the reduction of foundation overturning from Section 12.13.4 in ASCE 7 shall not be used. When using these alternative basic load combinations for proportioning foundations for loadings, which include seismic loads, the vertical seismic load effect, E_v, in Equation 12.4-4 of ASCE 7 is permitted to be taken equal to zero.

$D + L + (L_r \text{ or } S \text{ or } R)$ **(Equation 16-16)**

$D + L + (\omega W)$ **(Equation 16-17)**

$D + L + \omega W + S/2$ **(Equation 16-18)**

$D + L + S + \omega W/2$ **(Equation 16-19)**

$D + L + S + E/1.4$ **(Equation 16-20)**

$0.9D + E/1.4$ **(Equation 16-21)**

Exception~~s~~:

~~1.~~ Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.

~~2. Flat roof snow loads of 30 psf (1.44 kN/m^2) or less and roof live loads of 30 psf or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.~~

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. Most of the provisions being deleted were the product of a legacy code (UBC Section 1612.2.1 for roof configurations and Exception 1 to Section 1612.3.2 for snow and seismic loads). They have not appeared in recent editions of ASCE 7 and there are no proposals being considered by the committees of ASCE 7 to add them to the 2010 edition.

The exception to Section 1605.2.1 for factored load combinations in other sections of the IBC taking precedence over the Strength/LRFD load combinations in Section 1605.2.1 has been in the IBC since the 2003 edition. In that edition, there were other factored load combinations in Section 1605.4. They have since been removed from the IBC by Proposal S8-06/07-AMPC2,3 and there are no other factored load combinations in the IBC. Thus, the exception no longer serves a purpose and should be deleted.

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

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

ICCFILENAME: Brazil-S49-1605.2.1

S51–09/10
1605.2.1, 1605.3.1

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

Revise as follows:

1605.2.1 Basic load combinations. Where strength design or load and resistance factor design is used, structures and portions thereof shall resist the most critical effects from the following combinations of factored loads:

$1.4\,(D + F)$ **(Equation 16-1)**

$1.2\,(D + F + T) + 1.6\,(L + H) +$
$0.5\,(L_r \text{ or } S \text{ or } R)$ **(Equation 16-2)**

$1.2D + 1.6\,(L_r \text{ or } S \text{ or } R) + (f_1L \text{ or } \cancel{0.8}\ \underline{0.5}\ W)$ **(Equation 16-3)**

$1.2D + \cancel{1.6}\ \underline{1.0}\ W + f_1L + 0.5\,(L_r \text{ or } S \text{ or } R)$ **(Equation 16-4)**

$1.2D + 1.0E + f_1L + f_2S$ **(Equation 16-5)**

$0.9D + \cancel{1.6}\ \underline{1.0}\ W + 1.6H$ **(Equation 16-6)**

$0.9D + 1.0E + 1.6H$ **(Equation 16-7)**

where:

f_1 = 1 for floors in places of public assembly, for live loads in excess of 100 pounds per square foot (4.79 kN/m^2), and for parking garage live load, and
= 0.5 for other live loads.
f_2 = 0.7 for roof configurations (such as saw tooth) that do not shed snow off the structure, and
= 0.2 for other roof configurations.

Exception: Where other factored load combinations are specifically required by the provisions of this code, such combinations shall take precedence.

1605.3.1 Basic load combinations. Where allowable stress design (working stress design), as permitted by this code, is used, structures and portions thereof shall resist the most critical effects resulting from the following combinations of loads:

$D + F$ **(Equation 16-8)**

$D + H + F + L + T$ **(Equation 16-9)**

$D + H + F + (L_r \text{ or } S \text{ or } R)$ **(Equation 16-10)**

$D + H + F + 0.75\ (L + T) + 0.75\ (L_r \text{ or } S \text{ or } R)$ **(Equation 16-11)**

$D + H + F + (\underline{0.6}\ W \text{ or } 0.7E)$ **(Equation 16-12)**

$D + H + F + 0.75\ (\underline{0.6}\ W \text{ or } 0.7E) + 0.75L + 0.75\ (L_r \text{ or } S \text{ or } R)$ **(Equation 16-13)**

$0.6D + \underline{0.6}\ W + H$ **(Equation 16-14)**

$0.6D + 0.7E + H$ **(Equation 16-15)**

Exceptions:

1. Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m^2) or less and roof live loads of 30 psf or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. The need for correlation is due to ASCE 7 Proposal LCSC LC-10-R1, which has been approved by the Load Combinations Subcommittee and is being balloted by the Main Committee (Item #17 of the Second Main Committee Ballot on Load Combinations). It is expected that the Main Committee will approve the proposal.

In the load combinations of Section 1605.2.1 for strength/LRFD design, the load factors on wind load are being reduced because of the change in the specification of the design wind speed in Chapter 6 of ASCE 7-10. The wind speed in ASCE 7-10 is mapped at much longer return periods than in ASCE 7-05 (700-1700 years depending on occupancy category), which eliminates a discontinuity in risk between hurricane-prone coastal areas and the remainder of the country and better aligns the treatment of wind and earthquake effects.

Corresponding reductions are made to the basic load combinations for allowable stress design in Section 1605.3.1.

Note that this proposal does not contain similar changes to the alternative basic load combinations in Section 1605.3.2. The ASCE 7 proposal described above does not address these load combinations because they are not included in ASCE 7. Revisions to the alternative basic load combinations for compatibility with the revisions to the wind load provisions in ASCE 7-10 should be pursued by others. The deletion of the alternative basic load combinations is the subject of a separate proposal.

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

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

ICCFILENAME: Brazil-S52-1605.2.1

S52–09/10
1602.1, 1605.2.1, 1605.2.2, 1605.3.1, 1605.3.1.2, 1605.3.2.1

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

Revise as follows:

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

NOTATIONS.

D = Dead load.
E = Combined effect of horizontal and vertical earthquake induced forces as defined in Section 12.4.2 of ASCE 7.
F = Load due to fluids with well-defined pressures and maximum heights.
F_a = Flood load in accordance with Chapter 5 of ASCE 7.
H = Load due to lateral earth pressures, ground water pressure or pressure of bulk materials.
L = Live load, except roof live load, including any permitted live load reduction.
L_r = Roof live load including any permitted live load reduction.
R = Rain load.
S = Snow load.
T = Self-straining ~~force arising from contraction or expansion resulting from temperature change, shrinkage, moisture change, creep in component materials, movement due to differential settlement or combinations thereof~~ load.
W = Load due to wind pressure.

1605.2.1 Basic load combinations. Where strength design or load and resistance factor design is used, structures and portions thereof shall resist the most critical effects from the following combinations of factored loads:

$1.4 (D + F)$ **(Equation 16-1)**

$1.2 (D + F \cancel{+ T}) + 1.6 (L + H) +$
$0.5 (L_r \text{ or } S \text{ or } R)$ **(Equation 16-2)**

$1.2D + 1.6 (L_r \text{ or } S \text{ or } R) + (f_1L \text{ or } 0.8W)$ **(Equation 16-3)**

$1.2D + 1.6W + f_1L + 0.5 (L_r \text{ or } S \text{ or } R)$ **(Equation 16-4)**

$1.2D + 1.0E + f_1L + f_2S$ **(Equation 16-5)**

$0.9D + 1.6W + 1.6H$ **(Equation 16-6)**

$0.9D + 1.0E + 1.6H$ **(Equation 16-7)**

where:

f_1 = 1 for floors in places of public assembly, for live loads in excess of 100 pounds per square foot (4.79 kN/m^2), and for parking garage live load, and
= 0.5 for other live loads.
f_2 = 0.7 for roof configurations (such as saw tooth) that do not shed snow off the structure, and
= 0.2 for other roof configurations.

Exception: Where other factored load combinations are specifically required by the provisions of this code, such combinations shall take precedence.

1605.2.2 ~~Flood~~ Other loads. Where flood loads, F_a, are to be considered in the design, the load combinations of Section 2.3.3 of ASCE 7 shall be used. Where self-straining loads, T, are considered in design, their structural effects in combination with other loads shall be determined in accordance with Section 2.3.5 of ASCE 7.

1605.3.1 Basic load combinations. Where allowable stress design (working stress design), as permitted by this code, is used, structures and portions thereof shall resist the most critical effects resulting from the following combinations of loads:

$D + F$ **(Equation 16-8)**

$D + H + F + L \cancel{+ T}$ **(Equation 16-9)**

$D + H + F + (L_r \text{ or } S \text{ or } R)$ **(Equation 16-10)**

$D + H + F + 0.75 (L \cancel{+ T}) +$

0.75 (L_r or S or R) **(Equation 16-11)**

D + H + F + (W or 0.7E) **(Equation 16-12)**

D + H + F + 0.75 (W or 0.7E)
+ 0.75L + 0.75 (L_r or S or R) **(Equation 16-13)**

0.6D + W + H **(Equation 16-14)**

0.6D + 0.7E + H **(Equation 16-15)**

Exceptions:

1. Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m^2) or less and roof live loads of 30 psf or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.

1605.3.1.2 ~~Flood~~ Other loads. Where flood loads, F_a, are to be considered in design, the load combinations of Section 2.4.2 of ASCE 7 shall be used. Where self-straining loads, *T*, are considered in design, their structural effects in combination with other loads shall be determined in accordance with Section 2.4.4 of ASCE 7.

1605.3.2.1 Other loads. Where *F*, *H* or *T* are to be considered in design, each applicable load shall be added to the combinations specified in Section 1605.3.2. Where self-straining loads, *T*, are considered in design, their structural effects in combination with other loads shall be determined in accordance with Section 2.4.4 of ASCE 7.

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. The need for correlation is due to ASCE 7 Proposal LCSC LC-1-R1, which has been approved by the Load Combinations Subcommittee and is being balloted by the Main Committee (Item #4 of the Second Main Committee Ballot on Load Combinations). It is expected that the Main Committee will approve the proposal.

Self-straining forces are important to the design of many types of structures, for example, the pretensioning and post-tensioning forces in prestressed concrete structures. The self-straining load, *T*, has been in the load combinations of IBC Section 1605 since the 2003 edition, all editions of ASCE 7, and ANSI A58.1-72. However, a consensus emerged within the ASCE 7 Committee to remove *T* from the load combinations of Section 2.3 for strength/LRFD design and Section 2.4 for allowable stress design in favor of text in new Sections 2.3.5 and 2.4.4 requiring consideration of *T* and additional commentary. This is similar to what had been in ASCE 7 prior to the 1995 edition. For example, the load combinations in Sections 2.3 and 2.4 of ASCE 7-88 and ASCE 7-93 did not include *T* but Section 2.3.2 for allowable stress design and Section 2.4.3 for strength design stated that the structural effects of *T* (as well as *F*, *H* and *P*) "shall be considered in design."

Note that the statements being added to Sections 1605.2.2, 1605.3.1.2 and 1605.3.2.1 for self-straining loads, *T*, do not constitute charging text. They are cross references to Section 2.3.5 of ASCE 7-10 for Strength/LRFD load combinations and Section 2.4.4 of ASCE 7-10 for ASD load combinations, which provide the necessary charging text.

The statements in Sections 1605.2.2 and 1605.3.1.2 on flood loads, F_a, also do not constitute charging text but Section 1612.1 serves as the charging text for flood loads in the IBC in that it requires buildings, structures and portions thereof to be designed and constructed to resist the effects of flood loads in flood hazard areas. The revisions in this proposal may appear to eliminate charging text for *T* but the 2009 IBC contains no such charging text except indirectly through the definition of *T* in Section 1602.1.

Although Section 1605.3.2.1 on other loads for use with the alternative basic load combinations is being modified in this proposal, the deletion of these load combinations is the subject of a separate proposal. Should the deletion of the alternative basic load combinations be approved by the ICC membership, it is not the intent of the proponent to retain Section 1605.3.2.1 in the 2012 IBC for the purpose of adding the statement in this proposal.

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

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

ICCFILENAME: Brazil-S42-1602.1

S53–09/10
1605.3.1

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

Revise as follows:

1605.3.1 Basic load combinations. Where allowable stress design (working stress design), as permitted by this code, is used, structures and portions thereof shall resist the most critical effects resulting from the following combinations of loads:

$D + F$ **(Equation 16-8)**

$D + H + F + L + T$ **(Equation 16-9)**

$D + H + F + (L_r \text{ or } S \text{ or } R)$ **(Equation 16-10)**

$D + H + F + 0.75\ (L + T) +$
$0.75\ (L_r \text{ or } S \text{ or } R)$ **(Equation 16-11)**

$D + H + F + (W \text{ or } 0.7\ E)$ **(Equation 16-12)**

$D + H + F + 0.75\ (W \text{ or } 0.7\ E) +$
$0.75\ L + 0.75\ (L_r \text{ or } S \text{ or } R)$ **(Equation 16-13)**

$0.6\ D + W + H$ **(Equation 16-14)**

$0.6\ D + 0.7\ E + H$ **(Equation 16-15)**

Exceptions:

1. Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m^2) or less and roof live loads of 30 psf or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.
3. In Equation 16-14, the wind load, *W*, is permitted to be reduced 10 percent for design of the foundation other than anchorage of the structure to the foundation.
4. In Equation 16-15, 0.6 *D* is permitted to be increased to 0.9 *D* for the design of special reinforced masonry shear walls complying with Chapter 21.

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. The need for correlation is due to ASCE 7 Proposal LCSC LC-4, which has been approved by the Load Combinations Subcommittee (LCSC) and is being balloted by the Main Committee (Item #3 of the Second Main Committee Ballot on Load Combinations). It is expected that the Main Committee will approve the proposal.
The current ASD counteracting load combinations in Equations 16-14 and 16-15 are needed to maintain consistency with the Strength/LRFD counteracting load combinations in Equations 16-6 and 16-7 (Section 1605.2.1). Predictability of dead load cannot justify increasing the ASD counteracting dead load factor from 0.6 because of the large variability in the destabilizing force due to wind or earthquake forces. However, where the effects of fluctuating wind forces are aggregated at the foundation, a measure of conservatism is introduced with a dead load factor of 0.6 due to area-averaging, which justifies an increase. Rather than increase the dead load factor in ASD Load Combination 7 of ASCE-7 (IBC Equation 16-14), the LCSC concluded it was more rational to reduce the wind effects (Exception #3) for evaluating global stability, bearing and uplift at the structure-foundation interface. Similar approaches are taken for seismic actions in Exception #2 of Section 12.4.2.2 and in Section 12.13.4 of ASCE 7-05.

Exception #4 for special reinforced masonry shear walls is being added due to the determination of seismic load effects reduced by the response modification coefficient, R, which means that inelastic action occurs before reaching the limit state. In special reinforced masonry shear walls designed in accordance with Chapter 21, there is a minimum quantity or vertical reinforcement, a conservative value of allowable stress is assumed in proportioning the reinforcement, and there is protection against premature crushing at the compression face. Given these, the limit state of unrestrained overturning is not judged to be a concern, justifying the increase from 0.6 to 0.9 in the load factor for D in Equation 16-15.

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

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

ICCFILENAME: Brazil-S19-1605.3.1

S54–09/10
1605.1, 1605.3.1, 1605.3.2, 1605.3.2.1, 1806.1

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

1. Revise as follows:

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

1. The load combinations specified in Section 1605.2~~, 1605.3.1 or 1605.3.2~~ or 1605.3,
2. The load combinations specified in Chapters 18 through 23, and

3. The load combinations with overstrength factor specified in Section 12.4.3.2 of ASCE 7 where required by Section 12.2.5.2, 12.3.3.3 or 12.10.2.1 of ASCE 7. With the simplified procedure of ASCE 7 Section 12.14, the load combinations with overstrength factor of Section 12.14.3.2 of ASCE 7 shall be used.

Applicable loads shall be considered, including both earthquake and wind, in accordance with the specified load combinations. Each load combination shall also be investigated with one or more of the variable loads set to zero.

Where the load combinations with overstrength factor in Section 12.4.3.2 of ASCE 7 apply, they shall be used as follows:

1. The basic combinations for strength design with overstrength factor in lieu of Equations 16-5 and 16-7 in Section 1605.2.1.
2. The basic combinations for allowable stress design with overstrength factor in lieu of Equations 16-12, 16-13 and 16-15 in Section ~~1605.3.1~~ 1605.3.
3. ~~The basic combinations for allowable stress design with overstrength factor in lieu of Equations 16-20 and 16-21 in Section 1605.3.2.~~

1605.3 Load combinations using allowable stress design.

~~**1605.3.1 Basic load combinations.**~~ Where allowable stress design ~~(working stress design)~~, as permitted by this code, is used, buildings and other structures, and portions thereof, shall be designed to resist the most critical effects resulting from the following combinations of loads:

$D+F$ **(Equation 16-8)**

$D+H+F+L+T$ **(Equation 16-9)**

$D+H+F+(L_r \text{ or } S \text{ or } R)$ **(Equation 16-10)**

$D+H+F+0.75(L+T)+0.75(L_r \text{ or } S \text{ or } R)$ **(Equation 16-11)**

$D+H+F+(W \text{ or } 0.7E)$ **(Equation 16-12)**

$D+H+F+0.75(W \text{ or } 0.7E)+0.75L+0.75(L_r \text{ or } S \text{ or } R)$ **(Equation 16-13)**

$0.6D+W+H$ **(Equation 16-14)**

$0.6D+0.7E+H$ **(Equation 16-15)**

Exceptions:

1. Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m^2) or less and roof live loads of 30 psf or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.

2. Delete without substitution:

~~**1605.3.2 Alternative basic load combinations.** In lieu of the basic load combinations specified in Section 1605.3.1, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following combinations. When using these alternate basic load combinations that include wind or seismic loads, allowable stresses are permitted to be increased or load combinations reduced, where permitted by the material chapter of this code or the referenced standards. For load combinations that include the counteracting effects of dead and wind loads, only two-thirds of the minimum dead load likely to be in place during a design wind event shall be used. Where wind loads are calculated in accordance with Chapter 6 of ASCE 7, the coefficient ω in the following equations shall be taken as 1.3. For other wind loads, ω shall be taken as 1.0. When using these alternative load combinations to evaluate sliding, overturning and soil bearing at the soil-structure interface, the reduction of foundation overturning from Section 12.13.4 in ASCE 7 shall not be used. When using these alternative basic load combinations~~

~~for proportioning foundations for loadings, which include seismic loads, the vertical seismic load effect, E_v, in Equation 12.4-4 of ASCE 7 is permitted to be taken equal to zero.~~

~~$D + L + (L_r$ or S or $R)$~~ **~~(Equation 16-16)~~**

~~$D + L + (\omega W)$~~ **~~(Equation 16-17)~~**

~~$D + L + \omega W + S/2$~~ **~~(Equation 16-18)~~**

~~$D + L + S + \omega W/2$~~ **~~(Equation 16-19)~~**

~~$D + L + S + E/1.4$~~ **~~(Equation 16-20)~~**

~~$0.9D + E/1.4$~~ **~~(Equation 16-21)~~**

~~Exceptions:~~

1. ~~Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.~~
2. ~~Flat roof snow loads of 30 psf (1.44 kN/m^2) or less and roof live loads of 30 psf or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.~~

~~1605.3.2.1 Other loads.~~ ~~Where *F*, *H* or *T* are to be considered in design, each applicable load shall be added to the combinations specified in Section 1605.3.2.~~

3. Revise as follows:

1806.1 Load combinations. The presumptive load-bearing values provided in Table 1806.2 shall be used with the allowable stress design load combinations specified in Section 1605.3. ~~The values of vertical foundation pressure and lateral bearing pressure given in Table 1806.2 shall be permitted to be increased by one-third where used with the alternative basic load combinations of Section 1605.3.2 that include wind or earthquake loads.~~

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. Most of the provisions being deleted were the product of a legacy code. They have not appeared in recent editions of ASCE 7 nor are they likely to appear in the 2010 edition.

The provisions of IBC Sections 1605.1 through 1605.3.1.2 for the load combinations using strength/LRDF design and the basic load combinations using allowable stress design are based on Chapter 2 of ASCE 7 (Combinations of Loads). The ASCE 7 Committee develops and maintains the provisions of Chapter 2 and has not chosen not to add provisions for alternative load combinations using allowable stress design into the standard. The differences between the basic and alternative load combinations are such that structural performance of buildings and other structures can be expected to differ depending on which set of load combinations using allowable stress design are utilized in the design. The ICC code development process does not provide the means for adequate deliberation over the merits of each set of load combinations using allowable stress design in providing adequate levels of public safety. This deliberation should be done by the ASCE 7 Committee.

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

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

ICCFILENAME: Brazil-S24-1605.1

S55–09/10

106.1, 1607.1.1

Proponent: Larry Brown, representing National Association of Home Builders

Delete Section 106, Relocate 106.1 to become 1607.1.1 as follows:

<u>1607.1.1</u> ~~106.1~~ Live loads posted. Where the live loads for which each floor or portion thereof of a commercial or industrial building is or has been designed to exceed 50 psf (2.40 kN/m^2), such design live loads shall be conspicuously posted by the owner in that part of each *story* in which they apply, using durable signs. It shall be unlawful to remove or deface such notices

~~106.2 Issuance of certificate of occupancy.~~ ~~A certificate of occupancy required by Section 111 shall not be issued until the floor load signs, required by Section 106.1, have been installed.~~

(Moved to Section 109.2.1 in CCP ADM1-09/10)

~~**106.3 Restriction on loading.** It shall be unlawful to place, or cause or permit to be placed, on any floor or roof of a building, structure or portion thereof, a load greater than is permitted by this code.~~
(Moved to Section 102.11.2 in CCP ADM1-09/10)

Reason: The requirements for the actual determination and posting of the live load is better suited to be located with the actual provisions related to these live loads that are located in IBC Section 1607.1.1. This relocation of Section 106.41is similar to the provisions for Smoke Control Systems in Section 909.19, and a Change of Occupancy in Section 3408.2, both relative to the issuance of a Certificate of Occupancy.

Also, as show in a related proposed change, the requirement on restriction on loading of current Section 106.3 are moved to Section 102.11.2 (Specific application of the IBC), and the provisions on the issuance of a Certificate of Occupancy is located from Section 106.2 to Section 109.2 (Certificate of Completion and Occupancy).

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

Note: Consideration in this code change proposal is for relocation of Section 106.1 to Section 1607.1.1. The relocation of Section s 106.2 to 109.2.1 and 106.3 to 102.11.2 are proposed for consideration by the Administrative Code Development Committee in CCC ADM1-09/10, and are shown for information only.

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

ICCFILENAME: BROWN-S1-ITEM_#3_IBC_106_MOVED

S56-09/10
1603.1.3, Table 1607.1

Proponent: Philip Brazil, PE, SE, representing self

Revise as follows:

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
16. Garages (passenger vehicles only)	40	Note a
Trucks and buses	See Section	1607.6
<u>Piled snow (from snow removal operations)</u>	<u>Note m</u>	

(No changes to the remaining Table)

(No change to footnotes a through l)

<u>m.</u> <u>Piled snow from snow removal operations shall be based on a density of 40 pounds per cubic foot and anticipated maximum depths subject to the approval of the building official.</u>

1603.1.3 Roof snow load. The ground snow load, P_g, shall be indicated. In areas where the ground snow load, P_g, exceeds 10 pounds per square foot (psf) (0.479 kN/m^2), the following additional information shall also be provided, regardless of whether snow loads govern the design of the roof:

1. Flat-roof snow load, P_f.
2. Snow exposure factor, C_e.
3. Snow load importance factor, I.
4. Thermal factor, C_t.
5. <u>Piled snow load from snow removal operations (see Table 1607.1).</u>

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. The need for correlation is due to ASCE 7 Proposal LLSC-LL-4, which has been approved by the Live Load Subcommittee. It is expected that the Main Committee will ballot and approve the proposal. The specified density is based on case histories primarily conducted in the northeast region of the United States. The snow was typically piled using a utility vehicle or a truck with an attached plow. The density was either directly determined through measurements or derived based on the expected failure load. The observed densities ranged form 25 to 46 pcf.

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

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

FILENAME: Brazil-S2-T1607.1

S57-09/10
Table 1607.1; IRC Table R301.5

Proponent: Philip Brazil, PE, SE, representing self

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC RESIDENTIAL COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES

Part I—IBC Structural:

Revise as follows:

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
27. Residential		
One- and two-family dwellings		
Uninhabitable attics without storage [i]	10	
Uninhabitable attics with ~~limited~~ storage [i, j, k]	20	
Habitable attics and sleeping areas [k]	30	–
All other areas	40	
Hotels and multiple-family dwellings		
Private rooms and corridors serving them	40	
Public rooms and corridors serving them	100	

(No changes to the remaining Table not shown)

(No change to footnotes a through h)

I. Uninhabitable attics without storage are those where the maximum clear height between the joist and rafter is less than 42 inches, or where there are not two or more adjacent trusses with ~~the same~~ web configurations capable of ~~containing a~~ accommodating an assumed rectangle 42 inches ~~high~~ in height by ~~2 feet wide~~ 24 inches in width, or greater, ~~located~~ within the plane of the trusses. ~~For attics without storage~~, This live load need not be assumed to act concurrently with any other live load requirements.

j. ~~For~~ Uninhabitable attics with ~~limited~~ storage ~~and constructed with trusses, this live load need only be applied to those portions of the bottom chord~~ are those where the maximum clear height between the joist and rafter is 42 inches or greater, or where there are two or more adjacent trusses with ~~the same~~ web configurations ~~containing a~~ capable of accommodating an assumed rectangle 42 inches ~~high~~ in height by ~~2 feet wide~~ 24 inches in width, or greater, ~~located~~ within the plane of the trusses. ~~The rectangle shall fit between the top of the bottom chord and the bottom of any other truss member, provided that each of the following criteria is met~~

At the trusses, the live load need only be applied to those portions of the bottom chords where both of the following conditions are met:

i. The attic area is accessible ~~by a pull-down stairway or framed opening in accordance with Section 1209.2~~ from an opening not less than 20 inches in width by 30 inches in length that is located where the clear height in the attic is a minimum of 30 inches; and

ii. The slopes of the truss ~~shall have a~~ bottom chords ~~pitch less than 2:12~~ are no greater than 2 units vertical to 12 units horizontal.

~~iii.~~ The remaining portions of the bottom chords ~~of trusses~~ shall be designed for ~~the greater of actual imposed dead load or 10 psf,~~ a uniformly distributed ~~over the entire span~~ concurrent live load of not less than 10 lb/ft^2.

k. Attic spaces served by ~~a fixed stair~~ stairways other than pull-down type shall be designed to support the minimum live load specified for habitable attics and sleeping rooms.

(No change to footnote l)

Part II: IRC

Revise as follows:

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

USE	LIVE LOAD
Uninhabitable attics without storage [b]	10
Uninhabitable attics with ~~limited~~ storage [b, g]	20
Habitable attics and attics served with fixed stairs	30

(No changes to the remaining Table not shown)

(No change to footnote a)

b. Uninhabitable attics without storage are those where the maximum clear height between the joist and rafter is less than 42 inches, or where there are not two or more adjacent trusses with ~~the same~~ web configurations capable of ~~containing a~~ accommodating an assumed rectangle 42 inches ~~high~~ in height by ~~2 feet wide~~ 24 inches in width, or greater, ~~located~~ within the plane of the trusses. ~~For attics without storage,~~ This live load need not be assumed to act concurrently with any other live load requirements.

(No change to footnotes c through f)

g. ~~For~~ Uninhabitable attics with ~~limited~~ storage ~~and constructed with trusses, this live load need be applied only to those portions of the bottom chord~~ are those where the maximum clear height between the joist and rafter is 42 inches or greater, or where there are two or more adjacent trusses with ~~the same~~ web configurations ~~containing a~~ capable of accommodating an assumed rectangle 42 inches ~~high or greater~~ in height by ~~2 feet wide~~ 24 inches in width, or greater, ~~located~~ within the plane of the trusses. ~~The rectangle shall fit between the top of the bottom chord and the bottom of any other truss member, provided that each of the following criteria is met~~

At the trusses, the live load need only be applied to those portions of the bottom chords where all of the following conditions are met:

1. The attic area is accessible ~~by a pull-down stairway or framed opening in accordance with Section R807.1~~ from an opening not less than 20 inches in width by 30 inches in length that is located where the clear height in the attic is a minimum of 30 inches.
2. The slopes of the truss ~~has a~~ bottom chords ~~pitch less than 2:12~~ are no greater than 2 units vertical to 12 units horizontal.
3. Required insulation depth is less than the bottom chord member depth.

The remaining portions of the bottom chords ~~of trusses meeting the above criteria for limited storage~~ shall be designed for ~~the greater of the actual imposed dead load or 10 psf,~~ a uniformly distributed ~~over the entire span~~ concurrent live load of not less than 10 lb/ft^2.

(No change to footnote h)

Reason: The purpose for this proposal is to correlate the IBC and IRC with the 2010 edition of ASCE 7. The need for correlation is due to ASCE 7 Proposal LLSC-LL-9, which has been approved by the Live Load Subcommittee and is being balloted by the Main Committee (Item #5 of the Second Main Committee Ballot on Live/Dead Load Provisions). It is expected that the Main Committee will approve the proposal.

The changes are seen as largely editorial. In Footnotes (i) and (j), the threshold that is based on a 24-inch by 42-inch rectangular is changed to an assumed condition (rather than an actual one), which is considered more appropriate for a building code requirement.

In Footnote (j), the reference to "a pull-down stairway or framed opening in accordance with Section 1209.2" is replaced with minimum opening dimensions that are consistent with IBC Section 1209.2 on openings to attic areas. These dimensions are objective and considered more appropriate for a building code requirement, whereas "pull-down stairway" and "framed opening" are considered vague and subject to a wide variation in interpretation.

In Footnote (k), the reference to a "fixed stair" is changed to "stairways other than pull-down type" in conjunction with the deletion of "pull-down type stairway" in Footnote (j) and for consistency with the definitions of "stair" and "stairway" in Section 1002.1. These definitions apply to all instances of the terms throughout the IBC. "Stair" is a "change in elevation consisting of one or more risers." "Stairway" is "one or more flights of stairs…with the necessary landings and platforms connecting them to form a continuous and uninterrupted passage form one level to another" and is the better choice for the footnote. The change will revise the footnote to better convey its intent: require an otherwise uninhabitable attic to be designed for live loads specified for habitable attics where the attic is served by a stairway that could enable it become occupiable. The current threshold of "fixed stair" before design for live loads specified for habitable attics is required is considered vague and subject to a wide variation in interpretation.

In Table 1607.1 and Footnote (j), "limited" at uninhabitable attics with storage is considered superfluous and is deleted. The three categories of uninhabitable attics without storage, uninhabitable attics with storage and habitable attics are sufficiently clear to account for all design conditions. Retaining "limited" begs the question: what is an uninhabitable attic with more than limited storage?

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

PART I- IBC STRUCTURAL

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

PART II- IRC BUILDING/ENERGY

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

FILENAME: Brazil-S3-T1607.1

S58–09/10
Table 1607.1

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

Revise as follows:

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
4. Assembly areas and theaters		
Fixed seats (fastened to floor)	60	
~~Follow spot, projections and control rooms~~	~~50~~	
Lobbies	100	–
Movable seats	100	
Stages ~~and~~	~~125~~ 150	
Platforms	100	
Other assembly areas	100	

(Portions of Table not shown, remain unchanged)

Reason The primary purpose for this proposal is to align IBC Table 1607.1 with Table 4-1 of ASCE 7, which specifies separate live loads of 150 psf for stages and 100 psf for platforms in assembly areas. Follow spot, projection and control rooms are being deleted from Table 1607.1 because they are not listed in Table 4-1 of ASCE 7 and a uniform live load of 60 psf for projection rooms conflicts with Table C4-1 of ASCE-7, which specifies a uniform live load of 100 psf. With these changes, the items under "assembly areas" in Table 1607.1 will be within the scope of areas within assembly occupancies where live loads due to assembly use are warranted. There are areas of buildings with assembly occupancies, such as control rooms and dressing rooms, where classification as assembly occupancies are not warranted. Such areas are not listed with "assembly areas" in Table 4-1 of ASCE 7 and will no longer be listed with "assembly areas" in IBC Table 1607.1.

A separate proposal correlates the IBC with changes to the provisions of ASCE 7-10 where reduction of live loads at floors and occupied roofs is restricted or prohibited. That proposal adds Footnote (m) to all the uniform live loads under assembly areas in Item #4 of Table 1607.1 except follow spot, projection and control rooms. The addition of the footnote is not repeated in this proposal but, should this proposal and the proposal adding the footnote be approved by the membership, the proponent intends that Footnote (m) be specified with each uniform live load at Item #4 in Table 1607.1.

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

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

ICCFILENAME: Brazil-S16-T1607.1

S59–09/10
Table 1607.1

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

Revise as follows:

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
35. ~~Stairs and exits~~ Stairways and ramps		Note f
One- and two-family dwellings	40	
All other	100	
39. Walkways and elevated platforms (other than ~~exitways~~ means of egress)	60	–

(Portions of Table not shown, remain unchanged)

Reason: The purpose for this proposal is to more clearly identify the live loads required for components of the means of egress, including corridors, stairways and ramps. In Item 35, "stair" does not account for landings to or from, or intermediate landings between, flights of stairs. In Section 1002.1, "stair" is defined as a "change in elevation consisting of one or more risers" and "stairway" is defined as "one or more flights of stairs…with the necessary landings and platforms connecting them to form a continuous and uninterrupted passage form one level to another." Stairway is a more technically sound choice.

Also in Item 35, “exit” is one of three components of the means of egress along with the exit access and exit discharge. The distinction between them is related to the requirements of Chapter 10 for the means of egress, not live loads. In Item 39, “exitway” has no technical meaning and is too easily confused with corridors whose live loads are typically required to be the same as the occupancy served.

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

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

ICCFILENAME: Brazil-S44-T1607.1

S60–09/10

1605.2.1, Table 1607.1, 1607.9.1, 1607.9.1.4, 1607.9.2, 1607.11.2.2

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

1. Revise as follows:

1605.2.1 Basic load combinations. Where strength design or load and resistance factor design is used, structures and portions thereof shall resist the most critical effects from the following combinations of factored loads:

$1.4 (D + F)$	**(Equation 16-1)**
$1.2 (D + F + T) + 1.6 (L + H) + 0.5 (L_r \text{ or } S \text{ or } R)$	**(Equation 16-2)**
$1.2 D + 1.6 (L_r \text{ or } S \text{ or } R) + (f_1 L \text{ or } 0.8 W)$	**(Equation 16-3)**
$1.2 D + 1.6 W + f_1 L + 0.5 (L_r \text{ or } S \text{ or } R)$	**(Equation 16-4)**
$1.2 D + 1.0 E + f_1 L + f_2 S$	**(Equation 16-5)**
$0.9 D + 1.6 W + 1.6 H$	**(Equation 16-6)**
$0.9 D + 1.0 E + 1.6 H$	**(Equation 16-7)**

where:

f_1 = 1 for floors in ~~places of public~~ assembly areas and recreational uses (see Table 1607.1), for live loads, L, in excess of 100 pounds per square foot (4.79 kN/m^2), and for ~~parking~~ floors in passenger vehicle garages ~~live load~~; and

= 0.5 for other live loads, L.

f_2 = 0.7 for roof configurations (such as saw tooth) that do not shed snow off the structure; and

= 0.2 for other roof configurations.

Exception: Where other factored load combinations are specifically required by the provisions of this code, such combinations shall take precedence.

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
3. Armories and drill rooms	150 [m]	–
4. Assembly areas and theaters Fixed seats (fastened to floor) Follow spot, projections and control rooms Lobbies Movable seats Stages and platforms Other assembly areas	 60 [m] 50 100 [m] 100 [m] 125 [m] 100 [m]	–
~~6. Bowling alleys~~	~~75~~	~~–~~
~~10. Dance halls and ballrooms~~	~~100~~	~~–~~
~~11~~ 9. Dining rooms and restaurants	100 [m]	–
~~16~~ 14. Garages (passenger vehicles only) Trucks and buses	40 [m] See	Note a Section 1607.6
~~17. Grandstands (see stadium and arena bleachers)~~	~~–~~	~~–~~
~~18. Gymnasiums, main floors and balconies~~	~~100~~	~~–~~
~~22~~ 18. Libraries Corridors above first floor	 80	 1,000

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
Reading rooms Stack rooms	60 150 [b, m]	1,000 1,000
~~23~~ 19. Manufacturing Heavy Light	 250 [m] 125 [m]	 3,000 2,000
23. Recreational uses: Bowling alleys, poolrooms and similar uses Dance halls and ballrooms Gymnasiums Reviewing stands, grandstands and bleachers Stadiums and arenas with fixed seats (fastened to floor)	 75 [m] 100 [m] 100 [m] 100 [c, m] 60 [c, m]	
~~28. Reviewing stands, grandstands and bleachers~~		~~Note c~~
~~29~~ 25. Roofs: All roof surfaces subject to maintenance workers Awnings and canopies: Fabric construction supported by a lightweight rigid skeleton structure All other construction Ordinary flat, pitched, and curved roofs Primary roof members, exposed to a work floor: Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages All other occupancies Roofs used for other special purposes Roofs used for promenade purposes Roofs used for roof gardens ~~or~~ Roofs used for assembly purposes	 5 nonreducible 20 20 Note l 60 100 100[m]	 300 2,000 300 Note l
~~32~~ 28. Sidewalks, vehicular driveways and yards, subject to trucking	250 [d, m]	8,000 [e]
~~33. Skating rinks~~	~~100~~	–
~~34. Stadiums and arenas~~ ~~Bleachers~~ ~~Fixed seats (fastened to floor)~~	 ~~100 [c]~~ ~~60 [c]~~	~~–~~
~~36~~ 30. Storage warehouses (shall be designed for heavier loads if required for anticipated storage) Heavy Light	 250 [m] 125 [m]	 –
~~37~~ 32. Stores Retail First floor Upper floors Wholesale, all floors	 100 75 125 [m]	 1,000 1,000 1,000
~~40~~ 35. Yards and terraces, pedestrian	100 [m]	–

(Portions of table not shown are unchanged)

c. Design in accordance with the ICC 300.
m. Live load reduction is not permitted unless specific exceptions of Section 1607.9 apply.

(Footnotes not shown are unchanged)

1607.9.1 General. Subject to the limitations of Sections 1607.9.1.1 through ~~1607.9.1.4~~ 1607.9.1.3 and Table 1607.1, members for which a value of $K_{LL} A_T$ is 400 square feet (37.16 m^2) or more are permitted to be designed for a reduced live load in accordance with the following equation:

$$L = L_o \left[0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right]$$ **(Equation 16-22)**

$$\text{In SI: } L = L_o \left[0.25 + \frac{4.57}{\sqrt{K_{LL} A_T}}\right]$$

where:

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

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

2. Delete without substitution:

~~**1607.9.1.4 Group A occupancies.** Live loads of 100 psf (4.79 kN/m²) and at areas where fixed seats are located shall not be reduced in Group A occupancies.~~

3. Revise as follows:

1607.9.2 Alternate floor live load reduction. As an alternative to Section 1607.9.1 <u>and subject to the limitations of Table 1607.1</u>, floor live loads are permitted to be reduced in accordance with the following provisions. Such reductions shall apply to slab systems, beams, girders, columns, piers, walls and foundations.

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

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

~~3~~ <u>2</u>. A reduction shall not be permitted in passenger vehicle parking garages except that the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent.
~~4~~ <u>3</u>. For live loads not exceeding 100 psf (4.79 kN/m^2), the design live load for any structural member supporting 150 square feet (13.94 m^2) or more is permitted to be reduced in accordance with Equation 16-23.
~~5~~ <u>4</u>. For one-way slabs, the area, A, for use in Equation 16-23 shall not exceed the product of the slab span and a width normal to the span of 0.5 times the slab span.

R = *0.08* (A – 150) **(Equation 16-23)**
For SI: R = *0.861* (A – 13.94)

Such reduction shall not exceed the smallest of:

1. 40 percent for horizontal members,
2. 60 percent for vertical members, or
3. R as determined by the following equation:

R = 23.1 (1 + D / L_o) **(Equation 16-24)**

where:

A = Area of floor supported by the member, square feet (m^2).
D = Dead load per square foot (m^2) of area supported.
L_o = Unreduced live load per square foot (m^2) of area supported.
R = Reduction in percent.

1607.11.2.2 Special-purpose roofs. Roofs used for promenade purposes, roof gardens, assembly purposes or other special purposes, and marquees, shall be designed for a minimum live load, L_o, as specified in Table 1607.1. Such live loads are permitted to be reduced in accordance with Section 1607.9. ~~Live loads of 100 psf or more at areas of roofs classified as Group A occupancies shall not be reduced.~~

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. The need for correlation is due to ASCE 7 Proposals LLSC-LL9 and LLSC-LL11, which were approved by the Live Load Subcommittee and are being balloted by the Main Committee (Items #5 and #6 of the Second Main Committee Ballot on Live/Dead Load Provisions). It is expected that the Main Committee will approve the proposals.

The proposal focuses on correlating the IBC with changes to the provisions of ASCE 7-10 where reduction of live loads at floors and occupied roofs is restricted or prohibited. The applicable provisions in the IBC are currently located in Section 1607.9. Reduction of live loads is typically permitted except for live loads exceeding 100 psf, in passenger vehicle garages, and in Group A occupancies where the live load is 100 psf or where fixed seats are located. There are exceptions for members supporting two or more floors where the live load exceeds 100 psf or in passenger vehicle garages but the reduction is limited to 20 percent. The corresponding provisions in ASCE 7-05 are nearly identical except that Group A occupancies are identified as assembly occupancies.

The proposal adds a footnote to Table 1607.1 that prohibits live load reduction “unless specific exceptions of Section 1607.9 apply.” The footnote is specified at each use or occupancy in Table 1607.1 where live load reduction is to be restricted. With the addition of this footnote, Table 1607.1 will contain limitations on live load reduction and references to Table 1607.1 are added to Sections 1607.9.1 and 1607.9.2 to correlate with the footnote. Section 1607.9.1.4 (basic live load reduction), Item #1 of Section 1607.9.2 (alternative live load reduction), and the last sentence of Section 1607.11.2.2, on Group A occupancies are deleted because their purpose is supplanted by the changes to Table 1607.1. Sections 1607.9.1.2 and 1607.9.1.3 (basic live load reduction) and Items #2 and #3 of Section 1607.9.2 (alternative live load reduction) are retained because they specify exceptions to Section 1607.9 that the proposed footnote of Table 1607.1 references.

These changes will clarify where live load reduction is prohibited or restricted by effectively specifying the requirement at each applicable use or occupancy in Table 1607.1 and they will align the applicable provisions of IBC Section 1607 with the corresponding provisions in Chapter 4 of ASCE 7-10. The change will also eliminate reliance on occupancy classification (Group A), which is not related to structural design but to fire- and life-safety regulations, for determination of whether live load reduction is permitted.

The proposal also consolidates several separately listed items in Table 1607.1 into a single category of recreational use and will align the table with Table 4-1 of ASCE 7-10. This is seen as simplifying the data in the table by grouping similar uses together. With respect to this consolidation, Section 1607.9.1.4 (basic live load reduction) and Item #1 of Section 1607.9.2 (alternative live load reduction) currently prohibit live load reduction in areas of Group A occupancies as noted above. IBC Section 303.1 lists bowling alleys, dance halls, gymnasiums, and pool and billiard parlors as Group A-3 occupancies; arenas and skating rinks as Group A-4 occupancies; and bleachers, grandstands and stadiums as Group A-5 occupancies.

Skating rinks are deleted from Table 1607.1 rather than being an item under “recreational uses” in Table 1607.1 because it is not listed in Table 4-1 of ASCE 7 and it conflicts with Table C4-1 of ASCE-7, which specifies uniform live loads of 250 psf for ice skating rinks and 100 psf for roller skating rinks.

The application of a value of 1.0 for f_1 in Section 1605.2.1 is revised for consistency with the other changes in this proposal. The notation for “L” is added to make it clear that roof live load, L_r, is not intended.

This proposal was prepared in conjunction with a proposal to editorially correlate IBC Section 1607 with Chapter 4 of ASCE 7-10 and is intended to further revise Section 1607 without any overlapping or conflicting changes between the two proposals.

A separate proposal also revises Item 4 of Table 1607.1 with respect to the live loads. These revisions are not repeated in this proposal but, should both proposals be approved by the membership, the proponent intends that Footnote (m) be specified for the uniform live loads at stages and platforms.

A separate proposal also revises Item 29 of Table 1607.1 in conjunction with correlating the IBC with changes to the provisions of ASCE 7-10 where reduction of live loads at floors and occupied roofs is restricted or prohibited. These revisions are not repeated in this proposal but, should both proposals be approved by the membership, the proponent intends that Footnote (m) be specified for the uniform live load at roofs used for assembly purposes but that all other changes to Item 29 in this proposal be disregarded.

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

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

ICCFILENAME: Brazil-S45-T1607.1

S61–09/10

Table 1607.1; IRC Table R301.5

Proponent: Gary Ehrlich, PE, National Association of Home Builders (NAHB)

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

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

(No change to footnotes a through i)

j. For attics with limited storage ~~and constructed with trusses~~, this live load need only be applied to those portions of the ceiling joist or the truss bottom chord where there are two or more adjacent trusses or rafter/ceiling joist assemblies with the same web member or purlin brace, rafter tie, or collar tie configuration capable of containing a rectangle 42 inches high by 2 feet wide or greater, located within the plane of the truss or rafter/ceiling joist assembly. The rectangle shall fit between the top of the bottom chord and the bottom of any other truss member, or the top of ceiling joist and bottom of any other roof framing member, provided that each of the following criteria is met:
   i. The attic area is accessible by a pull-down stairway or framed opening in accordance with Section 1209.2, and

ii. The truss shall have a bottom chord pitch less than 2:12.
iii. Bottom chords of trusses shall be designed for the greater of actual imposed dead load or 10 psf, uniformly distributed over the entire span.

(No change to footnotes k and l)

PART II – IRC BUILDING/ENERGY

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

(No change to footnotes a through f)

g. For attics with limited storage~~ and constructed with trusses~~, this live load need be applied only to those portions of the ceiling joist or the truss bottom chord where there are two or more adjacent trusses or rafter/ceiling joist assemblies with the same web member or purlin brace, rafter tie, or collar tie configuration capable of containing a rectangle 42 inches high by 2 feet wide or greater, located within the plane of the truss or rafter/ceiling joist assembly. The rectangle shall fit between the top of the bottom chord and the bottom of any other truss member, or the top of ceiling joist and bottom of any other roof framing member, provided that each of the following criteria is met:

1. The attic area is accessible by a pull-down stairway or framed opening in accordance with Section 807.1.
2. The truss shall have a bottom chord pitch, or the ceiling joist shall have a slope, less than 2:12.
3. Required insulation depth is less than the bottom chord member or ceiling joist depth.

Bottom chords of trusses, and ceiling joists, meeting the above criteria for limited storage shall be designed for the greater of actual imposed dead load or 10 psf, uniformly distributed over the entire span.

(No change to footnote h)

Reason: (IBC) The purpose of this proposal is to allow the relaxed attic storage live load requirement to be applied to roof rafter/ceiling joist assemblies as well as trusses. Rafter/ceiling joist assemblies constructed in accordance with either Section 2308.10.4 are required to have collar ties (per Section 2308.10.4.1) and may have rafter ties (also per Section 2308.10.4.1) and purlin braces (per Section 2308.10.5). Similar requirements would apply for a roof assembly constructed in accordance with the AF&PA WFCM. It is possible these conventionally-framed assemblies can meet the same geometric criteria applied in footnote "f" for trusses. Therefore, it makes sense to permit the same relaxation to be applied to rafter/ceiling joist assemblies. This may be particularly useful in renovations where it is desired to add access to an existing attic, which might otherwise trigger extensive strengthening or replacement of the ceiling joists.
(IRC) The purpose of this proposal is to allow the relaxed attic storage live load requirement to be applied to roof rafter/ceiling joist assemblies as well as trusses. Rafter/ceiling joist assemblies constructed in accordance with Section R802 are required to have collar ties (per Section R802.3.1) and may have rafter ties (per Section R802.3.1) and purlin braces (per Section R802.5.1). It is possible these conventionally-framed assemblies can meet the same geometric criteria applied in footnote "g" for trusses. Therefore, it makes sense to permit the same relaxation to be applied to rafter/ceiling joist assemblies. This may be particularly useful in renovations where it is desired to add access to an existing attic, which might otherwise trigger extensive strengthening or replacement of the ceiling joists.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: Ehrlich-S2-T1607 1

S62–09/10

Table 1607.1; IRC Table R301.5

Proponent: John England, MCO, England Enterprises Inc., representing self

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
5. Balconies (exterior) and decks[h]	~~Same as occupancy served~~ <u>60 minimum or same as occupancy served-whichever is greater</u>	—

(Portions of table not shown remain unchanged)
(No change to footnotes)

PART II – IRC BUILDING/ENERGY

Revise as follows:

TABLE R310.5
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS
(in pounds per square foot)

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

(Portions of table not shown remain unchanged)
(No change to footnotes)

Reason: Decks have been collapsing in residential occupancies and a 60# live load is not unreasonable. Hot tubs, and large groups of people are on decks having parties --the 60# live load will provide more life safety to the occupants on the decks

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: England-S2-T1607.1

S63–09/10
202

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

Revise text as follows:

SECTION 202
DEFINITIONS

AWNING. An architectural projection that provides weather protection, identity or decoration and is <u>partially or</u> wholly supported by the building to which it is attached. An *awning* is comprised of a lightweight frame structure over which a covering is attached

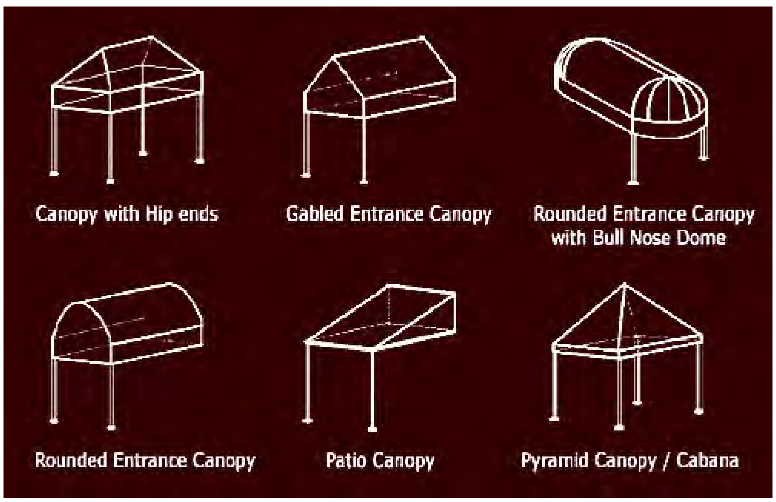
CANOPY. A permanent structure or architectural projection of rigid construction over which a covering is attached that provides weather protection, identity or decoration.~~,~~ A canopy is permitted to ~~and shall~~ be structurally independent or supported by attachment to a building ~~on one end and by not less than one stanchion on the outer end~~ on one or more sides. Canopies shall be sloped more than 25 degrees from the horizontal or so constructed so as to inhibit access other than for maintenance functions.

CORNICE. A projection at the top of a wall or a projecting element over an architectural feature, such as a doorway. Portions of a cornice which are sloped less than 25 degrees from the horizontal and are less than 10 feet (3.05 m) above the ground, more than 10 feet (3.05 m) below an adjacent roof, or located less than 10 feet (3.05 m) from operable openings above or adjacent to the level of the cornice, shall be designed for the live load from Table 1607.1.

MARQUEE. A permanent roofed structure attached to and supported by the building on one or more sides ~~and~~ that ~~projects into the public right-of-way~~ has a top surface which is sloped less than 25 degrees from the horizontal. A marquee shall be less than 10 feet (3.05 m) above the ground, more than 10 feet (3.05 m) below an adjacent roof, or located less than 10 feet (3.05 m) from operable openings above or adjacent to the level of the marquee.

Reason: The current definitions for Awning, canopy and marquee are not adequate.

Lightweight, fabric covered, frame structures also have stanchion(s), in which case the awning definition would not apply. This doesn't make them canopies. Awnings are listed in Table 1607.1, Item 11 with a live load of 5 psf.



Architectural projections of rigid construction over which a covering is attached don't always have stanchion(s). If they do not, they are not defined in the IBC. What if, instead of a stanchion, the canopy cantilevers from the building, or has a hanger rod, chain or cable suspension system?


Copywrited MMP

Cornices are not defined in the IBC, yet they are listed in Table 1607.1, Item 11 with a live load of 60 psf.

Currently, a Marquee must project over the public right-of-way. It is listed in Table 1607.1, Item 24 with a live load of 75 psf. If it doesn't project over the public right-of-way, what is it and what live load should it be designed for? The chapter-by-chapter synopsis for Chapter 32 on page xii of the 2009 IBC notes that "steps, columns, awnings, canopies, marquees, signs, windows, balconies and similar architectural features above grade" can all encroach into the public right-of-way. This effectively negates the definition of a marquee.

With these problems, the definitions in the IBC are not enforceable.

The definition of an "Awing" is retained. However, it can now have a stanchion. The proposed definition, which is tied to a 5 psf live load in Table 1607.1, is now keyed to the lightweight frame structure.

The definition of a "Canopy" is retained. However, instead of relying on a stanchion for its defining characteristic, it is defined by its permanent, rigid construction and its function of providing weather protection, identity or decoration.

From the position of structural engineers, architects and building officials, these definitions need to be able to be tied to Table 1607.1. The proposed revisions do this and include a discernable intent to allow for better code interpretation for other, undefined situations.

That is, when the canopy is like a roof, it is designed for 20 psf, like a roof structure. If a canopy, marquee or cornice has a reasonably flat surface, and is accessible, such as by a short ladder or an operable opening, so that the public might be inclined to get onto it, then it should be designed for a more robust live load.

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

S64–09/10
202, Table 1607.1

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

Revise as follows:

SECTION 202
DEFINITIONS

AWNING. An architectural projection that provides weather protection, identity or decoration and is partially or wholly supported by the building to which it is attached. An *awning* is comprised of a lightweight frame structure over which a covering is attached

CANOPY. A permanent structure or architectural projection of rigid construction over which a covering is attached that provides weather protection, identity or decoration. ~~,~~ A canopy is permitted to ~~and shall~~ be structurally independent or supported by attachment to a building ~~on one end and by not less than one stanchion on the outer end~~ on one or more sides. Canopies shall be sloped more than 25 degrees from the horizontal or so constructed so as to inhibit access other than for maintenance functions.

CORNICE. A projection at the top of a wall or a projecting element over an architectural feature, such as a doorway.

MARQUEE. A permanent roofed structure attached to and supported by the building on one or more sides ~~and projects into the public right-of-way~~.

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
11. Cornices[m]	60	—
24. Marquees[n]	75	—

(Remainder of Table remains unchanged)

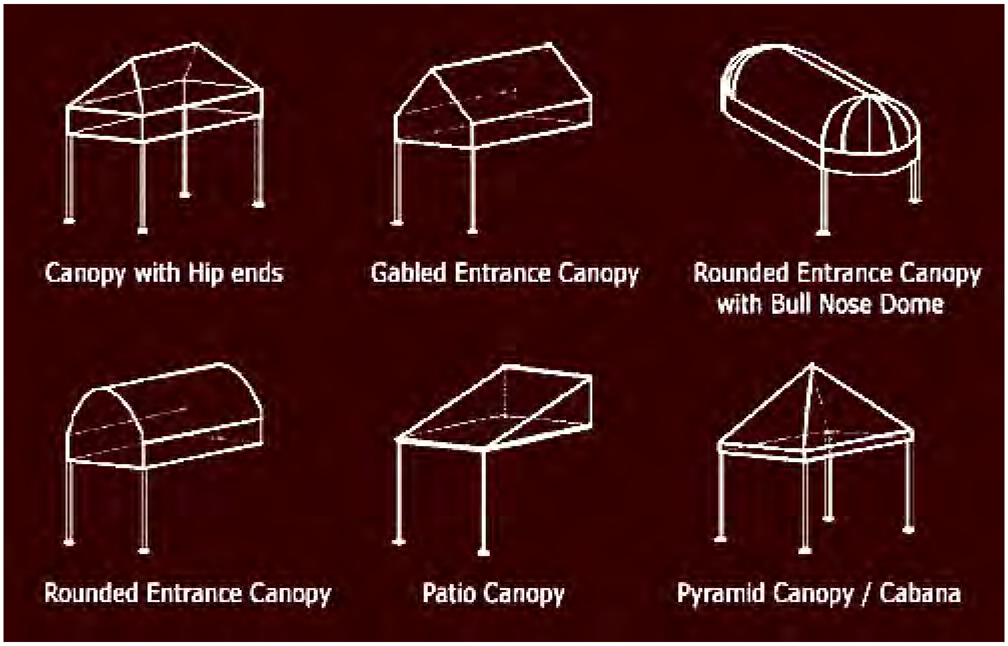
a. through l. (No change to text.)

m. Imposed live load for portions of a cornice which are sloped less than 25 degrees from the horizontal and are less than 10 feet above the ground, more than 10 feet below an adjacent roof, or located less than 10 feet from operable openings above or adjacent to the level of the cornice, shall be designed for the live load from Table 1607.1.

n. Imposed live load for portions of a marquee which are sloped less than 25 degrees from the horizontal. A marquee shall be less than 10 feet above the ground, more than 10 feet below an adjacent roof, or located less than 10 feet from operable openings above or adjacent to the level of the marquee.

Reason: The current definitions for Awning, canopy and marquee are not adequate.

Lightweight, fabric covered, frame structures also have stanchion(s), in which case the awning definition would not apply. This doesn't make them canopies. Awnings are listed in Table 1607.1, Item 11 with a live load of 5 psf.



Architectural projections of rigid construction over which a covering is attached don't always have stanchion(s). If they do not, they are not defined in the IBC. What if, instead of a stanchion, the canopy cantilevers from the building, or has a hanger rod, chain or cable suspension system?


Copywrited MMP

Cornices are not defined in the IBC, yet they are listed in Table 1607.1, Item 11 with a live load of 60 psf.

Currently, a Marquee must project over the public right-of-way. It is listed in Table 1607.1, Item 24 with a live load of 75 psf. If it doesn't project over the public right-of-way, what is it and what live load should it be designed for? The chapter-by-chapter synopsis for Chapter 32 on page xii of the 2009 IBC notes that "steps, columns, awnings, canopies, marquees, signs, windows, balconies and similar architectural features above grade" can all encroach into the public right-of-way. This effectively negates the definition of a marquee.

With these problems, the definitions in the IBC are not enforceable.

The definition of an "Awing" is retained. However, it can now have a stanchion. The proposed definition, which is tied to a 5 psf live load in Table 1607.1, is now keyed to the lightweight frame structure.

The definition of a "Canopy" is retained. However, instead of relying on a stanchion for its defining characteristic, it is defined by its permanent, rigid construction and its function of providing weather protection, identity or decoration.

From the position of structural engineers, architects and building officials, these definitions need to be able to be tied to Table 1607.1. The proposed revisions do this and include a discernable intent to allow for better code interpretation for other, undefined situations.

That is, when the canopy is like a roof, it is designed for 20 psf, like a roof structure. If a canopy, marquee or cornice has a reasonably flat surface, and is accessible, such as by a short ladder or an operable opening, so that the public might be inclined to get onto it, then it should be designed for a more robust live load. This version of this code change proposal accomplishes this through footnotes to Table 1607.1.

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

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

ICCFILENAME: HUSTON-S6-Table 1607.1

S65–09/10
Table 1607.1

Proponent: Larry Wainright, Qualtim, Inc., representing the Structural Building Components Association (SBCA)

Revise as follows:

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND
MINIMUM CONCENTRATED LIVE LOADS[g]
(Portions of table and footnotes not shown remain unchanged)

j. For attics with limited storage and constructed with trusses, this live load need only be applied to those portions of the bottom chord where there are two or more adjacent trusses with the same web configuration capable of containing a rectangle 42 inches high by 2 feet wide or greater, located within the plane of the truss. The rectangle shall fit between the top of the bottom chord and the bottom of any other truss member, provided that each of the following criteria is met:

i. The attic area is accessible by a pull-down stairway or framed opening in accordance with Section 1209.2 and
ii. The truss shall have a bottom chord pitch less than 2:12, <u>and</u>
iii. <u>Required insulation depth is less than the bottom chord member depth.</u>

Bottom chords of trusses <u>meeting the above criteria for limited storage</u> shall be designed for the greater of actual imposed dead load or 10 psf, uniformly distributed over the entire span.

Reason: The purpose of the code change is to update the code language in the IBC by harmonizing it with the language currently in the IRC. Table R301.5, footnote g states:

For attics with limited storage and constructed with trusses, this live load need be applied only to those portions of the bottom chord where there are two or more adjacent trusses with the same web configuration capable of containing a rectangle 42 inches high or greater by 2 feet wide or greater, located within the plane of the truss. The rectangle shall fit between the top of the bottom chord and the bottom of any other truss member, provided that each of the following criteria is met.

1. The attic area is accessible by a pull-down stairway or framed in accordance with Section R807.1.
2. The truss has a bottom chord pitch less than 2:12.
3. Required insulation depth is less than the bottom chord member depth.

The bottom chords of trusses meeting the above criteria for limited storage shall be designed for the greater of the actual imposed dead load or 10 psf, uniformly distributed over the entire span.

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

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

ICCFILENAME: WAINRIGHT-S1- TABLE 1607.1

S66–09/10
Table 1607.1; IRC Table R301.5

Proponent: Larry Wainright, Qualtim, Inc., representing the Structural Building Components Association (SBCA)

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC-STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND
MINIMUM CONCENTRATED LIVE LOADS[g]
(No changes to the Table)

(No change to footnotes a through i)

j. For attics with limited storage and constructed with trusses, this live load need only be applied to those portions of the bottom chord where there are two or more adjacent trusses with the same web configuration capable of containing a rectangle 42 inches high by 2 feet wide or provided that each of the following criteria is met:
   i. The attic area is accessible by a pull-down stairway or framed opening in accordance with Section 1209.2 and
   ii. The truss shall have a bottom chord pitch less than 2:12.
   ~~iii.~~ Bottom chords of trusses meeting the above criteria for limited storage shall be designed ~~for the greater of actual imposed dead load or 10 psf, uniformly distributed over the entire span~~ in accordance with Section 1606.2.

(No change to footnotes l and l)

PART II – IRC BUILDING/ENERGY

Revise as follows:

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

(Portions of table not shown remain unchanged)

(No change to footnotes a through f)

g. For attics with limited storage and constructed with trusses, this live load need be applied only to those portions of the bottom chord where there are two or more adjacent trusses with the same web configuration capable of containing a rectangle 42 inches high or greater by 2 feet wide or greater, located within the plane of the truss. The rectangle shall fit between the top of the bottom chord and the bottom of any other truss member, provided that each of the following criteria is met:
   1. The attic area is accessible by a pull-down stairway or framed opening in accordance with Section R807.1.
   2. The truss has a bottom chord pitch less than 2:12.
   3. Required insulation depth is less than the bottom chord member depth.

   The bottom chords of trusses meeting the above criteria for limited storage shall be designed ~~for the greater of the actual imposed dead load or 10 psf, uniformly distributed over the entire span.~~ in accordance with Section R301.4

(No change to footnotes h and i)

Reason:
PART I- The requirement for a minimum 10 pound load is contrary to Section 1606.2 which requires buildings to be designed for the actual dead load imposed.

1606.2 Design dead load. For purposes of design, the actual weights of materials of construction and fixed service equipment shall be used. In the absence of definite information, values used shall be subject to the approval of the building official.

Trusses should not be subject to different loading conditions than other building elements. Accepted engineering practice and the building code already stipulate that the use of the actual dead loads is appropriate.

PART II- The requirement for a minimum 10 pound load is unnecessary and places attics with limited storage and constructed with trusses at a competitive disadvantage with other framing methods. Section R301.4 requires the use of actual dead loads.

R301.4 Dead load. The actual weights of materials and construction shall be used for determining dead load with consideration for the dead load of fixed service *equipment*.

Trusses should not be subject to different loading conditions than other framing methods. Accepted engineering practice and the building code already stipulate the use of the actual dead loads is appropriate.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: WAINRIGHT-S2-1607.1

S67–09/10

1602.1, 1607.4, 1607.7.1-1607.7.1.2, 1607.7.2, 1607.7.3, 1607.8, 1607.9.1, 1607.10, 1607.11.2.1, 1607.11.2.2, 1607.12.3, Table 1607.1

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

Revise as follows:

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

VEHICLE BARRIER SYSTEM. A system of ~~building~~ components near open sides of a garage floor or ramp or building walls that act as restraints for vehicles.

1607.4 Concentrated loads. Floors and other similar surfaces shall be designed to support the uniformly distributed live loads prescribed in Section 1607.3 or the concentrated load, in pounds (kilonewtons), given in Table 1607.1, whichever produces the greater load effects. Unless otherwise specified, the indicated concentration shall be assumed to be uniformly distributed over an area 2-1/2 feet by 2-1/2 feet ~~[6-1/4 square feet (0.58 m^2)]~~ (762 mm by 762 mm) and shall be located so as to produce the maximum load effects in the ~~structural~~ members.

1607.7.1 Handrails and guards. Handrails and guards shall be designed to resist a load of 50 pounds per linear foot (plf) (0.73 kN/m) applied in any direction ~~at the top~~ along the handrail or top rail and to transfer this load through the supports to the structure. Glass handrail assemblies and guards shall also comply with Section 2407.

Exceptions:

1. For one- and two-family dwellings, only the single concentrated load required by Section 1607.7.1.1 shall be applied.
2. In Group I-3, F, H and S occupancies, for areas that are not accessible to the general public and that have an occupant load less than 50, the minimum load shall be 20 pounds per foot (0.29 kN/m).

1607.7.1.1 Concentrated load. Handrails and guards shall be ~~able~~ designed to resist a single concentrated load of 200 pounds (0.89 kN), applied in any direction at any point ~~along the top~~ on the handrail or top rail so as to produce the maximum load effects, and to transfer this load through the supports to the structure. This load need not be assumed to act concurrently with the loads specified in Section 1607.7.1.

1607.7.1.2 ~~Components~~ Intermediate rails. Intermediate rails (all those except the handrail and top rail), balusters and panel fillers shall be designed to withstand a horizontally applied normal load of 50 pounds (0.22 kN) on an area ~~equal to 1 square foot (0.093m^2)~~ not to exceed 12 inches by 12 inches (305 m by 305 m), including openings and space between rails, and located so as to produce the maximum load effects. Reactions due to this loading are not required to be superimposed with ~~those of~~ the loads specified in Section 1607.7.1 or 1607.7.1.1.

1607.7.2 Grab bars, shower seats and dressing room bench seats. Grab bars, shower seats and dressing room bench seat systems shall be designed to resist a single concentrated load of 250 pounds (1.11 kN) applied in any direction at any point on the grab bar or seat so as to produce the maximum load effects.

1607.7.3 Vehicle barrier systems. Vehicle barrier systems for passenger vehicles shall be designed to resist a single load of 6,000 pounds (26.70 kN) applied horizontally in any direction to the barrier system and shall have anchorage or attachment capable of transmitting this load to the structure. For design of the system, two loading conditions shall be analyzed. The first condition shall apply the load at a height of 1 foot, 6 inches (457 mm) above the floor or ramp surface. The second loading condition shall apply the load at 2 feet, 3 inches (686 mm) above the floor or ramp surface. The more severe load condition shall govern the design of the barrier restraint system. The load shall be assumed to act on an area not to exceed ~~1 square foot (305 mm^2)~~ 12 inches by 12 inches (305 mm by 305 mm), and located so as to produce the maximum load effects. This load is not required to ~~be assumed to~~ act concurrently with any handrail or guard loadings specified in Section 1607.7.1. Garages accommodating trucks and buses shall be designed in accordance with an approved method that contains provision for traffic railings.

1607.8 Impact loads. The live loads specified in Section ~~1607.3~~ 1607.2 shall be assumed to include adequate allowance for ordinary impact conditions. Provisions shall be made in the structural design for uses and loads that involve unusual vibration and impact forces.

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

$L = L_o\,[0.25 + 15 / (\sqrt{K_{LL}\, A_T})]$ **(Equation 16-22)**

In SI: $L = L_o\,[0.25 + 4.57 / (\sqrt{K_{LL}\, A_T})]$

where:

L = Reduced design live load per square foot (~~meter~~ $\underline{m^2}$) of area supported by the member.

L_o = Unreduced design live load per square foot (~~meter~~ $\underline{m^2}$) of area supported by the member (see Table 1607.1).

K_{LL} = Live load element factor (see Table 1607.9.1).

A_T = Tributary area, in square feet (~~square meters~~ $\underline{m^2}$).

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

1607.10 Distribution of floor loads. Where uniform floor live loads are involved in the design of structural members arranged so as to create continuity, the minimum applied loads shall be the full dead loads on all spans in combination with the floor live loads on spans selected to produce the greatest <u>load</u> effect at each location under consideration. It shall be permitted to reduce floor live loads in accordance with Section 1607.9.

1607.11.2.1 Flat, pitched and curved roofs. Ordinary flat, pitched and curved roofs, and awnings and canopies other than of fabric construction supported by <u>a</u> ~~lightweight rigid~~ skeleton structure~~s~~, are permitted to be designed for a reduced roof live load as specified in the following equations or other controlling combinations of loads <u>as specified</u> in Section 1605, whichever produces the greater load <u>effect</u>.

In structures such as greenhouses, where special scaffolding is used as a work surface for workers and materials during maintenance and repair operations, a lower roof load than specified in the following equations shall not be used unless approved by the building official. Such structures shall be designed for a minimum roof live load of 12 psf (0.58 kN/m^2).

$L_r = L_{\underline{r}o}\, R_1\, R_2$ **(Equation 16-25)**

where: $12 \leq L_r \leq 20$

For SI: $L_r = L_{\underline{r}o}\, R_1\, R_2$

where: $0.58 \leq L_r \leq 0.96$

<u>L_{ro} = Unreduced roof live load per square foot (m^2) of horizontal projection supported by the member (see Table 1607.1).</u>

L_r = Reduced <u>roof</u> live load per square foot (m^2) of horizontal projection ~~in pounds per square foot (kN/m^2)~~ <u>supported by the member</u>.

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

$R_1 = 1$ for $A_t \leq$ 200 square feet (18.58 m^2) **(Equation 16-26)**

$R_1 = 1.2 - 0.001\, A_t$ for 200 square feet $< A_t <$ 600 square feet **(Equation 16-27)**

For SI: $1.2 - 0.011\, A_t$ for 18.58 square meters $< A_t <$ 55.74 square meters

$R_1 = 0.6$ for $A_t \geq$ 600 square feet (55.74 m^2) **(Equation 16-28)**

where:

A_t = Tributary area (span length multiplied by effective width) in square feet (m^2) supported by ~~any structural~~ <u>the</u> member, and

$R_2 = 1$ for $F \leq 4$ **(Equation 16-29)**

R_2 = 1.2 – 0.05 F for 4 < F < 12 **(Equation 16-30)**

R_2 = 0.6 for $F \geq 12$ **(Equation 16-31)**

where:

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

1607.11.2.2 Special-purpose roofs. Roofs used for promenade purposes, roof gardens, assembly purposes or other special purposes, and marquees, shall be designed for a minimum live load, L_o, as specified in Table 1607.1. Such live loads are permitted to ~~be~~ have their uniformly distributed live loads reduced in accordance with Section 1607.9. Live loads of 100 psf or more at areas of roofs classified as Group A occupancies shall not be reduced.

1607.12.3 Lateral force. The lateral force on crane runway beams with electrically powered trolleys shall be calculated as 20 percent of the sum of the rated capacity of the crane and the weight of the hoist and trolley. The lateral force shall be assumed to act horizontally at the traction surface of a runway beam, in either direction perpendicular to the beam, and shall be distributed ~~according~~ with due regard to the lateral stiffness of the runway beam and supporting structure.

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
4. Assembly areas ~~and theaters~~ Fixed seats (fastened to floor) Follow spot, projections and control rooms Lobbies Movable seats Stages and platforms Other assembly areas	 60 50 100 100 125 100	–
5. Balconies ~~(exterior)~~ and decks [h]	Same as occupancy served	–
7. Catwalks for maintenance access	40	300
9. Corridors~~, except as otherwise indicated~~ First floor Other floors	 100 Same as occupancy served except as indicated	–
13. Elevator machine room grating (on area of ~~4 in^2~~ 2 inches by 2 inches)	–	300
14. Finish light floor plate construction (on area of ~~1 in^2~~ 1 inch by 1 inch)	–	200
18. Gymnasiums~~, main floors and balconies~~	100	–
29. Roofs:		
All roof surfaces subject to maintenance workers		300
Awnings and canopies:		
Fabric construction supported by a ~~lightweight rigid~~ skeleton structure	5 nonreducible	
All other construction	20	
Ordinary flat, pitched, and curved roofs	20	
Primary roof members, exposed to a work floor:		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages		
All other ~~occupancies~~ primary roof members		2,000
Roofs used for other special purposes		300
Roofs used for promenade purposes	Note l	Note l
Roofs used for roof gardens or assembly purposes	60 100	
35. Stairs and exits One- and two-family dwellings All other	 40 100	~~Note f~~ 300 [f] 300 [f]

(Portions of table not show are unchanged)

a. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 1607.1 or the following concentrated loads: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 pounds acting on an area of 4.5 inches by 4.5 inches; (2) for mechanical parking structures without slab or deck ~~which~~ that are used for storing passenger vehicles only, 2,250 pounds per wheel.

d. Other uniform loads in accordance with an approved method ~~which contains~~ containing provisions for truck loadings shall also be considered where appropriate.

f. The minimum concentrated load on stair treads ~~(~~shall be applied on an area of ~~4 square~~ 2 inches by 2 inches~~) is 300 pounds~~. This load need not be assumed to act concurrently with the uniform load.

(Footnotes not shown are unchanged)

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. The need for correlation is due to ASCE 7 Proposals LLSC-LL6, LLSC-LL9 and LLSC-LL11, which were approved by the Live Load Subcommittee (LLSC) and are being balloted by the Main Committee (Items #1, #3, #5 and #6 of the Second Main Committee Ballot on Live/Dead Load Provisions). It is expected that the Main Committee will approve the proposals.

The proposed changes are seen as editorial. The addition of a definition for the symbol of unreduced roof live load in Section 1607.11.2.1, along with the current definition for the symbol of reduced roof live load, is done for consistency with the current definitions for the symbols of unreduced and reduced live load in Section 1607.9. In Table 1607.1, the addition of "for maintenance access" after "catwalks" and the addition of separate items for the first floor and other floor at "corridors" is done for consistency with the current text in Table 4-1 of ASCE 7-05, which will remain unchanged in ASCE 7-10.

The addition of footnote (c) to the 60 psf uniform load for fixed seats (fastened to the floor) in Item #4 on assembly areas for consistency with the same footnote at bleachers and fixed seats (fastened to the floor) in Item #34 on stadiums and arenas. This was suggested as an additional revision to Proposal LLSC-LL9 for the same item in Table 4-1 of ASCE 7. The LLSC considered the suggestion to have merit but it was determined to be a technical change and beyond the scope of what was intended to be an editorial proposal. In spite of this, The ICC Structural Committee may wish to consider adding it to IBC Table 1607.1.

A separate proposal also revises Section 1607.7.1.2. These revisions are not repeated in this proposal but, should both proposals be approved by the membership, the proponent intends that the addition of "and top rail" in Section 1607.7.1.2 of this proposal be disregarded.

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

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

ICCFILENAME: Brazil-S50-1607.4

S68–09/10

1607.5, 1607.13, 1607.13.1

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

Revise as follows:

1607.5 Partition loads. In office buildings ~~and in~~ or other buildings where partitions ~~locations are subject to change~~ will be erected or rearranged, provisions for partition ~~weight~~ live load shall be made, whether or not partitions are shown on the construction documents, unless the specified live load exceeds 80 psf (3.83 kN/m^2). The partition live load shall not be less than a uniformly distributed live load of 15 psf (~~0.74~~ 0.72 kN/m^2).

1607.13 Interior walls and partitions. Interior walls and partitions that exceed 6 feet (1829 mm) in height, including their finish materials, shall have adequate strength to resist the loads to which they are subjected but not less than a uniformly distributed horizontal live load of 5 psf (0.24 kN/m^2) applied normal to the plane of the partition.

> **Exception:** Fabric partitions complying with Section 1607.13.1 shall not be required to resist the minimum horizontal live load of 5 psf (0.24 kN/m^2).

1607.13.1 Fabric partitions. Fabric partitions that exceed 6 feet (1829 mm) in height, including their finish materials, shall have adequate strength to resist the following load conditions:

1. A uniformly distributed horizontal ~~distributed~~ live load of 5 psf (0.24 kN/m^2) applied to the partition framing. The total area used to determine the ~~distributed~~ live load shall be the area of the fabric face between the framing members to which the fabric is attached. The total ~~distributed~~ live load shall be ~~uniformly~~ applied to such framing members in proportion to the length of each member.
2. A concentrated live load of 40 pounds (0.176 kN) applied ~~to an 8-inch (203 mm) diameter area [50.3 square inches (32 452 mm^2)]~~ on an area of the fabric face a maximum of 8 inches in diameter (32.43 m^2) and at a minimum height of 54 inches (1372 mm) above the floor.

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. The need for correlation is due to ASCE 7 Proposal SSC TC-2-CH4,5,11,12-01, which was approved by the Seismic Subcommittee on February 20, 2009 and is being balloted by the Main Committee (Item #1 of the Fourth Main Committee Ballot on Seismic Provisions). It is expected that the Main Committee will approve the proposal.
The proposed changes to Section 1607.5 are intended to reduce confusion over whether this load is a live load or a dead load and how it affects the determination of effective seismic weight. This load acts as a budget to account for partitions that may be installed during the life of the building or structure but are not typically installed when the building or structure is first constructed. The ASCE 7 proposal makes the same changes in Section 4.2.2 of ASCE 7 and also makes a correlating change in Item #2 of Section 12.7.2 in ASCE 7 by specifying the partition load as a live load.
The changes to Sections 1607.13 and 1607.13.1 are seen as editorial and are proposed for consistency with the format of similar provisions in ASCE 7.

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

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

ICCFILENAME: Brazil-S43-1607.5

S69–09/10
1602.1, 1605.4, 1607.6 (New), Table 1607.1

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

1. Add new definition as follows:

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

HELIPAD. A structural surface that is used for the landing, taking off, taxiing and parking of helicopters.

2. Revise as follows:

~~**1605.4**~~ **1607.6** ~~**Heliports and helistops**~~ **Helipads.** ~~Heliport and helistop landing areas~~ Helipads shall be designed for the following live loads~~, combined in accordance with Section 1605~~:

~~1. Dead load, *D*, plus the gross weight of the helicopter, D_h, plus snow load, *S*.~~

~~3~~ 1. ~~Dead load, *D*, plus~~ A uniform live load, *L*, ~~of:~~ as specified below. This load shall not be reduced.

   1.1. ~~100~~ 40 psf (~~4.79~~ 1.92 kN/m^2) where the design basis helicopter has a maximum take-off weight of 3,000 pounds (13.35 kN) or less.

   1.2. 60 psf (2.87 kN/m^2) where the design basis helicopter has a maximum take-off weight greater than 3,000 pounds (13.35 kN).

2. A single concentrated live load, *L*, of 3,000 pounds (13.35 kN) applied over an area of 4.5 inches by 4.5 inches (114 mm by 114 mm) and located so as to produce the maximum load effects on the structural elements under consideration. The concentrated load need not be assumed to act concurrently with other uniform or concentrated live loads.

~~2~~ 3. ~~Dead load, *D*, plus~~ Two single concentrated ~~impact~~ live loads, *L*, ~~approximately~~ 8 feet (2438 mm) apart applied ~~anywhere~~ on the ~~touchdown~~ landing pad (representing ~~each of~~ the helicopter's two main landing gear, whether skid type or wheeled type), each having a magnitude of 0.75 times the ~~gross~~ maximum take-off weight of the helicopter, and located so as to produce the maximum load effects on the structural elements under consideration. ~~Both loads acting together total 1.5 times the gross weight of the helicopter.~~ The concentrated loads shall be applied over an area of 8 inches by 8 inches (203 mm by 203 mm) and need not be assumed to act concurrently with other uniform or concentrated live loads.

   ~~**Exception:**~~ Landing areas designed for a design basis helicopter~~s~~ with ~~gross~~ maximum take-off weight~~s~~ not exceeding 3,000 pounds (13.35 kN) ~~in accordance with Items 1 and 2 shall be permitted to be designed using a 40 psf (1.92 kN/m^2) uniform live load in Item 3, provided the landing area is~~ shall be identified with a 3,000 pound (13.34 kN) weight limitation. ~~This 40 psf (1.92 kN/m^2) uniform live load shall not be reduced.~~ The landing area weight limitation shall be indicated by the numeral "3" (kips) located in the bottom right corner of the landing area as viewed from the primary approach path. The indication for the landing area weight limitation shall be a minimum 5 feet (1524 mm) in height.

   (Renumber subsequent sections)

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
20. Helipads	See Section 1607.6	

(Portions of table not shown are unchanged)

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. The need for correlation is due to ASCE 7 Proposal LLSC-LL-7, which has been approved by the Live Load Subcommittee and is being balloted by the Main Committee (Item #4 of the Second Main Committee Ballot on Live/Dead Load Provisions). It is expected that the Main Committee will approve the proposal.

The listing of loads other than live loads and the requirement in the charging text that the listed loads be combined in accordance with Section 1605 are deleted to eliminate conflicts with the load combinations of Section 1605. Current Items #1 through #3 appear to specify certain load combinations, not loads, and it is not clear whether these load combinations are in addition to, are intended to modify, or are intended to replace the load combinations of Section 1605. Sections 1605.1 through 1605.3.2.1 specify load combinations for buildings and other structures, and portions thereof, which would include helipads. If the current charging text intends to specify loads that shall be combined in accordance with Section 1605, the items listing loads should be limited to those loads unique to helipads. In this regard, references to the dead load, *D*, and snow load, *S*, serve no purpose. The loading provisions are also relocated to Section 1607.6 on live loads.

The change from "heliports and helistops" to "helipads" is for consistency with the same change being made in ASCE 7-10 and will have the effect of providing a term unique to the structural chapters of the IBC.

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

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

ICCFILENAME: Brazil-S7-1602.1

S70–09/10

1607.6, 1607.6.1, 1607.6.2-1607.6.5 (New), Table 1607.6

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

1. Delete and Substitute as follows:

~~**1607.6 Truck and bus garages.** Minimum live loads for garages having trucks or buses shall be as specified in Table 1607.6, but shall not be less than 50 psf (2.40 kN/m2), unless other loads are specifically justified and *approved* by the *building official*. Actual loads shall be used where they are greater than the loads specified in the table.~~

~~**1607.6.1 Truck and bus garage live load application.** The concentrated load and uniform load shall be uniformly distributed over a 10-foot (3048 mm) width on a line normal to the centerline of the lane placed within a 12-foot-wide (3658 mm) lane. The loads shall be placed within their individual lanes so as to produce the maximum stress in each structural member. Single spans shall be designed for the uniform load in Table 1607.6 and one simultaneous concentrated load positioned to produce the maximum effect. Multiple spans shall be designed for the uniform load in Table 1607.6 on the spans and two simultaneous concentrated loads in two spans positioned to produce the maximum negative moment effect. Multiple span design loads, for other effects, shall be the same as for single spans.~~

1607.6 Heavy Vehicle Loads. Structures or portions of structures which are subject to heavy vehicle loads shall be designed for the loads from Section 1607.6.1.

1607.6.1 Truck and bus loads. Where any structure does not have provisions to restrict access for trucks and buses that exceed the weight limitations set forth in Table 1607.1 footnote a, those portions of the structure subject to such loads shall be designed using the vehicular live loads, including consideration of impact and fatigue, in accordance with the codes and specifications required by the jurisdiction having authority for the design and construction of the roadways and bridges in the same location of the structure.

2. Add new text as follows:

1607.6.2 Fire truck loading. Where fire department access requires travel over or loading of a structure by fire department vehicles or similar emergency vehicles, the structure shall be designed for the greater of the following loads:

1607.6.2.1 Fire truck operational loads. The actual operational loads (including out rigger reactions and contact areas) of the vehicles as stipulated and / or approved by the local Fire Department or Building Official having jurisdiction for the structure.

1607.6.2.2 Truck and bus loads. The live loading required by section 1607.6.1.

1607.6.3 Truck and bus garages. Garages designed specifically to allow trucks or buses that exceed the weight limitations for passenger vehicles as set forth in Table 1607.1 footnote a, shall be designed using the vehicular live loads, per the Codes and Specifications required by the jurisdiction having authority for the design and construction of the roadways and bridges in the same location of the structure. (Note: design for impact and fatigue in a garage is not required).

> **Exception:** The design live loads and load placement are allowed to be determined using the actual vehicle weights for the vehicles allowed onto the garage floors, provided such loads and placement are based on rational engineering principles and are approved by the Building Official, but shall not be less than 50 psf (this live load shall not be reduced).

1607.6.4 Forklifts and moveable equipment. Where a structure is intended to have forklifts or other moveable equipment present, the structure shall be designed for the total vehicle load and the individual wheel loads for the anticipated vehicles as specified by the owner of the facility. These loads shall be posted per Section 1607.6.5.

1607.6.4.1 Impact and fatigue. Due to the nature of the operations of a facility with forklifts and other moveable equipment, impact loads and fatigue loading must be considered in the design of the supporting structure. This must include consideration for relative stiffness and differential deflections between adjacent framing members; positive and negative moments induced by a moving live load; effects of multiple vehicle loads in the same vicinity; and the punching shear on a slab based on the actual contact area of the wheel loads for the specific vehicle to be used. For the purposes of design, the vehicle and wheel loads shall be increased by 30 percent to account for impact.

1607.6.5 Posting. The maximum weight of the vehicles allowed into or on a garage or other structure shall be conspicuously posted by the owner in accordance with Section 106.1.

3. Delete without substitution:

~~TABLE 1607.6~~
~~UNIFORM AND CONCENTRATED LOADS~~

~~LOADING CLASSa~~	~~UNIFORM LOAD (pounds/linear foot of lane)~~	~~CONCENTRATED LOAD (pounds)b~~	
		~~For moment design~~	~~For shear design~~
~~H20-44 and HS20-44~~	~~640~~	~~18,000~~	~~26,000~~
~~H15-44 and HS15-44~~	~~480~~	~~13,500~~	~~19,500~~

~~For SI: 1 pound per linear foot = 0.01459 kN/m, 1 pound = 0.004448 kN, 1 ton = 8.90 kN.~~

~~a. An H loading class designates a two-axle truck with a semitrailer. An HS loading class designates a tractor truck with a semitrailer. The numbers following the letter classification indicate the gross weight in tons of the standard truck and the year the loadings were instituted.~~

~~b. See Section 1607.6.1 for the loading of multiple spans.~~

Reason: The current Section 1607.6 Truck and bus garages, is addressing truck and bus loads in garages only and does not give direction for heavy vehicle loads in other conditions outside of a "garage". The current section lists loading criteria that appears to be extracted from the live load section from the AASHTO (American Association of State Highway and Transportation Officials) Code. AASHTO is not a referenced standard in the IBC. The current section however does not give other critical loading criteria such as spacing of the concentrated loads or impact requirements. Buildings designed for repair or storage may need to be designed for higher levels of loading than are currently prescribed by Table 1607.6 due to tighter spacing requirements This new section clarifies that for conditions where heavy highway type vehicles have access onto a structure, then that structure will need to be designed using the same code and requirements that the roadways in that jurisdiction are designed under. This loading may in fact be the loading from AASHTO, or the loading for other elements such as lids of large detention tanks or utility vaults. It will likely vary from one Jurisdiction to another. Thus the RDP should consult with the Jurisdiction for design loads for these special conditions. The new language also gives criteria for addressing other heavy vehicle loads (Fire trucks and forklifts), which is currently absent in the current code, and is only mentioned under Section 1607.2 Loads not specified.

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

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

ICCFILENAME: HUSTON-S7-1607.6

S71–09/10

1012.1, 1013.1, 1013.1.1, 1602.1, Table 1607.1, 1607.7, 1607.7.1, 1607.7.1.1, 1607.7.1.2, 1607.7.2, 1607.7.3

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

1. Add new definitions as follows:

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

GRAB BAR SYSTEM. A bar and associated anchorages and attachments to the structural system for the support of body weight in locations such as toilets, showers and tub enclosures.

GUARDRAIL SYSTEM. A system of components, including anchorages and attachments to the structural system, near open sides of an elevated surface for the purpose of minimizing the possibility of a fall from the elevated surface by people, equipment or material.

HANDRAIL SYSTEM. A rail grasped by hand for guidance and support, and associated anchorages and attachments to the structural system.

2. Revise as follows:

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

VEHICLE BARRIER SYSTEM. A system of ~~building~~ components, including anchorages and attachments to the structural system, near open sides of a garage floor or ramp or building walls that act as restraints for vehicles.

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
19. Handrail, guardrail and grab bar systems ~~Handrails, guards and grab bars~~	See Section 1607.7	

(Portions of table not shown remain unchanged)

1607.7 Loads on handrail~~s~~, ~~guards~~ guardrail, grab bar~~s~~, seat~~s~~ and vehicle barrier systems. ~~Handrails~~ Handrail, ~~guards~~ guardrail, grab bar ~~bars~~, accessible seat ~~seats~~, accessible bench ~~benches~~ and vehicle barrier systems shall be designed and constructed to the structural loading conditions set forth in this section.

1607.7.1 Handrails and ~~guards~~ guardrail systems. Handrail ~~Handrails~~ and ~~guards~~ guardrail systems shall be designed to resist a load of 50 plf (0.73 kN/m) applied in any direction at the top ~~and to transfer this load through the supports to the structure~~. Glass handrail ~~assemblies~~ and ~~guards~~ guardrail systems shall also comply with Section 2407.

Exceptions:

1. For one- and two-family dwellings, only the single concentrated load required by Section 1607.7.1.1 shall be applied.
2. In Group I-3, F, H and S occupancies, for areas that are not accessible to the general public and that have an occupant load less than 50, the minimum load shall be 20 pounds per foot (0.29 kN/m).

1607.7.1.1 Concentrated load. Handrail ~~Handrails~~ and ~~guards~~ guardrail systems shall be able to resist a single concentrated load of 200 pounds (0.89 kN), applied in any direction at any point along the top~~, and to transfer this load through the supports to the structure~~. This load need not be assumed to act concurrently with the loads specified in Section 1607.7.1.

1607.7.1.2 Components. Intermediate rails (all those except the handrail), balusters and panel fillers shall be designed to withstand a horizontally applied normal load of 50 pounds (0.22 kN) on an area equal to 1 square foot ($0.093m^2$), including openings and space between rails. Reactions due to this loading are not required to be superimposed with those of Section 1607.7.1 or 1607.7.1.1.

1607.7.2 Grab bar~~s~~, shower seat~~s~~ and dressing room bench seat~~s~~ systems. Grab bar ~~bars~~, shower seat ~~seats~~ and dressing room bench seat systems shall be designed to resist a single concentrated load of 250 pounds (1.11 kN) applied in any direction at any point.

1607.7.3 Vehicle barrier systems. Vehicle barrier systems for passenger vehicles shall be designed to resist a single load of 6,000 pounds (26.70 kN) applied horizontally in any direction to the barrier system ~~and shall have anchorage or attachment capable of transmitting this load to the structure~~. For design of the system, two loading conditions shall be analyzed. The first condition shall apply the load at a height of 1 foot, 6 inches (457 mm) above the floor or ramp surface. The second loading condition shall apply the load at 2 feet, 3 inches (686 mm) above the floor or ramp surface. The more severe load condition shall govern the design of the vehicle barrier ~~restraint~~ system. The load shall be assumed to act on an area not to exceed 1 square foot ($305\ mm^2$), and is not required to be assumed to act concurrently with any handrail or guardrail system loadings specified in Section 1607.7.1. Garages accommodating trucks and buses shall be designed in accordance with an approved method that contains provision for traffic railings.

1012.1 Where required. *Handrails* for *stairways* and *ramps* shall be adequate in strength and attachment in accordance with Section 1607.7 for handrail systems. *Handrails* required for *stairways* by Section 1009.12 shall comply with Sections 1012.2 through 1012.9. *Handrails* required for *ramps* by Section 1010.8 shall comply with Sections 1012.2 through 1012.8.

1013.1 Where required. *Guards* shall be located along open-sided walking surfaces, including *mezzanines*, *equipment platforms*, *stairs*, *ramps* and landings that are located more than 30 inches (762 mm) measured vertically to the floor or grade below at any point within 36 inches (914 mm) horizontally to the edge of the open side. *Guards* shall be adequate in strength and attachment in accordance with Section 1607.7 for guardrail systems.

Exception: *Guards* are not required for the following locations:

1. On the loading side of loading docks or piers.
2. On the audience side of stages and raised platforms, including steps leading up to the stage and raised platforms.
3. On raised stage and platform floor areas, such as runways, ramps and side stages used for entertainment or presentations.
4. At vertical openings in the performance area of stages and platforms.
5. At elevated walking surfaces appurtenant to stages and platforms for access to and utilization of special lighting or equipment.
6. Along vehicle service pits not accessible to the public.
7. In assembly seating where *guards* in accordance with Section 1028.14 are permitted and provided.

1013.1.1 Glazing. Where glass is used to provide a *guard* or as a portion of the *guard* system, the *guard* shall also comply with Section 2407. Where the glazing provided does not meet the strength and attachment requirements of Section 1607.7 for guardrail systems, complying *guards* shall also be located along glazed sides of open-sided walking surfaces.

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. The need for correlation is due to ASCE 7 Proposal LLSC-LL9, which was approved by the Live Load Subcommittee and is being balloted by the Main Committee (Item #5 of the Second Main Committee Ballot on Live/Dead Load Provisions). It is expected that the Main Committee will approve the proposal.

This proposal takes into account the definitions of “guard” and “handrail” in the IBC by limiting the applicability of the proposed definitions in this proposal to Chapter 16, whereas the definitions of “guard” and “handrail” apply throughout the IBC. The definitions in this proposal will establish grab bar, guardrail and handrail systems as structural systems that are required to resist structural design loads and transfer these loads to the supporting structure. This will contrast with guards and handrails whose definitions are primarily utilized in code provisions related to egress and accessibility.

All instances of “guard” in the structural chapters, and all references to Section 1607.7 in the nonstructural chapters, of the 2009 IBC are included in this proposal.

This proposal was prepared in conjunction with a proposal to editorially correlate IBC Section 1607 with Chapter 4 of ASCE 7-10 and is intended to further revise Section 1607 without any overlapping or conflicting changes between the two proposals.

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

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

ICCFILENAME: Brazil-S25-1607.7

S72–09/10
1607.7.1.2

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

Revise as follows:

1607.7.1.2 Components. ~~Intermediate~~ Rails, ~~(all those except the handrail), balusters~~ other than handrails and the top rails of guards, and panel fillers shall be designed to withstand a horizontally applied normal load of 50 pounds (0.22 kN) on an area equal to 1 square foot ($0.093 m^2$), including openings and space between rails. Reactions due to this loading are not required to be superimposed with those of Section 1607.7.1 or 1607.7.1.1.

Reason: This proposal serves two purposes. The first is to replace parenthetical text, which is often viewed as commentary, with mandatory text that is also more comprehensive. The current text is extracted from ASCE 7-05 and is the source of the parenthetical “all those except the handrails.”

The second purpose is to revise text that is not comprehensive in that it requires intermediate rails, balusters and panel fillers to be designed for a load of 50 pounds and includes openings and space between the rails but not between the rails and balusters. This was brought to the attention of the Live Load Subcommittee, which chose to delete balusters rather than add them at the end of the first sentence after “rails.” The

Subcommittee also chose to retain the parenthetical text. The proposal revises the charging text so that specifying balusters will not be necessary by specifying all rails except handrails and the top rails of guards.

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

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

ICCFILENAME: Brazil-S9-1607.7.1.2

S73-09/10
1607.7.3

Proponent: Philip Brazil, PE, SE, representing self

Revise as follows:

1607.7.3 Vehicle barrier systems. Vehicle barrier systems for passenger vehicles shall be designed to resist a single load of 6,000 pounds (26.70 kN) applied horizontally in any direction to the barrier system and shall have anchorage or attachment capable of transmitting this load to the structure. For design of the system, ~~two loading conditions shall be analyzed. The first condition shall apply~~ the load shall be assumed to act at ~~a~~ heights of ~~1 foot, 6~~ 18 to 27 inches (457 to 686 mm) above the floor or ramp surface, located so as to produce the maximum load effects. ~~The second loading condition shall apply the load at 2 feet, 3 inches (686 mm) above the floor or ramp surface. The more severe load condition shall govern the design of the barrier restraint system.~~ The load shall be ~~assumed to act~~ applied on an area not to exceed ~~1 square foot (305 mm^2), and~~ 12 inches by 12 inches (305 mm by 305 mm). The load is not required to ~~be assumed to~~ act concurrently with any handrail or guard loadings specified in Section 1607.7.1. Garages accommodating trucks and buses shall be designed in accordance with an approved method that contains provision for traffic railings.

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. The need for correlation is due to an ASCE 7 Proposal LLSC-LL14, which has been approved by the Live Load Subcommittee. It is expected that the Main Committee will ballot and approve the proposal.

The proposed text will better define the design loads appropriate for vehicle barrier systems. The current text specifies the application of a horizontal load at two heights above the floor or ramp surface: 18 inches and 27 inches. These loads do not adequately account for the range of heights where passenger vehicles will impact a vehicle barrier system or the range of designs utilized for vehicle barrier systems, which include steel cables spanning horizontally between vertical supports. With the current text, it is possible for a vehicle barrier system to be designed with two horizontally spanning steel cables with heights above the floor or ramp surface of 18 inches and 27 inches, respectively. Such an arrangement does not adequately provide for vehicles whose impact with a vehicle barrier system will be between these heights. The proposed text will ensure that all heights between 18 and 27 inches are considered in the design of the vehicle barrier system and that each element of the system is designed for the maximum load effect.

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

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

FILENAME: Brazil-S4-1607.7.3

S74–09/10
1607.7.3

Proponent: Gary R. Searer, Wiss, Janney, Elstner Associates, Inc., representing self

Revise as follows:

1607.7.3 Vehicle barrier systems. Vehicle barrier systems for passenger vehicles shall be designed to resist a single service load of 6,000 pounds (26.70 kN) applied horizontally in any direction to the barrier system and shall have anchorage or attachment to the structure that shall not fail at less than 2.5 times this load. ~~capable of transmitting this load to the structure.~~ For design of the system, two loading conditions shall be analyzed. The first condition shall apply the load at a height of 1 foot, 6 inches (457 mm) above the floor or ramp surface. The second loading condition shall apply the load at 2 feet, 3 inches (686 mm) above the floor or ramp surface. The more severe load condition shall govern the design of the barrier restraint system. The load shall be assumed to act on an area not to exceed 1 square foot (0.0929 m^2), and is not required to be assumed to act concurrently with any handrail or guard loadings specified in Section 1607.7.1. Garages accommodating trucks and buses shall be designed in accordance with an approved method that contains provision for traffic railings.

Reason: In the prior code cycle (IBC S73-07/08), significant and convincing evidence was presented by the Parking Consultants Council of the National Parking Association (PCCNPA) that demonstrated that vehicles are generally heavier than when the provisions for vehicle barriers were first introduced into the code and that nonductile failure of connections was a potential problem. Consequently, the existing requirement in the International Building Code is potentially unconservative.

This proposal addresses two issues:

First, this proposal clarifies that the design load of 6,000 pounds is a service load as opposed to an ultimate load. If an engineer is using strength design or load and resistance factor design, the load should be increased by the appropriate live load factor.

Second, this proposal requires that the design have significant overstrength in the connection design. As it is currently worded, the code only requires that the connections be able to transmit the 6,000 pound load to the structure, potentially resulting in designs that are more likely to fail at the connections. Last cycle's proposal (IBC S73-07/08) to add a requirement that the anchorages be ductile and extend and deform to absorb impact energy prior to ultimate failure was deemed by the Committee to be too vague and unworkable for code language. This proposal will require that the minimum factor of safety for the connections will be 2.5 and will make it more likely that the vehicle barrier will deform and fail within the body of the system, consequently increasing the likelihood that the failure will be ductile and will dissipate more energy than if the failure occurs in the connections of the system to the structure.

The prior code change proposal (S73-07/08 by PCCNPA) and the relevant technical back-up can be found here: http://www.iccsafe.org/cs/codes/2007-08cycle/ProposedChanges/V1_S70-147.pdf

Cost Impact: There may be a small increase in the cost of construction of vehicle barrier systems if this proposal is approved

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

ICCFILENAME: Searer-S1-1607.2.3

S75-09/10
1607.8, 1607.8.1, 1607.8.2

Proponent: Philip Brazil, PE, SE, representing self

1. Revise as follows:

1607.8 Impact loads. The live loads specified in Section<u>s</u> 1607.3 <u>through 1607.7 shall be assumed to</u> include <u>adequate</u> allowance for <u>ordinary</u> impact conditions. Provisions shall be made in the structural design for uses and loads that involve unusual vibration and impact forces.

1607.8.1 Elevators. ~~Elevator loads~~ <u>Members, elements and components subject to dynamic loads from elevators</u> shall be ~~increased by 100 percent~~ <u>designed</u> for impact <u>loads</u> and ~~the structural supports shall be designed within the limits of~~ deflection <u>limits</u> prescribed by ASME A17.1.

1607.8.2 Machinery. For the purpose of design, the weight of machinery and moving loads shall be increased as follows to allow for impact: (1) ~~elevator machinery, 100 percent; (2)~~ light machinery, shaft- or motor-driven, 20 percent; ~~(3)~~ <u>and</u> (<u>2</u>) reciprocating machinery or power-driven units, 50 percent~~; (4) hangers for floors or balconies, 33 percent~~. Percentages shall be increased where specified by the manufacturer.

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. The need for correlation is due to ASCE 7 Proposal LLSC-LL-10, which has been approved by the Live Load Subcommittee. It is expected that the Main Committee will ballot and approve the proposal. The changes in Section 1607.8 are for consistency with the corresponding Section 4.6 of ASCE 7-10.

The changes in Section 1607.8.1 clarify that the impact load from elevators applies specifically to members, elements and components subject to dynamic loading from the elevator mechanism and direct the code user to the elevator standard (ASME A17.1) to determine the increases.

In Section 1607.8.2, elevator machinery is deleted because it is covered by Section 1607.8.1. Hangers for floors or balconies are deleted because live loads should be determined based on their occupancy or use with due consideration for dynamic effects as specified in Section 1607.8. The 33 percent increase has been determined to be an archaic application for a23 perceived lack of redundancy in the structural system. Such an increase should not be imposed on the load side of design equations and, instead, should be a consideration in material standards (e.g., resistance side of design equations).

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

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

FILENAME: Brazil-S6-1607.8

S76-09/10
1607.9-1607.9.2

Proponent: Philip Brazil, PE, SE, representing self

1. Revise as follows:

1607.9 Reduction in live loads. Except for uniform live loads at roofs, all other minimum uniformly distributed live loads, L_o, in Table 1607.1 are permitted to be reduced in accordance with Sections 1607.9.1 ~~or 1607.9.2~~ through 1607.9.6. Roof uniform live loads, other than special purpose roofs of Section 1607.11.2.2 are permitted to be reduced in accordance with Section 1607.11.2. Roof uniform live loads of special purpose roofs are permitted to be reduced in accordance with Sections 1607.9.1 ~~or 1607.9.2~~ through 1607.9.6.

1607.9.1 General. Subject to the limitations of Sections ~~1607.9.1.1~~ 1607.9.2 through ~~1607.9.1.4~~ 1607.9.6, members for which a value of $K_{LL} A_T$ is 400 square feet (37.16 m^2) or more are permitted to be designed for a reduced live load in accordance with the following equation:

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$$ **Equation 16-22)**

$$\text{For SI: } L = L_o\left(0.25 + \frac{4.57}{\sqrt{K_{LL}A_T}}\right)$$

where:

L = Reduced design live load per square foot (meter) of area supported by the member.
L_o = Unreduced design live load per square foot (meter) of area supported by the member (see Table1607.1).
K_{LL} = Live load element factor (see Table 1607.9.1).
A_T = Tributary area, in square feet (square meters).

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

~~1607.9.1.1~~ 1607.9.2 One-way slabs. The tributary area, A_T, for use in Equation 16-22 for one-way slabs shall not exceed an area defined by the slab span times a width normal to the span of 1.5 times the slab span.

~~1607.9.1.2~~ 1607.9.3 Heavy live loads. Live loads that exceed 100 psf (4.79 kN/m^2) shall not be reduced.

Exceptions:

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

~~1607.9.1.3~~ 1607.9.4 Passenger vehicle garages. The live loads shall not be reduced in passenger vehicle garages.

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

~~1607.9.1.4~~ 1607.9.5 Group A occupancies. Live loads of 100 psf (4.79 kN/m^2) and at areas where fixed seats are located shall not be reduced in Group A occupancies.

~~1607.9.1.6~~ 1607.9.6 Roofs members. Live loads of 100 psf (4.79 kN/m^2) or less shall not be reduced for roof members except as specified in Section 1607.11.2.

2. Delete without substitution:

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

1. ~~A reduction shall not be permitted in Group A occupancies.~~
2. ~~A reduction shall not be permitted when the live load exceeds 100 psf (4.79 kN/m²) except that the design live load for members supporting two or more floors is permitted to be reduced by 20 percent.~~

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

3. ~~A reduction shall not be permitted in passenger vehicle parking garages except that the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent.~~
4. ~~For live loads not exceeding 100 psf (4.79 kN/m²), the design live load for any structural member supporting 150 square feet (13.94 m²) or more is permitted to be reduced in accordance with the following equation:~~
5. ~~For one-way slabs, the area, *A*, for use in Equation 16-23 shall not exceed the product of the slab span and a width normal to the span of 0.5 times the slab span.~~

~~*R* = *0.08* (*A* – 150)~~ ~~**(Equation 16-23)**~~

~~For SI: *R* = *0.861* (*A* – 13.94)~~

~~Such reduction shall not exceed the smallest of:~~

~~1.~~ ~~40 percent for horizontal members,~~

~~2.~~ 60 percent for vertical members, or

3. ~~*R* as determined by the following equation:~~

~~*R* = 23.1 (1 + *D* / L_o)~~ ~~**(Equation 16-24)**~~

~~where:~~

~~*A* = Area of floor supported by the member, square feet (m²).~~

~~*D* = Dead load per square foot (m²) of area supported.~~

~~L_o = Unreduced live load per square foot (m²) of area supported.~~

~~*R* = Reduction in percent.~~

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. Most of the provisions being deleted were the product of a legacy code. They have not appeared in recent editions of ASCE 7 nor are they likely to appear in the 2010 edition.
The provisions of IBC Sections 1607.9.1 for the basic procedures to reduce live load on floors and roofs serving an occupancy function are based on Section 4.8 of ASCE 7 (Reduction in Live Loads). The ASCE 7 Committee develops and maintains the provisions of Section 4.8 and has not chosen not to add alternative procedures for the reduction of live loads. The differences between the basic and alternative procedures are such that structural performance of buildings and other structures can be expected to differ depending on which set of procedures are utilized in the design. The ICC code development process does not provide the means for adequate deliberation over the merits of each set of procedures in providing adequate levels of public safety. This deliberation should be done by the ASCE 7 Committee.

A separate proposal further aligns IBC Section 1607 on roof live load reduction with Chapter 4 of ASCE 7-10. That proposal deletes Section 1607.9.1.5 and the last sentence of Section 1607.9 and revises the second sentence of Section 1607.9. The deletions and revisions are not repeated in this proposal but, should both proposals be approved by the membership, the proponent intends that the last sentence of Section 1607.9 be deleted, not revised as indicated in this proposal.

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

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

FILENAME: Brazil-S5-1607.9

S77–09/10

1607.9, 1607.9.1.5, 1607.11.2, 1607.11.2.1, 1607.11.2.2, Table 1607.1

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

1. Revise as follows:

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

~~**1607.9.1.5 Roofs members.** Live loads of 100 psf (4.79 kN/m²) or less shall not be reduced for roof members except as specified in Section 1607.11.2.~~

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

1607.11.2.1 ~~Flat, pitched and curved~~ Ordinary roofs, awnings and canopies. Ordinary flat, pitched and curved roofs, and awnings and canopies other than of fabric construction supported by lightweight rigid skeleton structures, are permitted to be designed for a reduced uniformly distributed roof live load, L_r, as specified in the following equations or other controlling combinations of loads in Section 1605, whichever produces the greater load.

In structures such as greenhouses, where special scaffolding is used as a work surface for workers and materials during maintenance and repair operations, a lower roof load than specified in the following equations shall not be used unless approved by the building official. Such structures shall be designed for a minimum roof live load of 12 psf (0.58 kN/m^2).

L_r = $L_o R_1 R_2$ **(Equation 16-25)**

where: $12 \le L_r \le 20$

For SI: $L_r = L_o R_1 R_2$

where: $0.58 \le L_r \le 0.96$

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

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

R_1 = 1 for $A_t \le$ 200 square feet (18.58 m^2) **(Equation 16-26)**

R_1 = $1.2 - 0.001 A_t$ for 200 square feet < A_t < 600 square feet **(Equation 16-27)**
For SI: $1.2 - 0.011 A_t$ for 18.58 square meters < A_t < 55.74 square meters

R_1 = 0.6 for $A_t \ge$ 600 square feet (55.74 m^2) **(Equation 16-28)**

where:

A_t = Tributary area (span length multiplied by effective width) in square feet (m^2) supported by any structural member, and
R_2 = 1 for $F \le 4$ **(Equation 16-29)**
R_2 = $1.2 - 0.05 F$ for $4 < F < 12$ **(Equation 16-30)**
R_2 = 0.6 for $F \ge 12$ **(Equation 16-31)**

where:

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

1607.11.2.2 ~~Special-purpose roofs~~ Roof areas serving occupancy functions. Areas of roofs ~~used for promenade purposes,~~ that serve occupancy functions, such as roof gardens, or for assembly ~~purposes~~ or other ~~special~~ similar purposes, and marquees~~, shall be designed for a minimum live load, L_o, as specified in Table 1607.1. Such live loads~~ are permitted to ~~be~~ have their uniformly distributed live loads reduced in accordance with Section 1607.9. Live loads of 100 psf or more at areas of roofs classified as Group A occupancies shall not be reduced.

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
29. Roofs:		
All roof surfaces subject to maintenance workers		300
Awnings and canopies:		
Fabric construction supported by a lightweight rigid skeleton structure	5 nonreduceable	
All other construction	20	
Ordinary flat, pitched, and curved roofs (not serving an occupancy function)	20	
Primary roof members, exposed to a work floor:		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages		2,000
All other occupancies		300
~~Roofs used for other special purposes~~	~~Note l~~	~~Note l~~
~~Roofs used for promenade purposes~~	~~60~~	
~~Roofs used for roof gardens or assembly purposes~~	~~100~~	
Roofs serving an occupancy function:		
Roof gardens	60	
Assembly areas	100	
All other similar areas	Note l	Note l

(Portions of Table not show, remain unchanged)

(No change to footnotes a through f)

g. Where snow loads occur that are in excess of the design conditions, the structure shall be designed to support the loads due to the increased loads caused by drift buildup or a greater snow design determined by the building official (see Section 1608). ~~For special-purpose roofs, see Section 1607.11.2.2.~~

(No change to footnotes h through k)

l. ~~Roofs used for other special purposes~~ Areas of roofs serving on occupancy function, other than roof gardens and assembly areas, shall be designed for appropriate loads as approved by the building official.

2. Re-organize Table 1607.1 as follows:

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
29. Roofs:		
Ordinary flat, pitched, and curved roofs (not serving an occupancy function)	20	
Awnings and canopies:		
Fabric construction supported by a lightweight rigid skeleton structure	5 nonreducible	
All other construction	20	
~~Roofs used for other special purposes~~	~~Note l~~	~~Note l~~
~~Roofs used for promenade purposes~~	~~60~~	
~~Roofs used for roof gardens or assembly purposes~~	~~100~~	
Roofs serving an occupancy function:		
Roof gardens	60	
Assembly areas	100	
All other similar areas	Note l	Note l

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
Primary roof members, exposed to a work floor:		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages		2,000
All other occupancies		300
All roof surfaces subject to maintenance workers		300

(Portions of Table not show, remain unchanged)

Reason This proposal was prepared in conjunction with a proposal to editorially correlate IBC Section 1607 with the provisions of Chapter 4 of ASCE 7-10 related to floor live load reduction. This proposal focuses on editorial correlation for roof live load reduction and is intended to further align IBC Section 1607 with Chapter 4 of ASCE 7-10 without any overlapping or conflicting changes between the two proposals. The changes in this proposal are seen as largely editorial.

The items under “roofs” in Table 1607.1 are reorganized to align them with Section 1607.11.2 on the reduction of live loads at roofs. Section 1607.11.2 permits reductions in uniform live loads at roofs and marquees “in accordance with Section 1607.11.2.1 or 1607.11.2.2.” These sections, in turn, refer to items under “roofs” in Table 1607.1 except for marquees, which are separately listed in the table. The reorganization of the items under “roofs” is intended to reduce confusion over the applicability of roof live load reduction at “ordinary flat, pitched and curved roofs” in Section 1607.11.2.1, which applies to roofs that do not serve an occupancy function but are susceptible to loads from maintenance workers, and at roofs that serve an occupancy function in Section 1607.11.2.2, which are the structural equivalent of floors. Section 1607.11.2.1 also applies to awnings and canopies and the title of the section is changed accordingly. Table 1607.1 is reorganized so that it aligns with these sections.

The changes to Section 1607.9 and the deletion of Section 1607.9.1.5 eliminate superfluous text. Section 1607.9 provides the charging text for reduction of uniformly distributed live loads at floors. The changes retain the reference to Section 1607.11.2 on roof live load reduction but delete the text referring to special purpose roofs in favor of the charging text in Section 1607.11.2.

Section 1607.11.2.2 is changed to align it with the corresponding provisions in Section 4.8.2 of ASCE 7-10. Section 4.9.2 of ASCE 7-05 currently specifies roofs “that have an occupancy function” but the title of the section is “special purpose roofs.” The proponent is requesting the title be changed to “roof areas serving an occupancy function” in Section 4.8.2 of ASCE 7-10 and IBC Section 1607.11.2.2 is changed accordingly. Section 4.8.2 of ASCE 7-10 specifies roof gardens and areas used for “assembly or other similar purposes” as examples of roof areas that serve occupancy functions. Section 4.9.2 of ASCE 7-05 is similar. Table 4-1 of ASCE 7-05, however, lists roofs used for promenade purposes along with roofs used for roof gardens or for assembly or other special purposes. This listing of roofs used for promenade purposes is deleted in Table 4-1 of ASCE 7-10 and IBC Table 1607.1 is changed accordingly.

The proponent is requesting “other special purposes” be changed to “other similar purposes” in Section 4.8.2 and Table 4-1 of ASCE 7-10 in conjunction with the requested change in the title of Section 4.8.2 from “special purpose roofs” to “roof areas serving an occupancy function.” IBC Section 1607.11.2.2 and Table 1607.1 are changed accordingly. Note that the uniform live load at roofs used for other special purposes in Table 4-1 of ASCE 7-05 and 2009 IBC Table 1607.1, and at other similar areas of roofs serving an occupancy function in IBC Table 1607.1 of this proposal, is not specified in favor of a footnote specifying appropriate loads as approved by the authority having jurisdiction (ASCE 7) or building official (IBC) and is not affected by this proposal.

Footnotes (g) and (l) to Table 1607.1 are revised for consistency with the changes to Section 1607.11.2.2 above. The deletion in Footnote (g) also eliminates a superfluous cross-reference.

A separate proposal correlates the IBC with changes to the provisions of ASCE 7-10 where reduction of live loads at floors and occupied roofs is restricted or prohibited. That proposal adds Footnote (m) to “100 psf” at “roofs used for assembly purposes” in the item for roofs in Table 1607.1. The addition of the footnote is not repeated in this proposal but, should this proposal and the proposal adding the footnote be approved by the membership, the proponent intends that Footnote (m) be specified with “100 psf” at “assembly areas” under “roofs serving an occupancy function” in the item for roofs in Table 1607.1.

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

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

ICCFILENAME: Brazil-S14-1607.9

S78–09/10
1607.9.2

Proponent: Matt Rescorla, PE, Cubic Designs, Inc., representing self

Revise as follows:

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

1. A reduction shall not be permitted in Group A occupancies.
2. A reduction shall not be permitted where the live load exceeds 100 psf (4.79 kN/m^2) except that the design live load for columns ~~members supporting 2 or more floors~~ is permitted to be reduced by a maximum of 20 percent <u>where shown by the registered design professional that a rational approach has been used and that such reductions are warranted</u>.

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

3. A reduction shall not be permitted in passenger vehicle parking garages except that the live loads for members supporting two or more floors are permitted to be reduced by a maximum of 20 percent.
4. For live loads not exceeding 100 psf (4.79 kN/m^2), the design live load for any structural member supporting 150 square feet (13.94 m^2) or more is permitted to be reduced in accordance with Equation 16-23.
5. For one-way slabs, the area, *A*, for use in Equation 16-23 shall not exceed the product of the slab span and a width normal to the span of 0.5 times the slab span.

$R = 0.08(A - 150)$ **(Equation 16-23)**

For SI: $R = 0.861(A - 13.94)$

Such reduction shall not exceed the smallest of:

1. 40 percent for horizontal members;
2. 60 percent for vertical members; or
3. *R* as determined by the following equation.

$R = 23.1(1 + D/L_o)$ **(Equation 16-24)**

where:

A = Area of floor supported by the member, square feet (m2).
D = Dead load per square foot (m^2) of area supported.
L_o = Unreduced live load per square foot (m^2) of area supported.
R = Reduction in percent.

Reason: Typical minimum uniformly distributed live loads for Mezzanine design are dictated by Table 1607.1, under the loads of "Light Storage" at 125 psf. Leeway should be given to professional judgment in regards to live load reduction. Twenty years of professional experience has shown, the standard mezzanine (used for storage and designed for 125 psf) experiences a load ranging from 75-90 psf. A mezzanine used for production and experiencing loads from permanent equipment greater than or equal to 125 psf would not receive a reduction to its load.

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

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

ICCFILENAME: RESCORIA-S1-1607.9.2

S79–09/10
1602.1, Table 1607.1, 1607.11.3

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

Revise as follows:

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

DEAD LOADS. The weight of materials of construction incorporated into the building, including but not limited to walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, cladding, landscaping materials and other similarly incorporated architectural and structural items, and the weight of fixed service equipment, such as cranes, plumbing stacks and risers, electrical feeders, heating, ventilating and air-conditioning systems and automatic sprinkler systems.

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

OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
29. Roofs		
All roof surfaces subject to maintenance workers		300
Awnings and canopies		
Fabric construction supported by a lightweight rigid skeleton structure	5 nonreduceable	
All other construction	20	
Ordinary flat, pitched, and curved roofs	20	
Primary roof members, exposed to a work floor		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages		2,000
All other occupancies		300
Roofs used for other special purposes	Note l	Note l
Roofs used for promenade purposes	60	
Roofs used for roof gardens	60	
Roofs used for ~~roof gardens or~~ assembly purposes	100	

(Portions of Table not shown, remain unchanged. No changes to footnotes)

1607.11.3 Landscaped roofs. Where ~~roofs~~ areas of the roof are to be landscaped, the landscaped areas shall be designed to resist a uniform design live load ~~in the landscaped area shall be~~ of 20 psf (0.958 kN/m^2). The weight of the landscaping materials shall be considered as dead load and shall be computed on the basis of saturation of the soil. Adjacent areas without landscaping that are occupiable shall be designed to resist the live loads specified in Table 1607.1 for roof gardens.

Reason: The purpose for this proposal is to revise the IBC for consistency with an ASCE 7 proposal being considered by the Live Load Subcommittee. The proposed text will better define the design loads appropriate for roofs with landscaping. Such roofs typically consist of landscaped areas, which are not occupiable except by maintenance workers, and occupiable areas adjacent to the landscaped areas. Table 1607.1 does not currently distinguish between them except to specify a live load of 100 psf for roof gardens. A live load of 100 psf is not justified for landscaped areas. It is excessive for occupiable areas except where they are used for assembly purposes.

The proposal distinguishes between landscaped areas and adjacent areas without landscaping that are occupiable. A live load of 20 psf, the same as the unreduced live load for ordinary, flat, pitched and curved roofs, is currently specified in Section 1607.11.3 for roofs that are to be landscaped. The proposal revises this to areas of the roof that are to be landscaped. An entire roof devoted to landscaping should not be necessary before the structural demands due to landscaping areas are required to be considered. Section 1607.11.3 currently specifies the saturated weight of the landscaping materials as dead load. The proposal adds landscaping materials to the definition of "dead load" in Section 1602.1 for consistency.

For adjacent areas without landscaping that are occupiable, the proposal adds to Section 1607.11.3 the requirement that these areas be designed to resist the live loads specified in Table 1607.1 for roof gardens. The live load for roof gardens in Table 1607.1 is reduced to 60 psf. The listing in Table 1607.1 of 100 psf for assembly areas is retained to account for areas without landscaping that are used for assembly purposes.

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

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

ICCFILENAME: Brazil-S28-T1607.1

S80–09/10

Table 1607.1, 1607.11.2, 1607.11.2.2, 1607.11.3

Proponent: Traxler, City of Seattle representing Seattle Dept of Planning & Development

Revise as follows:

TABLE 1607.1
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, L_o, AND
MINIMUM CONCENTRATED LIVE LOADS~~g~~
***(**No change to table)*

(No change to footnotes a through k)

l. Roofs used for other special purposes shall be designed for appropriate loads as approved by the building official. Unoccupied landscaped areas of roofs shall be designed in accordance with Section 1607.11.3.

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

1607.11.3. ~~**1607.11.2.2**~~ **Special-purpose roofs.** Roofs used for promenade purposes, roof gardens, assembly purposes or other special purposes, and marquees, shall be designed for a minimum live load, L_o, as specified in Table 1607.1. Such live loads are permitted to be reduced in accordance with Section 1607.9. Live loads of 100 psf (4.79 kN/m^2) or more at areas of roofs classified as Group A occupancies shall not be reduced.

1607.11.3.1 Landscaped roofs. ~~Where roofs are to be landscaped, the~~ The uniform design live load in ~~the~~ unoccupied landscaped areas on roofs shall be 20 psf (0.958 kN/m^2). The weight of ~~the~~ all landscaping materials shall be considered as dead load and shall be computed on the basis of saturation of the soil.

Reason: Roofs used for roof gardens, commonly referred to as "green roofs", have generated many questions as to the appropriate load requirements. There has been confusion over what live loads are to be used for the landscaped versus non-landscaped portions of the roofs, and about how the "roof garden" provisions of Table 1607.1 row 29 coordinate with Section 1607.11.3. We are proposing an amendment that separates "special purpose roofs", including roof gardens, from the section on reduction of roof loads, and clarifying that the provisions for landscaped roofs only applies to unoccupied areas; occupied areas should be designed for loads appropriate to the particular occupancy. Note that the last sentence of what becomes Section 1607.11.3.1 in this proposal could be relocated to Section 1606.

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

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

ICCFILENAME: Traxler-S2-T1607.1

S81–09/10
1607.13

Proponent: Stephen Kerr, PE, SE, representing self

Revise as follows:

1607.13 Interior walls and partitions. Interior walls and partitions that exceed 6 feet (1829 mm) in height, including their finish materials, shall have adequate strength to resist the loads to which they are subjected but not less than a horizontal load of 5 psf (0.240 kN/m^2). For the purposes of calculating deflection, the loading of this section shall be treated as a wind load in accordance with Table 1604.3.

> **Exception:** Fabric partitions complying with Section 1607.13.1 shall not be required to resist the minimum horizontal load of 5 psf (0.240 kN/m^2).

Reason: Currently, Table 1604.3 does not have deflection limits for Live Loads on Interior walls. The 5.0 psf requirement in section 1607.13 is classified as a live load and would not require a deflection check. Under the legacy Uniform Building Code this load was treated as an "other load" and was required to meet the defection limits identical to those in IBC Table 1604.3. To avoid confusion for walls, and to require deflection checks on interior walls, the proposed code change is necessary.

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

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

ICCFILENAME: KERR-S3-1607.13

S82–09/10
1607.14 (New)

Proponent: Dennis Richardson PE, dbr group inc., representing self

Add new text as follows:

1607.14 Fire-resistance-rated exterior walls. Fire-resistance rated exterior walls required by Section 705.6 to have sufficient structural stability such that it will remain in place for the duration of time indicated by the required fire-resistance rating shall have adequate strength to resist the loads to which they are subjected but not less than a horizontal load of 10 psf (0.48 kN/m^2) applied perpendicular to the plane of the wall.

Reason: Design criteria do not exist for the current requirement found in Section 705.6 to justify the performance requirement for the out of plane structural support of fire-resistance rated exterior walls. A code change proposal has been submitted to provide proscriptive exceptions to the current performance related language found in the 2009 IBC. There needs to be some design criteria when the exceptions do not apply. Because of the uncertainty and dynamic nature of fire conditions, it is felt the interior partition wall load of 5 psf would be too low for the design of free standing exterior fire resistance rated walls expected to perform adequately during fire conditions. Because fires do have associated impact loads and air pressure associated with collapsing elements, a minimum out of plane loading of 10 psf is proposed as a lower bound when exterior fire resistance rated walls are required to be justified.

Cost Impact: Since this code change requires a minimum level of strength where the current code states vague performance objectives, this proposed change could increase construction cost.

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

ICCFILENAME: Richardson-S1-1607.14

S83–09/10

1602.1, 1608.3 (New), 1611.2

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

1. Add new definition as follows:

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

<u>**SUSCEPTIBLE BAY**. A roof or portion thereof with (1) a slope less than 1/4-inch per foot (0.0208 rad), or (2) where water will be impounded upon it, in whole or in part, and the secondary drainage system is functional but the primary drainage system is not functional. A roof surface with a slope of 1/4-inch per foot (0.0208 rad) or greater towards points of free drainage is not a susceptible bay.</u>

2. Add new text as follows:

<u>**1608.3 Ponding instability.** Susceptible bays of roofs shall be evaluated for ponding instability in accordance with Section 7.11 of ASCE 7.</u>

3. Revise as follows:

1611.2 Ponding instability. ~~For~~ <u>Susceptible bays of</u> roofs ~~with a slope less than 1/4 inch per foot [1.19 degrees (0.0208 rad)], the design calculations~~ shall ~~include verification of~~ <u>be investigated by structural analysis to ensure that they possess</u> adequate stiffness to preclude progressive deflection in accordance with Section 8.4 of ASCE 7.

Reason: The purpose for the proposal is to correlate the IBC with the 2010 edition of ASCE 7. The need for correlation is due to an ASCE 7 proposal on ponding instability, which was approved by the Snow/Rain Subcommittee and is being balloted by the Main Committee (Item #3 of the ASCE 7 Third Main Committee Ballot on Snow and Rain Provisions). It is expected that the Main Committee will approve the proposal.

Susceptible bays of roofs are required to meet the technical provisions of ASCE 7-10 for precluding progressive deflection. A "susceptible bay" is defined in Section 8.4 of ASCE 7-10 and this definition is being added to the IBC. Having a definition of "susceptible bay" in the IBC will provide a technical basis for determining which bays of a roof are susceptible bays and, thus, are required to meet the technical provisions of ASCE 7-10 for them. All bays of roofs other than susceptible bays are not required to meet these technical provisions.

Without the definition, the IBC will be without effective charging text. IBC Sections 1608.3 and 1611.2 rely on the determination of which bays are susceptible bays in order to determine the need to comply with the applicable provisions of ASCE 7. That determination is not possible unless a definition of "susceptible bay" is included in the IBC.

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

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

ICCFILENAME: Brazil-S37-1602.1

S84–09/10
1602, 1609.1.2.2-1710.3

Proponent: Jim Rossberg, SEI of ASCE, representing self

1. Add new text as follows:

SECTION 1602
DEFINITIONS AND NOTATIONS

NOTATIONS.

V_{asd} = nominal design wind speed (3-sec gust), miles per hour (mph) (km/hr) where applicable.

V_{ult} = ultimate design wind speeds (3-sec gust), miles per hour (mph) (km/hr) determined from Figures 1609A, 1609B, or 1609C or ASCE 7.

2. Revise as follows:

1603.1 General. Construction documents shall show the size, section and relative locations of structural members with floor levels, column centers and offsets dimensioned. The design loads and other information pertinent to the structural design required by Sections 1603.1.1 through 1603.1.9 shall be indicated on the *construction documents*.

> **Exception:** *Construction documents* for buildings constructed in accordance with the *conventional light-frame construction* provisions of Section 2308 shall indicate the following structural design information:
>
> 1. Floor and roof live loads.
> 2. Ground snow load, P_g.
> 3. ~~Basic~~ Ultimate design wind speed, V_{ult} , (3-second gust), miles per hour (mph) (km/hr) and nominal design wind speed, V_{asd}, as determined in accordance with Section 1609.3.1 and wind exposure.
> 4. *Seismic design category* and *site class*.
> 5. Flood design data, if located in *flood hazard areas* established in Section 1612.3.
> 6. Design load-bearing values of soils.

1603.1.4 Wind design data. The following information related to wind loads shall be shown, regardless of whether wind loads govern the design of the lateral-force-resisting system of the building:

1. ~~Basic~~ Ultimate design wind speed, V_{ult}, (3-second gust), miles per hour (km/hr) and nominal design wind speed, V_{asd} , as determined in accordance with Section 1609.3.1.
2. Wind importance factor, *I*, and *occupancy category*.
3. Wind exposure. Where more than one wind exposure is utilized, the wind exposure and applicable wind direction shall be indicated.
4. The applicable internal pressure coefficient.
5. Components and cladding. The design wind pressures in terms of psf (kN/m^2) to be used for the design of exterior component and cladding materials not specifically designed by the *registered design professional*

TABLE 1604.3
DEFLECTION LIMITS[a,b,c,h,i]
(No change to table)

(No change to footnotes a-e)
f. The wind load is permitted to be taken as 0.42 ~~0.7~~ times the "component and cladding" loads for the purpose of determining deflection limits herein.
(No change to footnotes g-i)

1605.2.1 Basic load combinations. Where strength design or *load and resistance factor design* is used, structures and portions thereof shall resist the most critical effects from the following combinations of factored loads:

$1.4\ (D+F)$ **(Equation 16-1)**

$1.2(D + F + T) + 1.6(L + H) + 0.5\ (L_r \text{ or } S \text{ or } R)$ **(Equation 16-2)**

$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (f_1 L \text{ or } \cancel{0.8W}\ \underline{0.5W})$ **(Equation 16-3)**

$1.2D + \cancel{1.6}\ \underline{1.0}W + f_1 L + 0.5(L_r \text{ or } S \text{ or } R)$ **(Equation 16-4)**

$1.2D + 1.0E + f_1 L + f_2 S$ **(Equation 16-5)**

$0.9D + \cancel{1.6}\ \underline{1.0}W + 1.6H$ **(Equation 16-6)**

$0.9D + 1.0E + 1.6H$ **(Equation 16-7)**

where:

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

f_2 = 0.7 for roof configurations (such as saw tooth) that do not shed snow off the structure, and
= 0.2 for other roof configurations.

Exception: Where other factored load combinations are specifically required by the provisions of this code, such combinations shall take precedence.

1605.3.1 Basic load combinations. Where *allowable stress design* (working stress design), as permitted by this code, is used, structures and portions thereof shall resist the most critical effects resulting from the following combinations of loads:

$D + F$ **(Equation 16-8)**

$D + H + F + L + T$ **(Equation 16-9)**

$D + H + F + (L_r \text{ or } S \text{ or } R)$ **(Equation 16-10)**

$D + H + F + 0.75(L + T) + 0.75\,(L_r \text{ or } S \text{ or } R)$ **(Equation 16-11)**

$D + H + F + (\underline{0.6}W \text{ or } 0.7E)$ **(Equation 16-12)**

$D + H + F + 0.75(\underline{0.6}W \text{ or } 0.7E) + 0.75L + 0.75\,(L_r \text{ or } S \text{ or } R)$ **(Equation 16-13)**

$0.6D + \underline{0.6}W + H$ **(Equation 16-14)**

$0.6D + 0.7E + H$ **(Equation 16-15)**

Exceptions:

1. Crane hook loads need not be combined with roof live load or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m^2) or less and roof live loads of 30 psf or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.

1605.3.2 Alternative basic load combinations. In lieu of the basic load combinations specified in Section 1605.3.1, structures and portions thereof shall be permitted to be designed for the most critical effects resulting from the following combinations. When using these alternative basic load combinations that include wind or seismic loads, allowable stresses are permitted to be increased or load combinations reduced where permitted by the material chapter of this code or the referenced standards. For load combinations that include the counteracting effects of dead and wind loads, only two-thirds of the minimum dead load likely to be in place during a design wind event shall be used. Where wind loads are calculated in accordance with Chapters 26 through 31 ~~Chapter 6~~ of ASCE 7, the coefficient in the following equations shall be taken as 0.78 ~~1.3~~. For other wind loads, shall be taken as 1. When using these alternative load combinations to evaluate sliding, overturning and soil bearing at the soil-structure interface, the reduction of foundation overturning from Section 12.13.4 in ASCE 7 shall not be used. When using these alternative basic load combinations for proportioning foundations for loadings, which include seismic loads, the vertical seismic *load effect*, E_v, *i*n Equation 12.4-4 of ASCE 7 is permitted to be taken equal to zero.

$D + L + (L_r \text{ or } S \text{ or } R)$ **(Equation 16-16)**

$D + L + (\omega W)$ **(Equation 16-17)**

$D + L + \omega W + S/2$ **(Equation 16-18)**
$D + L + S + \omega W/2$ **(Equation 16-19)**

$D + L + S + E/1.4$ **(Equation 16-20)**

$0.9D + E/1.4$ **(Equation 16-21)**

Exceptions:

1. Crane hook loads need not be combined with roof live loads or with more than three-fourths of the snow load or one-half of the wind load.
2. Flat roof snow loads of 30 psf (1.44 kN/m^2) or less and roof live loads of 30 psf or less need not be combined with seismic loads. Where flat roof snow loads exceed 30 psf (1.44 kN/m^2), 20 percent shall be combined with seismic loads.

1609.1.1 Determination of wind loads. Wind loads on every building or structure shall be determined in accordance with Chapters 26 to 30 ~~6~~ of ASCE 7 ~~or provisions of the alternate all-heights method in Section 1609.6~~. The type of opening protection required, the ~~basic~~ ultimate design wind speed, V_{ult}, and the exposure category for a site is permitted to be determined in accordance with Section 1609 or ASCE 7. Wind shall be assumed to come from any horizontal direction and wind pressures shall be assumed to act normal to the surface considered.

Exceptions:

1. Subject to the limitations of Section 1609.1.1.1, the provisions of ICC 600 shall be permitted for applicable Group R-2 and R-3 buildings.
2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of the AF&PA WFCM.
3. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AISI S230.
4. Designs using NAAMM FP 1001.
5. Designs using TIA-222 for antenna-supporting structures and antennas.
6. Wind tunnel tests in accordance with Chapter 31 ~~Section 6.6~~ of ASCE 7~~, subject to the limitations in Section 1609.1.1.2~~.

The wind speeds in Figure 1609A, 1609B and 1609C are ultimate design wind speeds, V_{ult}, and shall be converted in accordance with Section 1609.3.1 to nominal design wind speeds, V_{asd}, when the provisions of the standards referenced in Exceptions 1 through 5 are used.

4. Delete without substitution as follows:

~~**1609.1.1.2 Wind tunnel test limitations.** The lower limit on pressures for main wind-force-resisting systems and components and cladding shall be in accordance with Sections 1609.1.1.2.1 and 1609.1.1.2.2.~~

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

~~**1609.1.1.2.2 Lower limits on components and cladding.** The design pressures for components and cladding on walls or roofs shall be selected as the greater of the wind tunnel test results or 80 percent of the pressure obtained for Zone 4 for walls and Zone 1 for roofs as determined in Section 6.5 of ASCE 7, unless specific testing is performed that demonstrates it is the aerodynamic coefficient of the building, rather than shielding from nearby structures, that is~~

~~responsible for the lower values. Alternatively, limited tests at a few wind directions without specific adjacent buildings, but in the presence of an appropriate upwind roughness, shall be permitted to be used to demonstrate that the lower pressures are due to the shape of the building and not to shielding.~~

5. Revise as follows:

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

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

Exceptions:

1. Wood structural panels with a minimum thickness of $^7/_{16}$ inch (11.1 mm) and maximum panel span of 8 feet (2438 mm) shall be permitted for opening protection in one- and two-story buildings classified as Group R-3 or R-4 occupancy. Panels shall be precut so that they shall be attached to the framing surrounding the opening containing the product with the glazed opening. Panels shall be predrilled as required for the anchorage method and shall be secured with the attachment hardware provided. Attachments shall be designed to resist the components and cladding loads determined in accordance with the provisions of ASCE 7, with corrosion-resistant attachment hardware provided and anchors permanently installed on the building. Attachment in accordance with Table 1609.1.2 with corrosion-resistant attachment hardware provided and anchors permanently installed on the building is permitted for buildings with a mean roof height of 45 feet (13 716 mm) or less where V_{asd} determined in accordance with Section 1609.3.1 ~~wind speeds do~~ does not exceed 140 mph (63 m/s).
2. Glazing in *Occupancy Category* I buildings as defined in Section 1604.5, including greenhouses that are occupied for growing plants on a production or research basis, without public access shall be permitted to be unprotected.
3. Glazing in *Occupancy Category* II, III or IV buildings located over 60 feet (18 288 mm) above the ground and over 30 feet (9144 mm) above aggregate surface roofs located within 1,500 feet (458 m) of the building shall be permitted to be unprotected.

6. Add new text as follows:

1609.1.2.2. Modifications to ASTM E 1996. Section 6.2.2 of ASTM E 1996 shall be modified as follows:

6.2.2 Unless otherwise specified, select the wind zone based on the strength design wind speed, V_{ult}, as follows:
6.2.2.1 *Wind Zone 1* - 130 mph ≤ ultimate design wind speed, V_{ult} < 140 mph.
6.2.2.2 *Wind Zone 2* - 140 mph ≤ ultimate design wind speed, V_{ult} < 150 mph at greater than 1.6 km (one mile) from the coastline. The coastline shall be measured from the mean high water mark.
6.2.2.3 *Wind Zone 3* - ultimate design wind speed, V_{ult} ≥ 150 mph, or the ultimate design wind speed, V_{ult} ≥ 140 mph and within 1.6 km (one mile) of the coastline. The coastline shall be measured from the mean high water mark.

7. Revise as follows:

1609.2 Definitions. The following words and terms shall, for the purposes of Section 1609, have the meanings shown herein.

HURRICANE-PRONE REGIONS. Areas vulnerable to hurricanes defined as:

1. The U. S. Atlantic Ocean and Gulf of Mexico coasts where the ~~basic~~ ultimate design wind speed, V_{ult}, for Occupancy Category II buildings is greater than 115 ~~90~~ mph (40 m/s) and
2. Hawaii, Puerto Rico, Guam, Virgin Islands and American Samoa.

WIND-BORNE DEBRIS REGION. Areas within ~~Portions of~~ *hurricane- prone regions* located: ~~that are~~

1. W~~w~~ithin 1 mile (1.61 km) of the coastal mean high water line where the ~~basic~~ ultimate design wind speed V_{ult} is 130 ~~110 mph~~ (48 m/s) or greater; or
2. In areas ~~portions of *hurricane-prone regions*~~ where the ~~basic~~ ultimate design wind speed is V_{ult} 140 ~~120~~ mph (53 m/s) or greater~~; or Hawaii.~~

For Occupancy Category II buildings and structures and Occupancy Category III buildings and structures, except health care facilities, the windborne debris region shall be based on Figure 1609b. For Occupancy Category IV buildings and structures and Occupancy Category III health care facilities, the windborne debris region shall be based on Figure 1609c.

8. Add new definitions as follows:

1609.2 Definitions.

WIND SPEED, V_{ult}. Ultimate design wind speeds.

WIND SPEED, V_{asd}. Nominal design wind speeds.

9. Revise as follows:

1609.3 Basic wind speed. The ~~basic~~ ultimate design wind speed V_{ult}, in mph, for the determination of the wind loads shall be determined by ~~Figure 1609~~ Figures 1609A, 1609B and 1609C. The ultimate design wind speed, V_{ult}, for use in the design of Occupancy Category II buildings and structures shall be obtained from Figure 1609A. The ultimate design wind speed, V_{ult}, for use in the design of Occupancy Category III and IV buildings and structures shall be obtained from Figure 1609B. The ultimate design wind speed, V_{ult}, for use in the design of Occupancy Category I buildings and structures shall be obtained from Figure 1609C. ~~Basic~~ The ultimate design wind speed, V_{ult}, for the special wind regions indicated, near mountainous terrain and near gorges shall be in accordance with local jurisdiction requirements. ~~Basic~~ The ultimate design wind speeds, V_{ult}, determined by the local jurisdiction shall be in accordance with Section 26.5.1 ~~6.5.4~~ of ASCE 7.

In nonhurricane-prone regions, when the ~~basic~~ ultimate design wind speed, V_{ult}, is estimated from regional climatic data, the ~~basic~~ ultimate design wind speed, V_{ult}, shall be ~~not less than the wind speed associated with an annual probability of 0.02 (50-year mean recurrence interval), and the estimate shall be adjusted for equivalence to a 3-second gust wind speed at 33 feet (10 m) above ground in Exposure Category C. The data analysis shall be performed~~ determined in accordance with Section 26.5.3 ~~6.5.4.2~~ of ASCE 7.

10. Delete and substitute as follows:

~~FIGURE 1609~~
~~BASIC WIND SPEED (3-SECOND GUST)~~

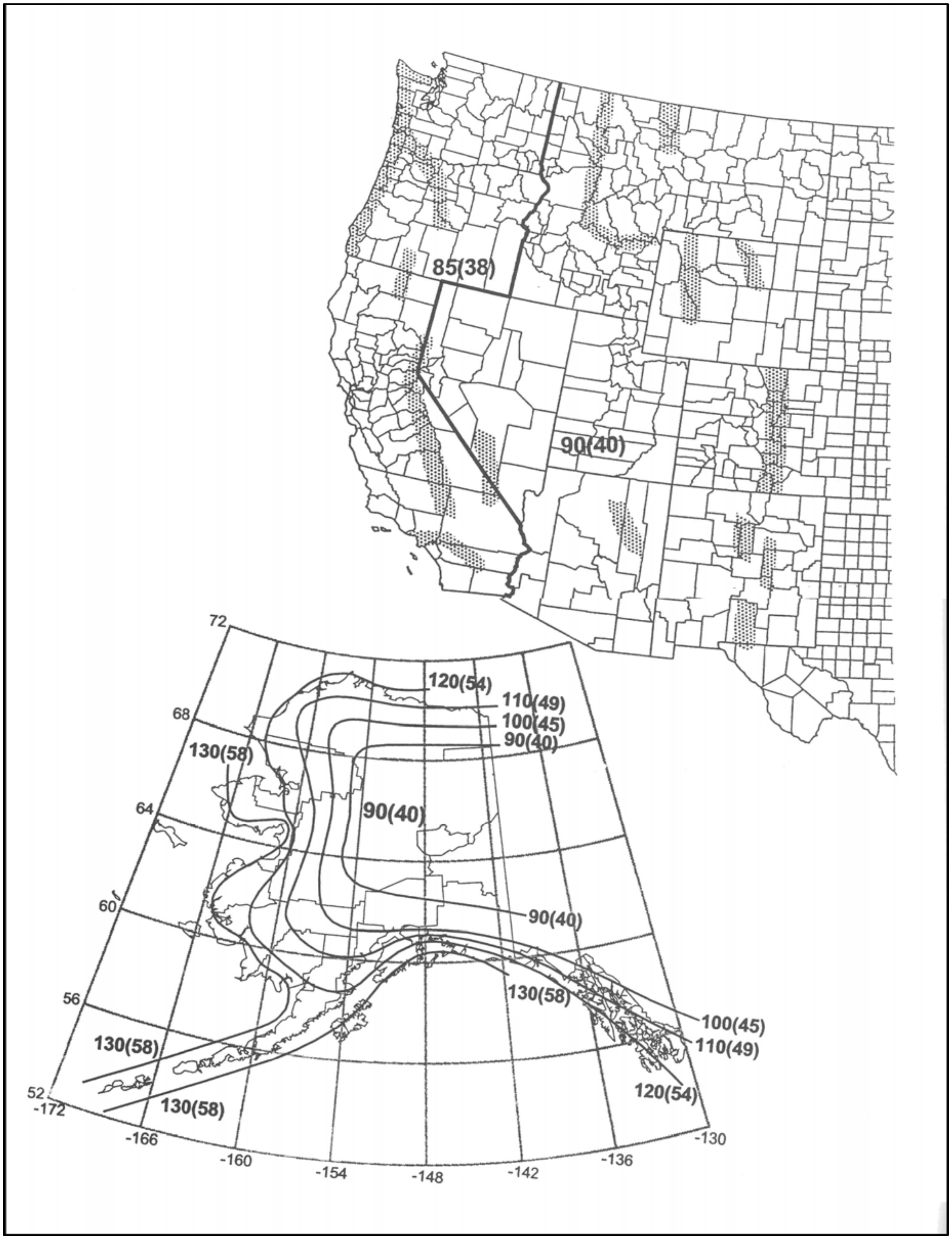
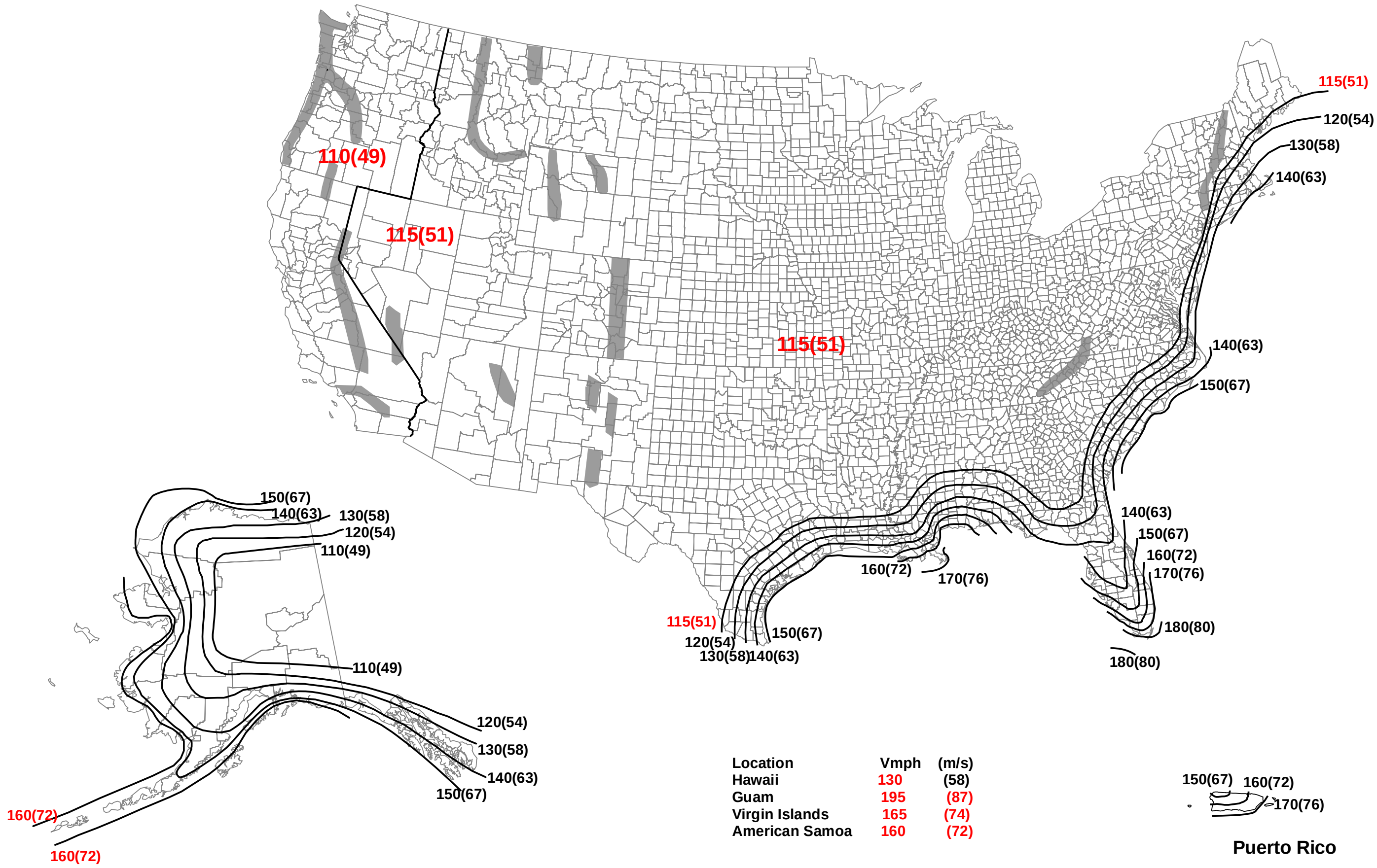


FIGURE 1609A
ULTIMATE DESIGN WIND SPEEDS, V_{ULT}, FOR OCCUPANCY CATEGORY II BUILDINGS AND OTHER STRUCTURES

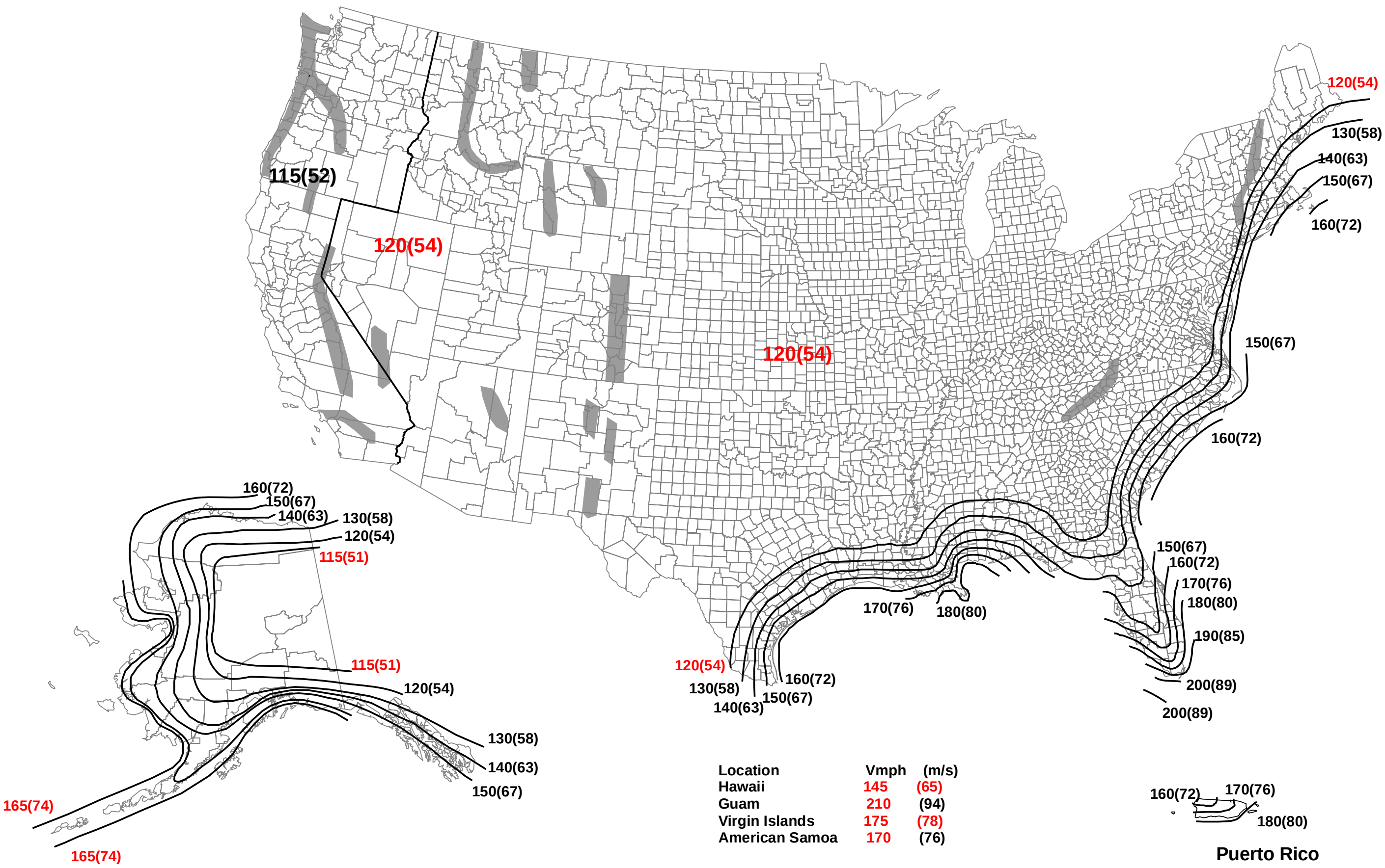


Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years.

Location	V mph	(m/s)
Hawaii - Special Wind Region Statewide	130	(58)
Guam	195	(87)
Virgin Islands	165	(74)
American Samoa	160	(72)

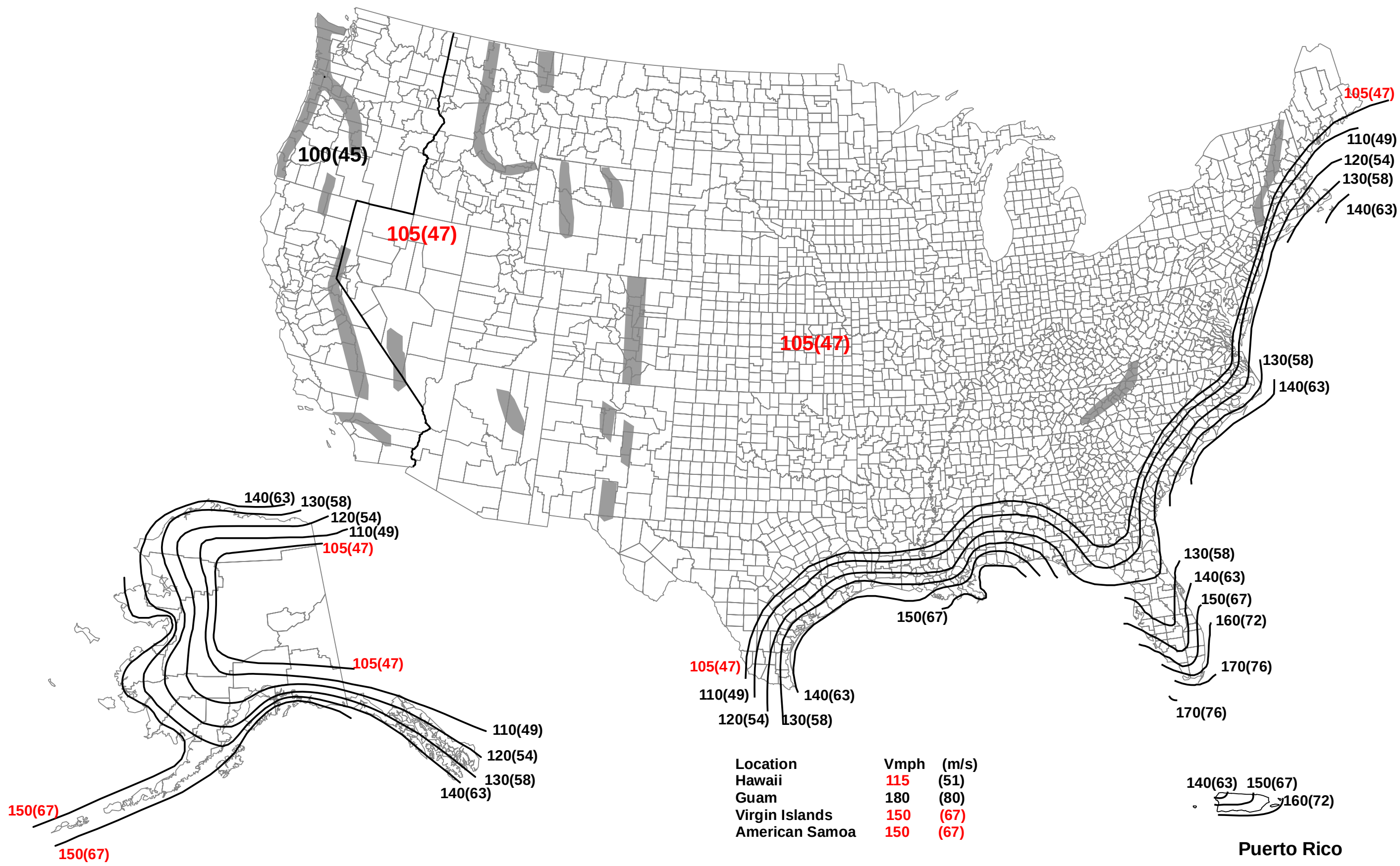
FIGURE 1609B
ULTIMATE DESIGN WIND SPEEDS, V_{ULT}, FOR OCCUPANCY CATEGORY III AND IV BUILDINGS AND OTHER STRUCTURES



Notes:
1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 3% probability of exceedance in 50 years.

Location	V mph	(m/s)
Hawaii - Special Wind Region Statewide	145	(65)
Guam	210	(94)
Virgin Islands	175	(78)
American Samoa	170	(76)

FIGURE 1609C
ULTIMATE DESIGN WIND SPEEDS, V_{ULT}, FOR OCCUPANCY CATEGORY I BUILDINGS AND OTHER STRUCTURES



Notes:
1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 15% probability of exceedance in 50 years.

Location	V mph	(m/s)
Hawaii - Special Wind Region Statewide	115	(51)
Guam	180	(80)
Virgin Islands	150	(67)
American Samoa	150	(67)

11. Revise as follows:

1609.3.1 Wind speed conversion. When required, the ~~3-second gust basic~~ ultimate design wind speeds of Figure 1609A, B and C shall be converted to nominal design wind speeds, V_{asd}, ~~fastest-mile wind speeds, V_{fm}~~, using Table 1609.3.1 or Equation 16-32.

$$\cancel{V_{fm} = \frac{V_{3S} - 10.5}{1.05}} \qquad \textbf{(Equation 16-32)}$$

~~where:~~

~~V_{3S} = 3-second gust basic wind speed from Figure 1609.~~

$V_{asd} = V_{ult}\sqrt{0.6}$

Where:
V_{asd} = nominal design wind speed applicable to methods specified in Exceptions 1 through 5 of Section 1609.1.1
V_{ult} = ultimate design wind speeds determined from Figures 1609A, 1609B, or 1609C

12. Delete and substitute as follows:

~~TABLE 1609.3.1~~
~~EQUIVALENT BASIC WIND SPEEDS~~~~[a,b,c]~~

~~V_{3S}~~	~~85~~	~~90~~	~~100~~	~~105~~	~~110~~	~~120~~	~~125~~	~~130~~	~~140~~	~~145~~	~~150~~	~~160~~	~~170~~
~~V_{fm}~~	~~71~~	~~76~~	~~85~~	~~90~~	~~95~~	~~104~~	~~109~~	~~114~~	~~123~~	~~128~~	~~133~~	~~142~~	~~152~~

~~For SI: 1 mile per hour = 0.44 m/s.~~
~~a. Linear interpolation is permitted.~~
~~b. *V3S* is the 3-second gust wind speed (mph).~~
~~c. *Vfm* is the fastest mile wind speed (mph).~~

TABLE 1609.3.1
WIND SPEED CONVERSIONS[a,b,c]

V_{ul}	100	110	120	130	140	150	160	170	180	190	200
V_{asd}	78	85	93	101	108	116	124	132	139	147	155

a. Linear interpolation is permitted
b. V_{asd} = nominal design wind speed applicable to methods specified in Exceptions 1 through 5 of Section 1609.1.1
c. V_{ult} = ultimate design wind speeds determined from Figures 1609A, 1609B, or 1609C

13. Revise as follows:

1609.4.2 Surface roughness categories. A ground surface roughness within each 45-degree (0.79 rad) sector shall be determined for a distance upwind of the site as defined in Section 1609.4.3 from the categories defined below, for the purpose of assigning an exposure category as defined in Section 1609.4.3.

Surface Roughness B. Urban and suburban areas, wooded areas or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.

Surface Roughness C. Open terrain with scattered obstructions having heights generally less than 30 feet (9144 mm). This category includes flat open country~~,~~ and grasslands~~, and all water surfaces in *hurricane-prone regions*~~.

Surface Roughness D. Flat, unobstructed areas and water surfaces ~~outside *hurricane-prone regions*~~. This category includes smooth mud flats, salt flats and unbroken ice.

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

Exposure B. For buildings with a mean roof height of less than or equal to 30 feet, Exposure B shall apply where the ground surface roughness ~~condition~~, as defined by Surface Roughness B, prevails in the upwind direction for a distance of at least 1,500 feet (457 m). For buildings with a mean roof height greater than 30 feet, Exposure B shall apply where Surface Roughness B prevails in the upwind direction for a distance of at least 2,600 feet (792 m) or 20 times the height of the building, whichever is greater.

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

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

Exposure D. Exposure D shall apply where the ground surface roughness, as defined by Surface Roughness D, prevails in the upwind direction for a distance of at least 5,000 feet (1524 m) or 20 times the height of the building, whichever is greater. Exposure D shall also apply where the ground surface roughness immediately upwind of the site is B or C, and the site is within a distance of 600 feet (183 m) or 20 times the building height, whichever is greater, from an exposure D condition as defined in the previous sentence. ~~Exposure D shall extend inland from the shoreline for a distance of 600 feet (183 m) or 20 times the height of the building, whichever is greater.~~

1609.5.3 Rigid tile. Wind loads on rigid tile roof coverings shall be determined in accordance with the following equation:

$$M_a = q_h C_L b L L_a [1.0 - GC_p] \qquad \textbf{(Equation 16-33)}$$

$$\text{For SI: } M_a = \frac{q_h C_L b L L_a \left[1.0 - GC_p\right]}{1{,}000}$$

where:

b = Exposed width, feet (mm) of the roof tile.

CL = Lift coefficient. The lift coefficient for concrete and clay tile shall be 0.2 or shall be determined by test in accordance with Section 1716.2.

GC_p = Roof pressure coefficient for each applicable roof zone determined from Chapter 30 ~~6~~ of ASCE 7. Roof coefficients shall not be adjusted for internal pressure.

L = Length, feet (mm) of the roof tile.

L_a = Moment arm, feet (mm) from the axis of rotation to the point of uplift on the roof tile. The point of uplift shall be taken at 0.76*L* from the head of the tile and the middle of the exposed width. For roof tiles with nails or screws (with or without a tail clip), the axis of rotation shall be taken as the head of the tile for direct deck application or as the top edge of the batten for battened applications. For roof tiles fastened only by a nail or screw along the side of the tile, the axis of rotation shall be determined by testing. For roof tiles installed with battens and fastened only by a clip near the tail of the tile, the moment arm shall be determined about the top edge of the batten with consideration given for the point of rotation of the tiles based on straight bond or broken bond and the tile profile.

Ma = Aerodynamic uplift moment, feet-pounds (N-mm) acting to raise the tail of the tile.

q_h = Wind velocity pressure, psf (kN/m^2) determined from Section 27.3.2 ~~6.5.10~~ of ASCE 7.

Concrete and clay roof tiles complying with the following limitations shall be designed to withstand the aerodynamic uplift moment as determined by this section.

1. The roof tiles shall be either loose laid on battens, mechanically fastened, mortar set or adhesive set.
2. The roof tiles shall be installed on solid sheathing which has been designed as components and cladding.
3. An underlayment shall be installed in accordance with Chapter 15.
4. The tile shall be single lapped interlocking with a minimum head lap of not less than 2 inches (51 mm).
5. The length of the tile shall be between 1.0 and 1.75 feet (305 mm and 533 mm).
6. The exposed width of the tile shall be between 0.67 and 1.25 feet (204 mm and 381 mm).
7. The maximum thickness of the tail of the tile shall not exceed 1.3 inches (33 mm).
8. Roof tiles using mortar set or adhesive set systems shall have at least two-thirds of the tile's area free of mortar or adhesive contact.

14. Delete without substitution:

~~**1609.6 Alternate all-heights method.** The alternate wind design provisions in this section are simplifications of the ASCE 7 Method 2—Analytical Procedure.~~

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

1. ~~The building or other structure is less than or equal to 75 feet (22 860 mm) in height with a height-to-leastwidth ratio of 4 or less, or the building or other structure has a fundamental frequency greater than or equal to 1 hertz.~~
2. ~~The building or other structure is not sensitive to dynamic effects.~~
3. ~~The building or other structure is not located on a site for which channeling effects or buffeting in the wake of upwind obstructions warrant special consideration.~~
4. ~~The building shall meet the requirements of a simple diaphragm building as defined in ASCE 7 Section 6.2, where wind loads are only transmitted to the main wind-force-resisting system (MWFRS) at the diaphragms.~~

5. ~~For open buildings, multispan gable roofs, stepped roofs, sawtooth roofs, domed roofs, roofs with slopes greater than 45 degrees (0.79 rad), solid free-standing walls and solid signs, and rooftop equipment, apply ASCE 7 provisions.~~

~~**1609.6.1.1 Modifications.** The following modifications shall be made to certain subsections in ASCE 7: in Section 1609.6.2, symbols and notations that are specific to this section are used in conjunction with the symbols and notations in ASCE 7 Section 6.3.~~

~~**1609.6.2 Symbols and notations.** Coefficients and variables used in the alternate all-heights method equations are as follows:~~

~~*Cnet*~~	~~=~~	~~Net-pressure coefficient based on *Kd* [(*G*) (*Cp*) – (*GCpi*)], in accordance with Table 1609.6.2(2).~~
~~*G*~~	~~=~~	~~Gust effect factor for rigid structures in accordance with ASCE 7 Section 6.5.8.1.~~
~~*Kd*~~	~~=~~	~~Wind directionality factor in accordance with ASCE 7 Table 6-4.~~
~~*Pnet*~~	~~=~~	~~Design wind pressure to be used in determination of wind loads on buildings or other structures or their components and cladding, in psf (kN/m2).~~
~~*qs*~~	~~=~~	~~Wind stagnation pressure in psf (kN/m2) in accordance with Table 1609.6.2(1).~~

~~**1609.6.3 Design equations.** When using the alternate all-heights method, theMWFRS,and components and cladding of every structure shall be designed to resist the effects of wind pressures on the building envelope in accordance with Equation 16-34.~~

~~*Pnet* = *qsKzCnet* [*IKzt*]~~ ~~**(Equation 16-34)**~~

~~Design wind forces for theMWFRSshall not be less than 10 psf (0.48 kN/m2) multiplied by the area of the structure projected on a plane normal to the assumed wind direction (see ASCE 7 Section 6.1.4 for criteria). Design net wind pressure for components and cladding shall not be less than 10 psf (0.48 kN/m2) acting in either direction normal to the surface.~~

~~**1609.6.4 Design procedure.** The MWFRS and the components and cladding of every building or other structure shall be designed for the pressures calculated using Equation 16-34.~~

~~**1609.6.4.1 Main wind-force-resisting systems.** The MWFRS shall be investigated for the torsional effects identified in ASCE 7 Figure 6-9.~~

~~**1609.6.4.2 Determination of *Kz* and *Kzt*.** Velocity pressure exposure coefficient, *Kz*, shall be determined in accordance with ASCE 7 Section 6.5.6.6 and the topographic factor, *Kzt*, shall be determined in accordance with ASCE 7 Section 6.5.7.~~

1. ~~For the windward side of a structure, *Kzt* and *Kz* shall be based on height *z*.~~
2. ~~For leeward and sidewalls, and for windward and leeward roofs, *Kzt* and *Kz* shall be based on mean roof height *h*.~~

~~**1609.6.4.3 Determination of net pressure coefficients, *Cnet*.** For the design of the MWFRS and for components and cla~~dding, the sum of the internal and external net pressure shall be based on the net pressure coefficient, *Cnet*.

1. ~~The pressure coefficient, *Cnet*, for walls and roofs shall be determined from Table 1609.6.2(2).~~
2. ~~Where *Cnet* has more than one value, the more severe wind load condition shall be used for design.~~

~~**1609.6.4.4 Application of wind pressures.** When using the alternate all-heights method, wind pressures shall be applied simultaneously on, and in a direction normal to, all building envelope wall and roof surfaces.~~

~~**1609.6.4.4.1 Components and cladding.**Wind pressure for each component or cladding element is applied as follows using *Cnet* values based on the effective wind area, *A*, contained within the zones in areas of discontinuity of width and/or length "a," "2a" or "4a" at: corners of roofs and walls; edge strips for ridges, rakes and eaves; or field areas onwalls or roofs as indicated in figures in tables in ASCE 7 as referenced in Table 1609.6.2(2) in accordance with the following:~~

1. ~~Calculated pressures at local discontinuities acting over specific edge strips or corner boundary areas.~~
2. ~~Include "field" (Zone 1, 2 or 4, as applicable) pressures applied to areas beyond the boundaries of the areas of discontinuity.~~

3. ~~Where applicable, the calculated pressures at discontinuities (Zones 2 or 3) shall be combined with design pressures that apply specifically on rakes or eave overhangs.~~

15. Revise as follows:

1405.14 Vinyl siding. Vinyl siding conforming to the requirements of this section and complying with ASTM D 3679 shall be permitted on exterior walls of buildings located in areas where ~~the~~ V_{asd} as determined in accordance with Section 1609.3.1 ~~basic wind speed specified in Chapter 16~~ does not exceed 100 miles per hour (45 m/s) and the *building height* is less than or equal to 40 feet (12 192 mm) in Exposure C. Where construction is located in areas where ~~the~~ V_{asd} as determined in accordance with Section 1609.3.1 ~~basic wind speed~~ exceeds 100 miles per hour (45 m/s), or building heights are in excess of 40 feet (12 192 mm), tests or calculations indicating compliance with Chapter 16 shall be submitted. Vinyl siding shall be secured to the building so as to provide weather protection for the exterior walls of the building.

1504.5 Edge securement for low-slope roofs. Low-slope membrane roof system metal edge securement, except gutters, shall be designed and installed for wind loads in accordance with Chapter 16 and tested for resistance in accordance with ANSI/SPRI ES-1, except ~~the basic~~ V_{ult} wind speed shall be determined from Figure 1609A, 1609B, or 1609C as applicable.

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

V_{asd} determined in accordance with Section 1609.3.1 ~~BASIC WIND SPEED FROM FIGURE 1609 (mph)b~~	MAXIMUM MEAN ROOF HEIGHT (ft)a,c		
	Exposure category		
	B	C	D
85	170	60	30
90	110	35	15
95	75	20	NP
100	55	15	NP
105	40	NP	NP
110	30	NP	NP
115	20	NP	NP
120	15	NP	NP
Greater than 120	NP	NP	NP

a. Mean roof height as defined in ASCE 7.
b. For intermediate values of V_{asd} ~~basic wind speed~~, the height associated with the next higher value of V_{asd} ~~wind speed~~ shall be used, or direct interpolation is permitted.
c. NP = gravel and stone not permitted for any roof height.

TABLE 1507.2.7.1(1)
CLASSIFICATION OF ASPHALT ROOF SHINGLES PER ASTM D 7158[a]

V_{asd} determined in accordance with Section 1609.3.1 ~~MAXIMUM BASIC WIND SPEED FROM FIGURE 1609~~	CLASSIFICATION REQUIREMENT

(Portions of Table not shown, remain unchanged)

TABLE 1507.2.7.1(2)
CLASSIFICATION OF ASPHALT SHINGLES PER ASTM D 3161

V_{asd} determined in accordance with Section 1609.3.1 ~~MAXIMUM BASIC WIND SPEED FROM FIGURE 1609~~	CLASSIFICATION REQUIREMENT

(Portions of Table now shown, remain unchanged)

1507.2.8.1 High wind attachment. Underlayment applied in areas subject to high winds (V_{asd} greater than 110 mph as determined in accordance with Section 1609.3.1 ~~in accordance with Figure 1609~~) shall be applied with corrosion-resistant fasteners in accordance with the manufacturer's instructions. Fasteners are to be applied along the overlap at a maximum spacing of 36 inches (914 mm) on center.

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

GENERAL — CLAY OR CONCRETE ROOF TILE				
Maximum V_{asd} determined in accordance with Section 1609.3.1 ~~basic wind speed (mph)~~	**Mean roof height (feet)**	**Roof slope up to < 3:12**	**Roof slope 3:12 and over**	
INTERLOCKING CLAY OR CONCRETE ROOF TILE WITH PROJECTING ANCHOR LUGS[d, e] (Installations on spaced/solid sheathing with battens or spaced sheathing)				
Maximum V_{asd} determined in accordance with Section 1609.3.1 ~~basic wind speed (mph)~~	**Mean roof height (feet)**	**Roof slope up to <5:12**	**Roof slope 5:12 < 12:12**	**Roof slope 12:12 and over**
INTERLOCKING CLAY OR CONCRETE ROOF TILE WITH PROJECTING ANCHOR LUGS (Installations on solid sheathing without battens)				
Maximum V_{asd} determined in accordance with Section 1609.3.1 ~~basic wind speed (mph)~~	**Mean roof height (feet)**	**All roof slopes**		

(Portions of Table not shown, remain unchanged)

1705.4 Wind resistance. The statement of special inspections shall include wind requirements for structures constructed in the following areas:

1. In wind Exposure Category B, where ~~the~~ V_{asd} as determined in accordance with Section 1609.3.1 ~~3-second-gust basic wind speed~~ is 120 miles per hour (mph) (52.8 m/s) or greater.
2. In wind Exposure Category C or D, where ~~the~~ V_{asd} as determined in accordance with Section 1609.3.1 ~~3-second- gust basic wind speed~~ is 110 mph (49 m/s) or greater.

1706.1 Special inspections for wind requirements. *Special inspections* itemized in Sections 1706.2 through 1706.4, unless exempted by the exceptions to Section 1704.1, are required for buildings and structures constructed in the following areas:

1. In wind Exposure Category B, where ~~the~~ V_{asd} as determined in accordance with Section 1609.3.1 ~~3-second-gust basic wind speed~~ is 120 miles per hour (52.8 m/sec) or greater.
2. In wind Exposure Categories C or D, where ~~the~~ V_{asd} as determined in accordance with Section 1609.3.1 ~~3-second-gust basic wind speed~~ is 110 mph (49 m/sec) or greater.

1710.3 Structural observations for wind requirements. Structural observations shall be provided for those structures sited where ~~the~~ V_{asd} as determined in accordance with Section 1609.3.1 ~~basic wind speed~~ exceeds 110 mph (49 m/sec) ~~determined from Figure 1609~~, where one or more of the following conditions exist:

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

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. 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.

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 V_{asd} as determined in accordance with Section 1609.3.1 of the *International Building Code* exceeds 110 mph.

TABLE 2304.6.1
MAXIMUM V_{asd} determined in accordance with Section 1609.3.1
~~BASIC WIND SPEED~~ (mph) (3-SECOND GUST) PERMITTED FOR
WOOD STRUCTURAL PANEL WALL SHEATHING USED TO RESIST WIND PRESSURESa,b,c

MINIMUM NAIL		MINIMUM WOOD STRUCTURAL PANEL SPAN RATING	MINIMUM NOMINAL PANEL THICKNESS (inches)	MAXIMUM WALL STUD SPACING (inches)	PANEL NAIL SPACING		MAXIMUM V_{asd} DETERMINED IN ACCORDANCE WITH SECTION 1609.3.1 ~~WIND SPEED (MPH)~~		
							Wind exposure category		
Size	Penetration (inches)				Edges (inches o.c.)	Field (inches o.c.)	B	C	D

(Portions of Table not shown, remain unchanged)

a. Panel strength axis shall be parallel or perpendicular to supports. Three-ply plywood sheathing with studs spaced more than 16 inches on center shall be applied with panel strength axis perpendicular to supports.
b. The table is based on wind pressures acting toward and away from building surfaces in accordance with Section 30.7 ~~6.4.2.2~~ ofASCE7. Lateral requirements shall be in accordance with Section 2305 or 2308.
c. Wood structural panels with span ratings ofwall-16 orwall-24 shall be permitted as an alternate to panels with a 24/0 span rating. Plywood siding rated 16 o.c. or 24 o.c. shall be permitted as an alternate to panels with a 24/16 span rating.Wall-16 and plywood siding 16 o.c. shall be used with studs spaced a maximum of 16 inches o.c.

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/m^2) for combined roof and ceiling, exterior walls, floors and partitions.

   Exceptions:

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

   3.2. Live loads shall not exceed 40 psf (1916 N/m^2) for floors.
   3.3. Ground snow loads shall not exceed 50 psf (2395 N/m^2).

4. V_{asd} as determined in accordance with Section 1609.3.1 ~~Wind speeds~~ shall not exceed 100 miles per hour (mph) (44 m/s) (3-second gust).

Exception: V_{asd} as determined in accordance with Section 1609.3.1 ~~Wind speeds~~ shall not exceed 110 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 Basic wind speed greater than 100 mph (3-second gust). Where the V_{asd} as determined in accordance with Section 1609.3.1 ~~basic wind speed~~ exceeds 100 mph (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&PAWFCM or ICC 600.

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

V_{asd} determined in accordance with Section 1609.3.1 ~~BASIC WIND SPEED (3-second gust)~~	ROOF SPAN (feet)							OVERHANGS (pounds/feet)d
	12	20	24	28	32	36	40	OVERHANGS (pounds/feet)[d]

(Portions of Table not shown, remain unchanged)

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

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

	Mean Roof Height (feet)									
EXPOSURE	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**	**55**	**60**
B	**1.00**	**1.00**	**1.00**	**1.00**	**1.05**	**1.09**	**1.12**	**1.16**	**1.19**	**1.22**
C	**1.21**	**1.29**	**1.35**	**1.40**	**1.45**	**1.49**	**1.53**	**1.56**	**1.59**	**1.62**
D	**1.47**	**1.55**	**1.61**	**1.66**	**1.70**	**1.74**	**1.78**	**1.81**	**1.84**	**1.87**

b. he uplift connection requirements are based on the framing being spaced 24 inches on center. Multiply by 0.67 for framing spaced 16 inches on center and multiply by 0.5 for framing spaced 12 inches on center.
c. The uplift connection requirements include an allowance for 10 pounds of dead load.
d. The uplift connection requirements do not account for the effects of overhangs. The magnitude of the above loads shall be increased by adding the overhang loads found in the table. The overhang loads are also based on framing spaced 24 inches on center. The overhang loads given shall be multiplied by the overhang projection and added to the roof uplift value in the table.
e. The uplift connection requirements are based upon wind loading on end zones as defined in Figure 28.6.3 ~~6-2~~ of ASCE 7. Connection loads for connections located a distance of 20 percent of the least horizontal dimension of the building from the corner of the building are permitted to be reduced by multiplying the table connection value by 0.7 and multiplying the overhang load by 0.8.
f. For wall-to-wall and wall-to-foundation connections, the capacity of the uplift connector is permitted to be reduced by 100 pounds for each full wall above. (For example, if a 500-pound rated connector is used on the roof framing, a 400-pound rated connector is permitted at the next floor level down).
g. Interpolation is permitted for intermediate values of V_{asd} ~~basic wind speeds~~ and roof spans.
h. The rated capacity of approved tie-down devices is permitted to include up to a 60-percent increase for wind effects where allowed by material specifications.

CHAPTER 35
REFERENCED STANDARDS

ASCE/SEI
American Society of Civil Engineers/Structural Engineering Institute
1801 Alexander Bell Drive
Reston, VA 20191-440

7-10 ~~7-05~~ Minimum Design Loads for Buildings and Other Structures ~~including Supplements No. 1 and 2, excluding Chapter 14 and Appendix 11A~~

Reason: The purpose of ths proposal is to update and coordinate the provisions of the 2012 IBC with those of the 2010 edition of ASCE 7 for the determination of wind loads. Although consisting of 30 small parts, the underlying reason for this change is to adopt into the 2012 IBC the new wind speed maps that have been adopted into ASCE 7.

Over the past 10 years, new data and research has been performed that indicates that the hurricane wind speeds provided in the current maps of the IBC-09 and ASCE-05 are too conservative and need to be adjusted downward. Significantly more hurricane data have become available thereby allowing for substantial improvements in the hurricane simulation model that is used to create the wind speed maps.
These new data have resulted in an improved representation of the hurricane wind field, including the modeling of the sea-land transition and the hurricane boundary layer height; new models for hurricane weakening after landfall; and an improved statistical model for the Holland *B* parameter which controls the wind pressure relationship. The new hurricane hazard model yields hurricane wind speeds that are lower than those given in ASCE 7-05 and IBC-09 even though the overall rate of intense storms (as defined by central pressure) produced by the new model is increased compared to those produced by the hurricane simulation model used to develop previous maps.

In preparing the new maps, the ASCE 7 standards committee decided to use multiple ultimate event or strength design maps in conjunction with a wind load factor of 1.0 for strength design – for allowable stress design, the factor was reduced from 1.0 to 0.6. Several factors that are important to an accurate wind load standard led to this decision:

(i) An ultimate event or strength design wind speed map makes the overall approach consistent with that used in seismic design in that they both map ultimate events and use a load factor of 1.0 for strength design.

(ii) Utilizing different maps for the different Occupancy Categories eliminates the problems associated with using “importance factors” that vary with category. The difference in the importance factors in hurricane prone and non-hurricane prone regions for Category I structures prompted many questions and have been removed from ASCE 7-10.

(iii) The use of multiple maps eliminates the confusion associated with the recurrence interval associated with the existing map - the map was not a uniform fifty year return period map. This therefore created a situation where the level of safety provided for within the overall design was not consistent along the hurricane coast.

Utilizing the new wind speed maps and integrating their use into the IBC necessitated the introduction of the terms V_{ult} and V_{asd} to be associated with the “ultimate” design wind speed and the “nominal” design wind speed. Because of the number of different provisions which use the wind speed map to “trigger” different requirements it was necessary to modify the conversion section (1609.3.1) so that those provisions were not changed. The terms “ultimate design wind speed” and “nominal design wind speed” were incorporate in numerous locations to aid in drawing the users attention to the different types of wind speeds – similar to what was done with the change from fastest mile to 3-second gust wind speeds.

Beyond the adoption of the new strength design wind speed maps, the 2010 edition of ASCE 7 also includes a new simplified method for use in the determination of wind loads for buildings up to 160’ in height. In addition, the wind load calculation provisions have been removed from Chapter 6 of ASCE 7 and been reorganized into 6 separate chapters (26 thru 31) for the sake of clarity and ease of use. This of course necessitated multiple coordination revisions with the IBC text.

ASCE/SEI 7 has been a referenced standard of the IBC since its inception and as such it is well known to the building community. ASCE/SEI 7 is published and maintained by the Structural Engineering Institute of the American Society of Civil Engineers (SEI/ASCE). The document is a nationally recognized consensus standard developed in full compliance with the ASCE *Rules for Standards Committees*. The ASCE standards process is fully accredited by the American National Standards Institute (ANSI).

As of the submission date of this code change, the ASCE 7 Standards Committee is completing the committee balloting portion of the 2010 edition of ASCE/SEI 7. The document is designated ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* and it is expected that it will be completed and available for purchase prior to the ICC Final Action Hearings in May of 2010 . Any person interested in obtaining a public comment copy of ASCE/SEI 7-10 may do so by contacting the proponent at jrossberg@asce.org .

Cost Impact: The overall, national cost impact is believed to be neutral.

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

ICCFILENAME: ROSSBERG-S1-1602 NOTATIONS

S85–09/10
1609.1.1, 3108.1

Proponent: Scott Beard PE, SE, City of Tacoma, representing the Structural Engineers Association of Washington.

Revise as follows:

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

Exceptions:

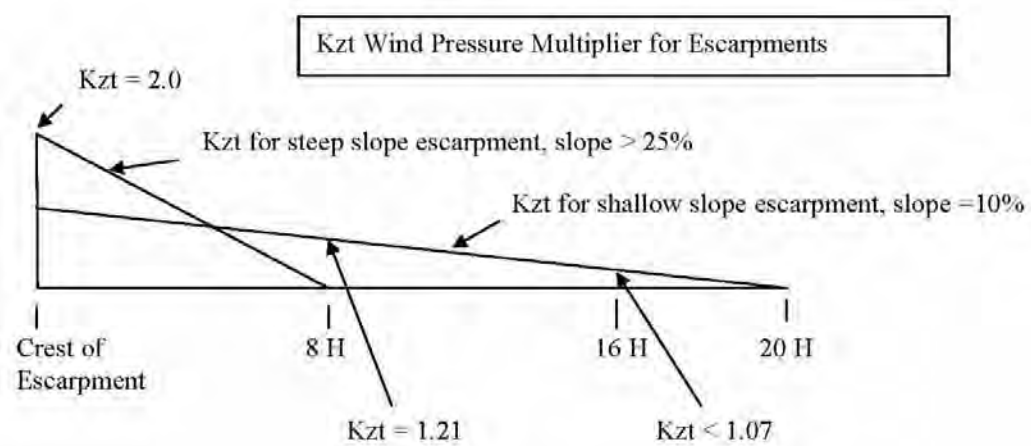
1. Subject to the limitations of Section 1609.1.1.1, the provisions of ICC 600 shall be permitted for applicable Group R-2 and R-3 buildings.
2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of the AF&PA WFCM.
3. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AISI S230.
4. Designs using NAAMM FP 1001.

5. Designs using TIA-222 for antenna-supporting structures and antennas, provided the extent of Topographic Category 2, escarpments, in Section 2.6.6.2 of TIA-222 shall extend 16 times the height of the escarpment.
6. Wind tunnel tests in accordance with Section 6.6 of ASCE 7, subject to the limitations in Section 1609.1.1.2.

3108.1 General. Towers shall be designed and constructed in accordance with the provisions of TIA-222. In Section 2.6.6.2 of TIA-222, the extent of Topographic Category 2, escarpments, shall extend 16 times the height of the escarpment

Exception: Single free-standing poles used to support antennas not greater than 75 feet (22 860 mm), measured from the top of the pole to grade, shall not be required to be noncombustible

Reason: The TIA 222-G standard attempted to simplify the Topographic Wind Speedup Effect. They made a mistake in their application to escarpments. They missed a controlling condition:



TIA correctly found the worst case wind speed-up at the crest for a steep slope, but did not realize that lesser sloped escarpments create pressure increases that cannot be safely ignored, beyond the "steep slope" influence. TIA stopped considering the topographic wind speed-up effect at 8H from the crest. At this location, a shallow slope can still produce a 21% increase in wind pressure.

We checked with TIA, since there are several ways to address this situation. Their preference is to change the 8H value in the standard, to 16H. They say that they will address this in a future revision to the Standard, but in the meantime we need to cover this situation for safety reasons.

The reason that 16H was chosen, rather than 20H, is that there is an equivalent cutoff in the ASCE 7 formula on the face of the escarpment. The cutoff value is within a reasonable engineering value.

Per the TIA 222-G standard, a designer may use the full topographic wind speed-up method of ASCE 7, if they wish to avoid any conservatism created by the simplified method. This revision simply makes sure that the simplified method is safe in all cases.

I attached a copy of the correspondence between the SEAW Wind Committee and the TIA standards committee confirming that changing 8H to 16H would be the preferred way to modify the section.

Cost Impact: This will not increase costs if the designer makes use of all of the design options provided by the Standard It will increase costs in some situations, if the simplified method is used, but costs will still be less than when this mistake was introduced in the 2009 IBC.

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

ICCFILENAME: BEARD-S1-1609.1.1

S86-09/10

1609.1.1, 1609.1.1.2, Chapter 35

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

1. Revise as follows:

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

Exceptions:

1. Subject to the limitations of Section 1609.1.1.1, the provisions of ICC 600 shall be permitted for applicable Group R-2 and R-3 buildings.
2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of the AF&PA WFCM.
3. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AISI 230.
4. Designs using NAAMM FP 1001.
5. Designs using TIA/EIA-222 for antenna-supporting structures and antennas.
6. Wind tunnel tests in accordance with Section 6.6 of ASCE 7, subject to the limitations in Section 1609.1.1.2.
7. <u>Wind tunnel tests in accordance with ASCE/SEI 49, subject to the limitations in Section 1609.1.1.2.</u>

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

2. Add standard to Chapter 35 as follows:

American Society of Civil Engineers/Structural Engineering Institute
<u>49-09</u> <u>Wind Tunnel Testing for Buildings and Other Structures</u>

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

This proposed change is a follow-up to S81-07/08 which was a result of the CTC's investigation of the area of study entitled "Review of NIST WTC Recommendations". The scope of the activity is noted as:

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

The reason this code change was not approved was due to the lack of completion/availability of the standard ASCE/SEI 49 entitled "Wind Tunnel Testing for Buildings and Other Structures". At the time this code change is submitted, the standard has gone through the requiste comment process and the standard is under appeal. As such, this proposal is submitted in anticipation of the standard being completed by the Final Action Hearings.

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

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

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

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

The proposed revision that stipulates that the minimum design loads can not be less than the minimums of ASCE 7 (10 psf) is in response to the committees concern stated in the reason for disapproval of S16 -06/07. It is CTC's understanding that the standard will have been completed by the 2009 Baltimore Code Development Hearings.

References:

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

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

Analysis: A review of the standard(s) proposed for inclusion in the code, ACSE49-09, 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.

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

FILENAME: HEILSTEDT-S1-1609.1.1

S87–09/10
1609.1.1, 1609.1.1.1, Chapter 35; IRC R301.1.1, R301.2.1.1, Chapter 44

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards and Phil Samblanet, The Masonry Society, representing The Masonry Society.

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC-STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

1. Revise as follows:

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

Exceptions:

1. Subject to the limitations of Section 1609.1.1.1, the provisions of ICC 600 shall be permitted for applicable Group R-2 and R-3 buildings.
2. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of the AF&PA WFCM.
3. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of AISI S230.
4. Designs using NAAMM FP 1001.
5. Designs using TIA-222 for antenna-supporting structures and antennas.
6. Wind tunnel tests in accordance with Section 6.6 of ASCE 7, subject to the limitations in Section 1609.1.1.2.
7. Subject to the limitations of Section 1609.1.1.1, residential structures using the provisions of TMS 404.

1609.1.1.1 Applicability. The provisions of ICC 600 are applicable only to buildings located within Exposure B or C as defined in Section 1609.4. The provisions of ICC 600, AF&PA WFCM, TMS 404, and AISI S230 shall not apply to buildings sited on the upper half of an isolated hill, ridge or escarpment meeting the following conditions:

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

2. Add standard to Chapter 35 as follows:

TMS
404-09 Standard for Masonry High Wind Residential Construction

PART II – IRC BUILDING/ENERGY

1. Revise as follows:

R301.1.1 Alternative provisions. As an alternative to the requirements in Section R301.1 the following standards are permitted subject to the limitations of this code and the limitations therein. Where engineered design is used in conjunction with these standards, the design shall comply with the *International Building Code*.

1. American Forest and Paper Association (AF&PA) *Wood Frame Construction Manual* (WFCM).
2. American Iron and Steel Institute (AISI) *Standard for Cold-Formed Steel Framing—Prescriptive Method for One- and Two-Family Dwellings* (AISI S230).
3. ICC-400 *Standard on the Design and Construction of Log Structures*.

4. TMS 404 *Standard for Masonry High Wind Residential Construction*,

R301.2.1.1 Design criteria. In regions where the basic wind speeds from Figure R301.2(4) equal or exceed 100 miles per hour (45 m/s) in *hurricane-prone regions*, or 110 miles per hour (49 m/s) elsewhere, the design of buildings shall be in accordance with one of the following methods. The elements of design not addressed by those documents in Items 1 through ~~4~~5 shall be in accordance with this code.

1. American Forest and Paper Association (AF&PA) *Wood Frame Construction Manual for One- and Two-Family Dwellings* (WFCM); or
2. International Code Council (ICC) *Standard for Residential Construction in High Wind Regions* (ICC-600); or
3. *Minimum Design Loads for Buildings and Other Structures* (ASCE-7); or
4. American Iron and Steel Institute (AISI), *Standard for Cold-Formed Steel Framing—Prescriptive Method For One- and Two-Family Dwellings* (AISI S230)~~.~~; or
5. The Masonry Society (TMS) *Standard for Masonry High Wind Residential Construction* (TMS 404).

6~~5~~. Concrete construction shall be designed in accordance with the provisions of this code.
7~~6~~. Structural insulated panel (SIP) walls shall be designed in accordance with the provisions of this code.

2. Add standard to Chapter 44 as follows:

TMS
404-09 Standard for Masonry High Wind Residential Construction

Reason: This modification proposes to introduce a standard for design and construction of masonry residential structures in high wind areas based on a new, mandatory language reference standard TMS 404, *Standard for Masonry High Wind Residential Construction*. The methodology used to develop the *Standard for Masonry High Wind Residential Construction* is based upon the strength design provisions of the 2005 and 2008 editions of the TMS 402/ACI 530/ASCE 5, *Building Code Requirements for Masonry Structures* and the factored combinations of dead and wind loads in accordance with the 2005 edition of ASCE 7, *Minimum Design Loads for Buildings and Other Structures*.

This new design standard is a corollary to the American Forest and Paper Association (AF&PA) *Wood Frame Construction Manual for One- and Two-Family Dwellings* (WFCM) and the American Iron and Steel Institute (AISI), *Standard for Cold-Formed Steel Framing—Prescriptive Method For One- and Two-Family Dwellings (COFS/PM.* The Standard provides minimum requirements for structural component survivability during high wind events of masonry walled residential one and two-family structures up to and including 30 ft. in height for design wind speeds of 100 to 150 mph, 3 second gust as defined by ASCE 7-05. The standard is an alternative to the ICC-600 *Standard for Residential Construction in High Wind Regions* adopted in the 2009 Edition of the IBC. The TMS 404 Standard is also more flexible than the ICC-600 Standard in that it allows actual design of the masonry sections through a series of tables that determines applied loads that are then used in design tables to determine the size and spacing of reinforcement for walls, lintels, and bond beams. The ICC-600 is a prescriptive standard that lists reinforcement configurations based on wind zone and building size.

Those interested in reviewing a draft of the TMS 404 Standard for Masonry High Wind Residential Construction are encouraged to download a working draft of the document at the following link:
http://www.masonrysociety.org/html/resources/TMS-404/TMS404.htm

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, TMS404-09, 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.

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: THOMPSON-SAMBLANET-S3-1609.1.1

S88–09/10
1609.1.2

Proponent: Kurt Roeper, representing Ingersoll Rand

Revise as follows:

1609.1.2 Protection of openings. In wind-borne debris regions, glazing in buildings shall be impact-resistant or protected with an impact-resistant covering meeting the impact requirements of ICC 500, an approved impact-resistant standard or ASTM E 1996 and ASTM E 1886 referenced herein as follows:

1. Glazed openings located within 30 feet (9144 mm) of grade shall meet the requirements of the large missile test of ASTM E 1996.
2. Glazed openings located more than 30 feet (9144 mm) above grade shall meet the provisions of the small missile test of ASTM E 1996.

Exceptions:

1. Wood structural panels with a minimum thickness of 7/16 inch (11.1 mm) and maximum panel span of 8 feet (2438 mm) shall be permitted for opening protection in one- and two-story buildings classified as Group R-3 or R-4 occupancy. Panels shall be precut so that they shall be attached to the framing surrounding the opening containing the product with the glazed opening. Panels shall be predrilled as required for the anchorage method and shall be secured with the attachment hardware provided. Attachments shall be designed to resist the components and cladding loads determined in accordance with the provisions of ASCE 7, with corrosion-resistant attachment hardware provided and anchors permanently installed on the building. Attachment in accordance with Table 1609.1.2 with corrosion-resistant attachment hardware provided and anchors permanently installed on the building is permitted for buildings with a mean roof height of 45 feet (13 716 mm) or less where wind speeds do not exceed 140 mph (63 m/s).
2. Glazing in *Occupancy Category* I buildings as defined in Section 1604.5, including greenhouses that are occupied for growing plants on a production or research basis, without public access shall be permitted to be unprotected.
3. Glazing in *Occupancy Category* II, III or IV buildings located over 60 feet (18 288 mm) above the ground and over 30 feet (9144 mm) above aggregate surface roofs located within 1,500 feet (458 m) of the building shall be permitted to be unprotected.

Reason: The design provision for wall pier detailing was originally introduced by SEAOC in 1987 to legacy Uniform Building Code and was included in 1988 UBC through 1997 UBC. The wall pier detailing provision prescribed under Section 1908.1.4 was intended for high seismic zones equivalent to current SDC D, E or F. 1908.1.3 was added as a complement of wall pier detailing in SDC C (formerly seismic zones 2A and 2B under legacy model code.) ACI 318 Commentary R 21.1.1 emphasized "*it is essential that structures assigned to higher SDC's possess a higher degree of toughness*", and further encourages practitioners to use special structural wall system in regions of high seismic risk. ASCE 7 Table 12.2-1 permits intermediate precast structural wall system in SDC D, E or F. Current Section 1908.1.3 does not limit to just structures assigned to SDC C. The required shear strength under 21.3.3, referenced in current Sec. 21.4.5, is based on V_u under either nominal moment strength or two times the code prescribed earthquake force. The required shear strength in 21.6.5.1, referenced in Sec. 21.9.10.2 (IBC 1908.1.4), is based on the probable shear strength, V_e under the probable moment strength, M_{pr}. In addition, the spacing of required shear reinforcement is 8 inches on center under current 21.4.5 instead of 6 inches on center with seismic hooks at both ends under 21.9.10.2. Requirement of wall pier under 21.9.10.2 would enhance better ductility.

Current practice in commercial buildings constructed using precast panels wall system have large window and door openings and/or narrow wall piers. Wall panels varying up to three stories high with openings resembles wall frame which is not currently recognized under any of the defined seismic-force resisting systems other than consideration of structural wall system. Conformance to special structural wall system design and detailing of wall piers ensures minimum life safety performance in resisting earthquake forces for structures in SDC D, E or F. Proposed modification separates wall piers designed for structures assigned to SDC C from those assigned to 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.

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

ICCFILENAME: ROEPER-S1-1609.1.2

S89–09/10
1609.1.2

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

Revise as follows:

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

1. Glazed openings located within 30 feet (9144 mm) of grade shall meet the requirements of the large missile test of ASTM E 1996.
2. Glazed openings located more than 30 feet (9144 mm) above grade shall meet the provisions of the small missile test of ASTM E 1996.

   Exceptions:

   1. Wood structural panels with a minimum thickness of 7/16 inch (11.1 mm) and maximum panel span of 8 feet (2438 mm) shall be permitted for opening protection in one- and two-story buildings classified as Group R-3 or R-4 occupancy. Panels shall be precut so that they shall be attached to the framing surrounding the opening containing the product with the glazed opening. Panels shall be predrilled as required for the anchorage method and shall be secured with the attachment hardware provided. Attachments shall be designed to resist the components and cladding loads determined in accordance with the provisions of ASCE 7, with corrosion-resistant attachment hardware provided and anchors permanently installed on the building. Attachment in accordance with Table 1609.1.2 with corrosion-resistant attachment hardware provided and anchors permanently installed on the building is permitted for buildings with a mean roof height of 45 feet (13 716 mm) or less where wind speeds do not exceed 140 mph (63 m/s).
   2. Glazing in *Occupancy Category* I buildings as defined in Section 1604.5, including greenhouses that are occupied for growing plants on a production or research basis, without public access shall be permitted to be unprotected.
   3. Glazing in *Occupancy Category* II, III or IV buildings located over 60 feet (18 288 mm) above the ground and over 30 feet (9144 mm) above aggregate surface roofs located within 1,500 feet (458 m) of the building shall be permitted to be unprotected.

Reason: This proposal clarifies that openings – and not just glazing – in buildings are to be impact-resistant or protected with impact-resistant coverings in wind-borne debris regions. Non-glazed openings are vulnerable to impact by wind-borne debris which could compromise the integrity of the building envelope in preventing damage or destruction by hurricane-strength wind. Thus, non-glazed openings should be held to the same impact-resistance performance requirements as glazed openings.

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

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

ICCFILENAME: WOESTMAN-S3-1609.1.2

S90–09/10
1609.1.2.2, CHAPTER 35

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 text 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.

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

ICCFILENAME: WOESTMAN-S1-1609.1.2.2

S91–09/10
1612.2; IEBC 202

Proponent: Gary R. Searer, Wiss, Janney, Elstner Associates, Inc.,

THIS IS A 2 PART CODE CHANGE. BOTH PARTS WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDER FOR THIS COMMITTEE.

PART I- IBC STRUCTURAL

Revise as follows:

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

SUBSTANTIAL DAMAGE. Damage of any origin sustained by a building or structure whereby the cost of restoring the building or structure to its ~~before-damaged~~ pre-damage condition would equal or exceed 50 percent of the ~~market~~ replacement value of the building or structure before the damage occurred.

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

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

(Definitions not shown are unchanged)

PART II- EXISTING BUILDING

Revise as follows:

SECTION 202
GENERAL DEFINITIONS

SUBSTANTIAL DAMAGE. For the purposes of determining compliance with the flood provisions of this code, damage of any origin sustained by a building or structure whereby the cost of restoring the building or structure to its ~~before-damaged~~ pre-damage condition would equal or exceed 50 percent of the ~~market~~ replacement value of the building or structure before the damage occurred.

SUBSTANTIAL IMPROVEMENT. For the purposes of determining compliance with the flood provisions of this code, any repair, alteration, addition or improvement of a building or structure, the cost of which equals or exceeds 50 percent of the ~~market~~ replacement value of the building or structure~~,~~ before the improvement or repair is started. If the building or structure has sustained substantial damage, any repairs are considered substantial improvement regardless of the actual repair work performed. The term does not, however, include either:

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

(Definitions not shown are unchanged)

Reason: This proposal does three things:
1. Adds the words "building or" and "or structure" to the definitions of substantial damage and substantial improvement.
2. Changes the term "before-damaged" to "pre-damage"
3. Changes the criteria for determining "substantial" from 50 percent of the market value to 50 percent of the replacement cost of the building or structure.

The first two changes are editorial and are intended to clarify the intended meaning of the definitions. The third change is intended to simplify the method by which substantial damage or substantial improvement are determined. Currently, the definitions require the determination as to the "market value" of a building or structure as it existed before the damage or alteration; this might arguably require the services of a real estate appraiser or real estate agent to determine the "market value". Instead, by linking the definitions to "replacement value", the engineer's job is made more simple because the replacement value can easily be determined using a nationally recognized cost estimation resource such as RS Means or National Construction Estimator.

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

PART I-IBC STRUCTURAL

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

PART II- IEBC:

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

ICCFILENAME: Searer-S2-1612-EB1-506.2.2.3

S92–09/10

IBC 801.5, 1403.5; IPC [B] 309.2; IFGC [B] 301.11; IMC [B] 301.13, 401.4, 501.2.1, [B] 602.4, [B] 603.13, 1305.2.1

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

THIS IS A 4 PART CODE CHANGE. ALL PARTS WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. SEE THE TENTATIVE HEARING ORDER FOR THIS COMMITTEE.

PART I – IBC

Revise as follows:

801.5 Applicability. For buildings in flood hazard areas as established in Section 1612.3, interior finishes, trim and decorative materials that extend below the elevation required by Section 1612.4 ~~design flood elevation~~ shall be flood-damage-resistant materials.

1403.5 Flood resistance. For buildings in flood hazard areas as established in Section 1612.3, exterior walls extending below the elevation required by Section 1612.4 shall be constructed with flood damage resistant materials ~~design flood elevation shall be resistant to water damage~~. Wood shall be pressure-preservative treated in accordance with AWPA U1 for the species, product and end use using a preservative listed in Section 4 of APWA U1 or decay-resistant heartwood of redwood, black locust or cedar.

PART II – IPC

Revise as follows:

[B] 309.2 Flood hazard. For structures located in flood hazard areas, the following systems and equipment shall be located ~~at or above~~ and installed as required by Section 1612.4 of the *International Building Code* ~~the design flood elevation~~.

> **Exception:** The following systems are permitted to be located below the ~~design flood elevation~~ the elevation required by Section 1612.4 of the *International Building Code* for utilities and attendant equipment provided that the systems are designed and installed to prevent water from entering or accumulating within their components and the systems are constructed to resist hydrostatic and hydrodynamic loads and stresses, including the effects of buoyancy, during the occurrence of flooding to up to such ~~the design flood~~ elevation.

1. All water service pipes.
2. Pump seals in individual water supply systems where the pump is located below the design flood elevation.
3. Covers on potable water wells shall be sealed, except where the top of the casing well or pipe sleeve is elevated to at least 1 foot (305 mm) above the design flood elevation.
4. All sanitary drainage piping.
5. All storm drainage piping.
6. Manhole covers shall be sealed, except where elevated to or above the design flood elevation.
7. All other plumbing fixtures, faucets, fixture fittings, piping systems and equipment.
8. Water heaters.
9. Vents and vent systems.

PART III – IFGC

Revise as follows:

[B] 301.11 Flood hazard. For structures located in flood hazard areas, the appliance, equipment and system installations regulated by this code shall be located at or above the elevation required by Section 1612.4 of the *International Building Code* for utilities and attendant equipment ~~design flood elevation and shall comply with the flood-resistant construction requirements of the *International Building Code*~~.

> **Exception:** The appliance, equipment and system installations regulated by this code are permitted to be located below the ~~design flood~~ elevation required by Section 1612.4 of the *International Building Code* for utilities and

attendant equipment provided that they are designed and installed to prevent water from entering or accumulating within the components and to resist hydrostatic and hydrodynamic loads and stresses, including the effects of buoyancy, during the occurrence of flooding to such elevation. ~~the design flood elevation shall comply with the flood-resistant construction requirements of the *International Building Code*.~~

PART IV – IMC

Revise as follows:

[B] 301.13 Flood hazard. For structures located in flood hazard areas, mechanical systems, equipment and appliances shall be located at or above the elevation required by Section 1612.4 of the *International Building Code* for utilities and attendant equipment ~~design flood elevation~~.

Exception: Mechanical systems, equipment and appliances are permitted to be located below the ~~design flood~~ elevation required by Section 1612.4 of the of the *International Building Code* for utilities and attendant equipment provided that they are designed and installed to prevent water from entering or accumulating within the components and to resist hydrostatic and hydrodynamic loads and stresses, including the effects of buoyancy, during the occurrence of flooding up to such elevation. ~~to the design flood elevation in compliance with the flood-resistant construction requirements of the *International Building Code*.~~

401.4 Intake opening location. Air intake openings shall comply with all of the following:
4. Intake openings on structures in flood hazard areas shall be at or above the elevation required by Section 1612.4 of the *International Building Code* for utilities and attendant equipment ~~design flood elevation~~.

501.2.1 Location of exhaust outlets. The termination point of exhaust outlets and ducts discharging to the outdoors shall be located with the following minimum distances:

1. For ducts conveying explosive or flammable vapors, fumes or dusts: 30 feet (9144 mm) from property lines; 10 feet (3048 mm) from operable openings into buildings; 6 feet (1829 mm) from exterior walls and roofs; 30 feet (9144 mm) from combustible walls and operable openings into buildings which are in the direction of the exhaust discharge; 10 feet (3048 mm) above adjoining grade.
2. For other product-conveying outlets: 10 feet (3048 mm) from the property lines; 3 feet (914 mm) from exterior walls and roofs; 10 feet (3048 mm) from operable openings into buildings; 10 feet (3048 mm) above adjoining grade.
3. For all *environmental air* exhaust: 3 feet (914 mm) from property lines; 3 feet (914 mm) from operable openings into buildings for all occupancies other than Group U, and 10 feet (3048 mm) from mechanical air intakes. Such exhaust shall not be considered hazardous or noxious.
4. Exhaust outlets serving structures in flood hazard areas shall be installed at or above the elevation required by Section 1612.4 of the *International Building Code* for utilities and attendant equipment ~~design flood elevation~~.
5. For specific systems see the following sections:
   - 5.1. Clothes dryer exhaust, Section 504.4.
   - 5.2. Kitchen hoods and other kitchen exhaust *equipment*, Sections 506.3.12, 506.4 and 506.5.
   - 5.3. Dust stock and refuse conveying systems, Section 511.
   - 5.4. Subslab soil exhaust systems, Section 512.4
   - 5.5. Smoke control systems, Section 513.10.3
   - 5.6. Refrigerant discharge, Section1105.7
   - 5.7. Machinery room discharge, Section 1105.6.1

[B] 602.4 Flood hazard. For structures located in flood hazard areas, plenum spaces shall be located above the elevation required by Section 1612.4 of the *International Building Code* for utilities and attendant equipment ~~design flood elevation~~ or shall be designed and constructed to prevent water from entering or accumulating within the plenum spaces during floods up to such ~~the design flood~~ elevation. If the plenum spaces are located below the elevation required by Section 1612.4 of the *International Building Code* for utilities and attendant equipment ~~design flood elevation~~, they shall be capable of resisting hydrostatic and hydrodynamic loads and stresses, including the effects of buoyancy, during the occurrence of flooding up to such ~~to the design flood~~ elevation.

[B] 603.13 Flood hazard areas. For structures in flood hazard areas, ducts shall be located above the elevation required by Section 1612.4 of the *International Building Code* for utilities and attendant equipment ~~design flood elevation~~ or shall be designed and constructed to prevent water from entering or accumulating within the ducts during floods up to such ~~the design flood~~ elevation. If the ducts are located below the elevation required by Section 1612.4 of the *International Building Code* for utilities and attendant equipment ~~design flood elevation~~, the ducts shall be capable of resisting hydrostatic and hydrodynamic loads and stresses, including the effects of buoyancy, during the occurrence of flooding up to such ~~the design flood~~ elevation.

1305.2.1 Flood hazard. All fuel oil pipe, equipment and appliances located in flood hazard areas shall be located above the elevation required by Section 1612.4 of the *International Building Code* for utilities and attendant equipment ~~design flood elevation~~ or shall be capable of resisting hydrostatic and hydrodynamic loads and stresses, including the effects of buoyancy, during the occurrence of flooding up to such ~~the design flood~~ elevation.

Reason:
Part I – The purpose of this code change is to provide consistency between the elevations of buildings and structures that are specified in Section 1612.4 and the elevations below which flood damage-resistant materials are to be used. For specific elevation requirements for flood damage-resistant materials, Section 1612.4 refers to ASCE 24, *Flood Resistant Design and Construction*.
Part II - In the *International Building Code,* Section 1612.4 refers to ASCE 24, *Flood Resistant Design and Construction,* which specifies elevations as a function of flood zone and Structure Category. This proposal will result in consistency between the elevations of buildings and structures that are specified in ASCE 24 and the elevations of the plumbing systems and equipment that serve those buildings and structures, which are also specified in ASCE 24.
Part III – In the *International Building Code,* Section 1612.4 refers to ASCE 24, *Flood Resistant Design and Construction,* which specifies elevations as a function of flood zone and Structure Category and which specifies the performance under loads that needs to be provided if equipment is located below such elevations.. This proposal will result in consistency between the elevations of buildings and structures that are specified in ASCE 24 and the elevations of equipment that serve those buildings and structures, which are also specified in ASCE 24.

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

PART I – IBC

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

PART II – IPC

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

PART III – IFGC

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

PART IV – IMC

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

ICCFILENAME: QUINN-S2 4 parts

S93–09/10
1612.5

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

Revise as follows:

1612.5 Flood hazard documentation. The following documentation shall be prepared and sealed by a registered design professional and shall be submitted to the building official:

1. For construction in flood hazard areas not subject to high-velocity wave action:
   1.1. The elevation of the lowest floor, including basement, as required by the lowest floor elevation inspection in Section 110.3.3.
   1.2. For fully enclosed areas below the design flood elevation where provisions to allow for the automatic entry and exit of floodwaters do not meet the minimum requirements in Section 2.6.2.1, ASCE 24, construction documents shall include a statement that the design will provide for equalization of hydrostatic flood forces in accordance with Section 2.6.2.2 of ASCE 24.

1.3. For dry floodproofed nonresidential buildings, construction documents shall include a statement that the dry floodproofing is designed in accordance with ASCE 24.

2. For construction in flood hazard areas subject to high-velocity wave action:
   2.1. The elevation of the bottom of the lowest horizontal structural member as required by the lowest floor elevation inspection in Section 110.3.3.
   2.2. Construction documents shall include a statement that the building is designed in accordance with ASCE 24, including that the pile or column foundation and building or structure to be attached thereto is designed to be anchored to resist flotation, collapse and lateral movement due to the effects of wind and flood loads acting simultaneously on all building components, and other load requirements of Chapter 16.
   2.3. For breakaway walls designed to resist a nominal load of ~~less that 10 psf (0.48 kN/m^2) or~~ more than 20 psf (0.96 kN/m^2), construction documents shall include a statement that the breakaway wall is designed in accordance with ASCE 24.

Reason: In Section 4.6.1, ASCE 24, *Flood Resistant Design and Construction*, requires breakaway walls and their connections to be designed in accordance with the requirements of ASCE 7, *Minimum Design Loads for Buildings and Other Structures.* ASCE 7 does not allow such walls to be designed for less than 10 psf (see Section 5.3.2.3 in ASCE 7-02). Therefore, this code change is appropriate for consistency with both standards. In addition, the regulations of the National Flood Insurance Program do not provide for breakaway walls with a design safe loading resistance of less than 10 psf (see 44 C.F.R. §60.3(e)(5)).

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

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

ICCFILENAME: QUINN-S1-1612.5

S94–09/10
1612.6 (New), Chapter 35

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

1. Add a new text as follows:

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

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

<u>**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.</u>

<u>**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.</u>

<u>**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.</u>

<u>**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.</u>

> <u>**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.</u>

2. Add standard to Chapter 35 as follows:

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

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.

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

ICCFILENAME: MAHONEY-S2-1612.6 NEW

S95–09/10
1613.1-1613.7

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

1. Revise as follows:

1613.1 Scope. Every structure, and portion thereof, including nonstructural components that are permanently attached to structures and their supports and attachments, shall be designed and constructed to resist the effects of earthquake motions in accordance with ASCE 7, excluding Chapters 14 and Appendix 11A. The *seismic design category* for a structure shall ~~is permitted to~~ be determined in accordance with Section 11.6 of ~~1613 or~~ ASCE 7.

Exceptions:

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

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

~~**DESIGN EARTHQUAKE GROUND MOTION.** The earthquake ground motion that buildings and structures are specifically proportioned to resist in Section 1613.~~

~~**MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION.** The most severe earthquake effects caused by this code.~~

MECHANICAL SYSTEMS. For the purposes of determining seismic loads in ASCE 7, mechanical systems shall include plumbing systems as specified herein.

~~**ORTHOGONAL.** To be in two horizontal directions, at 90 degrees (1.57 rad) to each other.~~

SEISMIC DESIGN CATEGORY. A classification assigned to a structure based on its occupancy category and the severity of the design earthquake ground motion at the site.

~~**SEISMIC-FORCE-RESISTING SYSTEM.** That part of the structural system that has been considered in the design to provide the required resistance to the prescribed seismic forces.~~

~~**SITE CLASS.** A classification assigned to a site based on the types of soils present and their engineering properties as defined in Secton1613.5.2.~~

~~**SITE COEFFICIENTS.** The values of F_a and F_v indicated in Tables 1613.5.3(1) and 1613.5.3(2) respectively.~~

2. Delete without substitution:

~~**1613.3 Existing buildings.** *Additions*, *alterations*, repairs or change of occupancy of existing buildings shall be in accordance with Chapter 34.~~

~~**1613.4 Special inspections.** Where required by Sections 1705.3 through 1705.3.5, the statement of special inspections shall include the *special inspections* required by Section 1705.3.6.~~

~~**1613.5 Seismic ground motion values.** Seismic ground motion values shall be determined in accordance with this section.~~

~~**1613.5.1 Mapped acceleration parameters.** The parameters S_s and S_1 shall be determined from the 0.2 and 1-second spectral response accelerations shown on Figures 1613.5(1) through 1613.5(14). Where S_1 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.~~

~~**1613.5.2 Site class definitions.** Based on the site soil properties, the site shall be classified as either *Site Class* A, B, C, D, E or F in accordance with Table 1613.5.2. When the soil properties are not known in sufficient detail to determine the *site class*, *Site Class*D shall be used unless the *building official* or geotechnical data determines that *Site Class* E or F soil is likely to be present at the site.~~

~~TABLE 1613.5.2~~
~~SITE CLASS DEFINITIONS~~

SITE CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 100 feet, SEE SECTION 1613.5.5		
		Soil shear wave velocity, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_s > 5,000$	N/A	N/A
B	Rock	$2,500 < \bar{v}_s \leq 5,000$	N/A	N/A
C	Very dense soil and soft rock	$1,200 < \bar{v}_s \leq 2,500$	$\bar{N} > 50$	$\bar{s}_u \geq 2,000$
D	Stiff soil profile	$600 \leq \bar{v}_s \leq 1,200$	$15 \leq \bar{N} \leq 50$	$1,000 \leq \bar{s}_u \leq 2,000$
E	Soft soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	$\bar{s}_u < 1,000$
E	—	Any profile with more than 10 feet of soil having the following characteristics: 1. Plasticity index $PI > 20$, 2. Moisture content $w \geq 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ feet)		

~~For SI: 1 foot = 304.8 mm, 1 square foot = 0.0929m^2, 1 pound per square foot = 0.0479 kPa. N/A = Not applicable~~

~~**1613.5.3 Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters.** The maximum considered earthquake spectral response acceleration for short periods, S_{MS}, and at 1-second period, S_{M1}, adjusted for *site class* effects shall be determined by Equations 16-36 and 16-37, respectively:~~

$S_{MS} = F_a S_s$ **~~(Equation 16-36)~~**
$S_{M1} = F_v S_1$ **~~(Equation 16-37)~~**

~~where:~~

~~F_a = Site coefficient defined in Table 1613.5.3(1).~~
~~F_v = Site coefficient defined in Table 1613.5.3(2).~~
~~S_S = The mapped spectral accelerations for short periods as determined in Section 1613.5.1.~~
~~S_1 = The mapped spectral accelerations for a 1-second period as determined in Section 1613.5.1.~~

~~TABLE 1613.5.3(1)~~
~~VALUES OF SITE COEFFICIENT F_a~~[a]

~~SITE CLASS~~	~~MAPPED SPECTRAL RESPONSE ACCELERATION AT SHORT PERIOD~~				
	~~$S_s \le 0.25$~~	~~$S_s = 0.50$~~	~~$S_s = 0.75$~~	~~$S_s = 1.00$~~	~~$S_s \ge 1.25$~~
~~A~~	~~0.8~~	~~0.8~~	~~0.8~~	~~0.8~~	~~0.8~~
~~B~~	~~1.0~~	~~1.0~~	~~1.0~~	~~1.0~~	~~1.0~~
~~C~~	~~1.2~~	~~1.2~~	~~1.1~~	~~1.0~~	~~1.0~~
~~D~~	~~1.6~~	~~1.4~~	~~1.2~~	~~1.1~~	~~1.0~~
~~E~~	~~2.5~~	~~1.7~~	~~1.2~~	~~0.9~~	~~0.9~~
~~F~~	~~Note b~~	~~Note b~~	~~Note b~~	~~Note b~~	~~Note b~~

~~a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period, *Ss*.~~
~~b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.~~

~~TABLE 1613.5.3(2)~~
~~VALUES OF SITE COEFFICIENT F_V~~[a]

~~SITE CLASS~~	~~MAPPED SPECTRAL RESPONSE ACCELERATION AT 1-SECOND PERIOD~~				
	~~$S_1 \le 0.1$~~	~~$S_1 = 0.2$~~	~~$S_1 = 0.3$~~	~~$S_1 = 0.4$~~	~~$S_1 \ge 0.5$~~
~~A~~	~~0.8~~	~~0.8~~	~~0.8~~	~~0.8~~	~~0.8~~
~~B~~	~~1.0~~	~~1.0~~	~~1.0~~	~~1.0~~	~~1.0~~
~~C~~	~~1.7~~	~~1.6~~	~~1.5~~	~~1.4~~	~~1.3~~
~~D~~	~~2.4~~	~~2.0~~	~~1.8~~	~~1.6~~	~~1.5~~
~~E~~	~~3.5~~	~~3.2~~	~~2.8~~	~~2.4~~	~~2.4~~
~~F~~	~~Note b~~	~~Note b~~	~~Note b~~	~~Note b~~	~~Note b~~

~~a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period, *S*1.~~
~~b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.~~

~~**1613.5.4 Design spectral response acceleration parameters.** Five-percent damped design spectral response acceleration at short periods, *SDS*, and at 1-second period, *SD*1, shall be determined from Equations 16-38 and 16-39, respectively:~~

$$\cancel{S_{DS} = \frac{2}{3} S_{MS}}$$ **~~(Equation 16-38)~~**

$$\cancel{S_{D1} = \frac{2}{3} S_{M1}}$$ **~~(Equation 16-39)~~**

~~where:~~

~~S_{MS} = The maximum considered earthquake spectral response accelerations for short period as determined in Section 1613.5.3.~~
~~S_{M1} = The maximum considered earthquake spectral response accelerations for 1-second period as determined in Section 1613.5.3.~~

~~**1613.5.5 Site classification for seismic design.** Site classification for *Site Class* C, D or E shall be determined from Table 1613.5.5.~~

~~The notations presented below apply to the upper 100 feet (30 480 mm) of the site profile. Profiles containing distinctly different soil and/or rock layers shall be subdivided into those layers designated by a number that ranges~~

~~from 1 to *n* at the bottom where there is a total of *n* distinct layers in the upper 100 feet (30 480 mm). The symbol *i* then refers to any one of the layers between 1 and *n*.~~

~~where:~~

~~v_{si} = The shear wave velocity in feet per second (m/s).~~
~~d_i = The thickness of any layer between 0 and 100 feet (30 480 mm).~~

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

~~**(Equation 16-40)**~~

$$\sum_{i=1}^{n} d_i = \text{100 feet (30 480 mm)}$$

~~*Ni* is the Standard Penetration Resistance (ASTM D 1586) not to exceed 100 blows/foot (328 blows/m) as directly measured in the field without corrections. When refusal is met for a rock layer, *Ni* shall be taken as 100 blows/foot (328 blows/m).~~

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

~~**(Equation 16-41)**~~

~~where *Ni* and *di* in Equation 16-41 are for cohesionless soil, cohesive soil and rock layers.~~

$$\bar{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

~~**(Equation 16-42)**~~

~~where:~~

$$\sum_{i=1}^{m} d_i = d_s$$

~~Use d_i and N_i for cohesionless soil layers only in Equation 16-42.~~
~~d_s = The total thickness of cohesionless soil layers in the top 100 feet (30 480 mm).~~
~~m = The number of cohesionless soil layers in the top 100 feet (30 480 mm).~~
~~s_{ui} = The undrained shear strength in psf (kPa), not to exceed 5,000 psf (240 kPa), ASTM D 2166 or D~~

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}}$$

~~**(Equation 16-43)**~~

$$\sum_{i=1}^{k} d_i = d_c$$

~~d_c = The total thickness of cohesive soil layers in the top 100 feet (30 480 mm).~~
~~k = The number of cohesive soil layers in the top 100 feet (30 480 mm).~~
~~P_I = The plasticity index, ASTM D 4318.~~
~~w = The moisture content in percent, ASTM D 2216.~~

TABLE 1613.5.5
SITE CLASSIFICATION[a]

SITE CLASS	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
E	< 600 ft/s	< 15	< 1,000 psf
D	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
C	1,200 to 2,500 ft/s	> 50	> 2,000

For SI: 1 foot per second = 304.8 mm per second, 1 pound per square foot = 0.0479kN/m2.
a. If the s_u method is used and the N_{ch} and s_u criteria differ, select the category with the softer soils (for example, use Site Class E instead of D).

Where a site does not qualify under the criteria for *Site Class* F and there is a total thickness of soft clay greater than 10 feet (3048 mm) where a soft clay layer is defined by: s_u 500 psf (24 kPa),*w* _ 40 percent, and PI _ 20, it shall be classified as *Site Class* E.

The shear wave velocity for rock, *Site Class* B, shall be either measured on site or estimated by a geotechnical engineer or engineering geologist/seismologist for competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shearwave velocity or classified as *Site Class* C.

The hard rock category, *Site Class* A, shall be supported by shear wave velocity measurements either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 100 feet (30 480 mm), surficial shear wave velocity measurements are permitted to be extrapolated to assess *vs* .

The rock categories, *Site Classes* A and B, shall not be used if there is more than 10 feet (3048 mm) of soil between the rock surface and the bottom of the spread footing or mat foundation.

1613.5.5.1 Steps for classifying a site.

1. Check for the four categories of *Site Class* F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as *Site Class* F and conduct a site-specific evaluation.
2. Check for the existence of a total thickness of soft clay > 10 feet (3048 mm) where a soft clay layer is defined by: s_u < 500 psf (24 kPa), $w \geq 40$ percent and $PI > 20$. If these criteria are satisfied, classify the site as *Site Class* E.
3. Categorize the site using one of the following three methods with v_s , N, and s_u and computed in all cases as specified.
   - 3.1. v_s for the top 100 feet (30 480 mm) (v_s method).
   - 3.2. N for the top 100 feet (30 480 mm) (Nmethod).
   - 3.3. N_{ch} for cohesionless soil layers (PI< 20) in the top 100 feet (30 480 mm) and average, s_u for cohesive soil layers (PI > 20) in the top 100 feet (30 480 mm) (s_u method).

1613.5.6 Determination of seismic design category. Structures classified as *Occupancy Category* I, II or III that are located where the mapped spectral response acceleration parameter at 1-second period, S_1, is greater than or equal to 0.75 shall be assigned to *Seismic Design Category* E. Structures classified as *Occupancy Category* IV that are located where the mapped spectral response acceleration parameter at 1-second period, S_1, is greater than or equal to 0.75 shall be assigned to *Seismic Design Category* F. All other structures shall be assigned to a *seismic design category* based on their *occupancy category* and the design spectral response acceleration coefficients, S_{DS} and S_{D1}, determined in accordance with Section 1613.5.4 or the sitespecific procedures of ASCE 7. Each building and structure shall be assigned to the more severe *seismic design category* in accordance with Table 1613.5.6(1) or 1613.5.6(2), irrespective of the fundamental period of vibration of the structure, *T*.

~~**TABLE 1613.5.6(1)**~~
~~**SEISMIC DESIGN CATEGORY BASED ON**~~
~~**SHORT-PERIOD RESPONSE ACCELERATIONS**~~

~~**VALUE OF S_{DS}**~~	~~**OCCUPANCY CATEGORY**~~		
	~~**I or II**~~	~~**III**~~	~~**IV**~~
~~$S_{DS} < 0.167g$~~	~~A~~	~~A~~	~~A~~
~~$0.167g \le S_{DS} < 0.33g$~~	~~B~~	~~B~~	~~C~~
~~$0.33g \le S_{DS} < 0.50g$~~	~~C~~	~~C~~	~~D~~
~~$0.50g \le S_{DS}$~~	~~D~~	~~D~~	~~D~~

~~**TABLE 1613.5.6(2)**~~
~~**SEISMIC DESIGN CATEGORY BASED ON**~~
~~**1-SECOND PERIOD RESPONSE ACCELERATION**~~

~~**VALUE OF S_{D1}**~~	~~**OCCUPANCY CATEGORY**~~		
	~~**I or II**~~	~~**III**~~	~~**IV**~~
~~$S_{D1} < 0.067g$~~	~~A~~	~~A~~	~~A~~
~~$0.067g \le S_{D1} < 0.133g$~~	~~B~~	~~B~~	~~C~~
~~$0.133g \le S_{D1} < 0.20g$~~	~~C~~	~~C~~	~~D~~
~~$0.20g \le S_{D1}$~~	~~D~~	~~D~~	~~D~~

~~**1613.5.6.1 Alternative seismic design category determination.** Where S_1 is less than 0.75, the *seismic design category* is permitted to be determined from Table 1613.5.6(1) alone when all of the following apply:~~

1. ~~In each of the two orthogonal directions, the approximate fundamental period of the structure, T_a, in each of the two orthogonal directions determined in accordance with Section 12.8.2.1 of ASCE 7, is less than 0.8 T_s determined in accordance with Section 11.4.5 of ASCE 7.~~
2. ~~In each of the two orthogonal directions, the fundamental period of the structure used to calculate the story drift is less than T_s.~~
3. ~~Equation 12.8-2 of ASCE 7 is used to determine the seismic response coefficient, C_s~~
4. ~~The diaphragms are rigid as defined in Section 12.3.1 of ASCE 7 or, for diaphragms that are flexible, the distances between vertical elements of the seismic-force-resisting system do not exceed 40 feet (12 192 mm).~~

(Renumber remaining sections)

Reason: In light of the changes to the ICC code development process and schedule, this code change is submitted as a possible last resort to avoid retention in the 2012 IBC of superseded earthquake ground motion maps and Seismic Design Category (SDC) provisions. The code change submittal schedule has made it problematic that the updated earthquake design maps developed for the 2009 *NEHRP Recommended Seismic Provisions* can be successfully moved through the full ASCE 7 consensus process in time for consideration for the 2012 IBC. Although the FEMA/BSSC Code Resource Support Committee (CRSC) would prefer that ground motion maps remain in the IBC, it has concluded that deletion of the maps from the code is a better approach than retaining maps that have been superseded. A companion FEMA/BSSC CRSC code change proposes the updated ground motion maps as currently being considered in the ASCE 7 process. It is our intention to withdrawal this proposal if ASCE 7-10 and the updated ground motion maps are approved for adoption into the IBC.

This code change proposes deletion of the earthquake ground motion maps and the associated SDC provisions, citing instead ASCE 7 for maps and SDC assignments. The definition of Seismic Design Category is retained as this term is used in Section 1613.1. The definition wording used is taken from ASCE 7-05 but the wording will be updated if needed to reflect ASCE 7-10 revisions. The definition of mechanical systems is retained. The balance of Section 1613.2 definitions are deleted as they are terms no longer used in the IBC.

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

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

ICCFILENAME: WINKEL-COBEEN-S1-1613.1

S96–09/10
1613.5.1, Figures 1613.5(1) - 1613.5(14)

Proponent: Jim Rossberg, Structural Engineering Institute of ASCE, representing self

Revise as follows:

1613.5.1 Mapped acceleration parameters. The parameters S_S and S_1 shall be determined from the 0.2 and 1 second spectral response accelerations shown on Figures ~~1613.5(1) through 1613.5(14)~~ 1613.5(1) and 1613.5(2). Where S_1, 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.

Delete Figures 1613.5(1) through 1613.5(14) and substitute as follows:

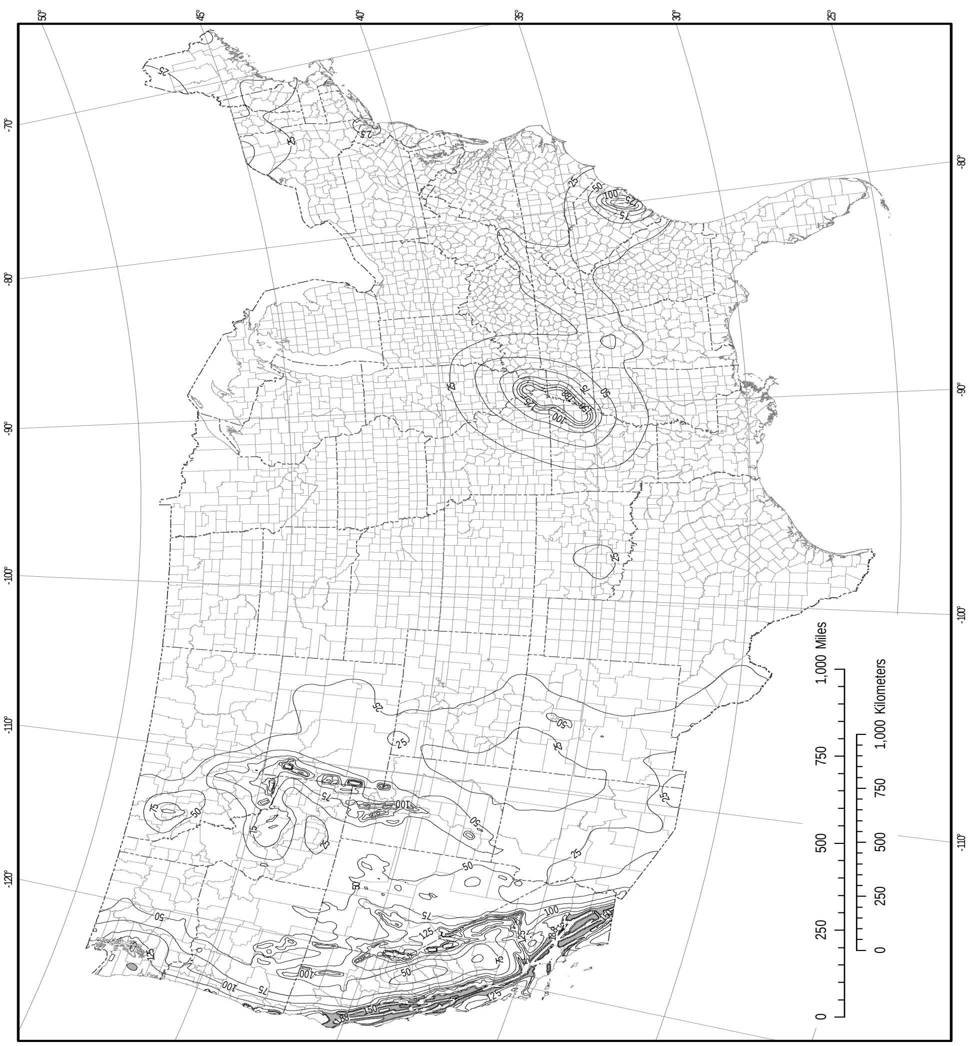
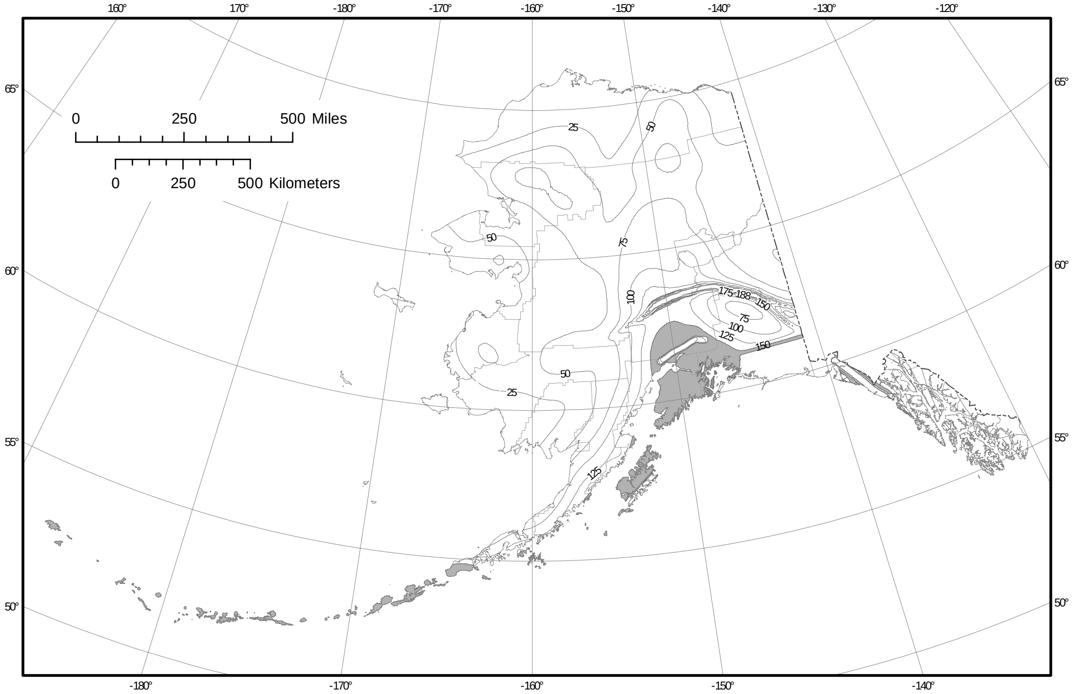
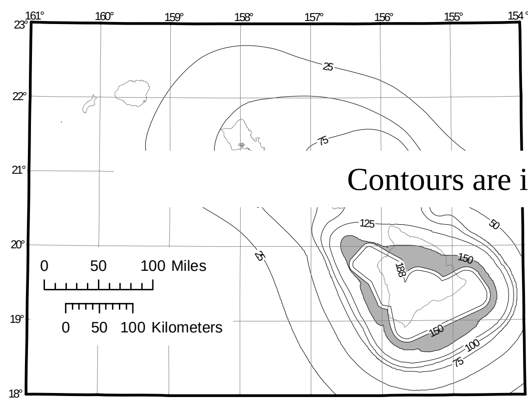


Figure 1613.5(1) Maximum Considered Earthquake (MCE) Ground Motion of 0.2 s Spectral Response Acceleration (5% of Critical Damping), Site Class B

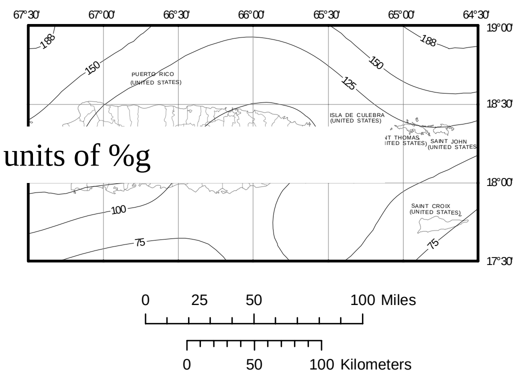


Areas with a constant acceleration value of 150% g

Contours are in units of %g



Contours are in units of %g



Notes:

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 2008 USGS National Seismic Hazard Maps 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)be used to determine the mapped value for a specified location.

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

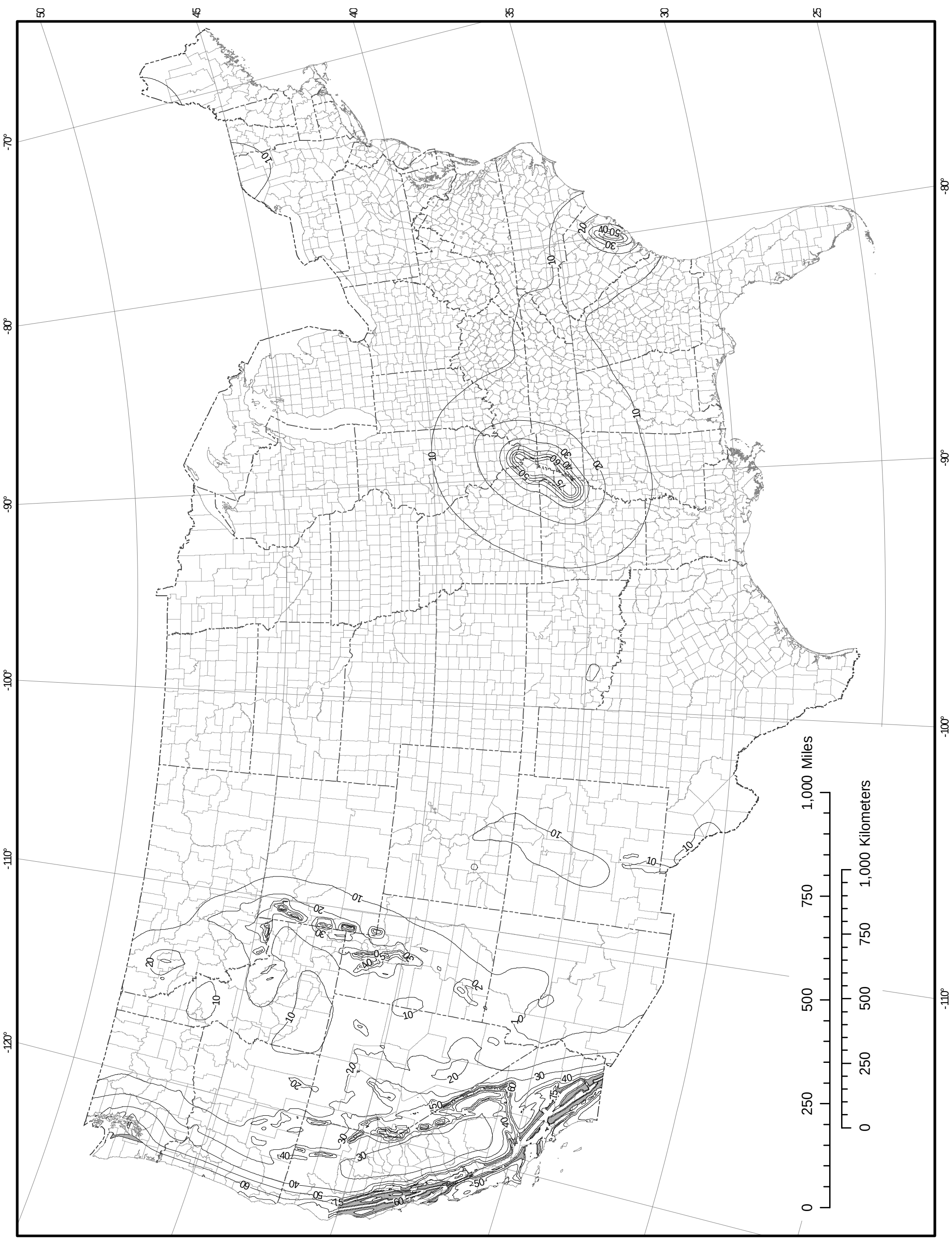
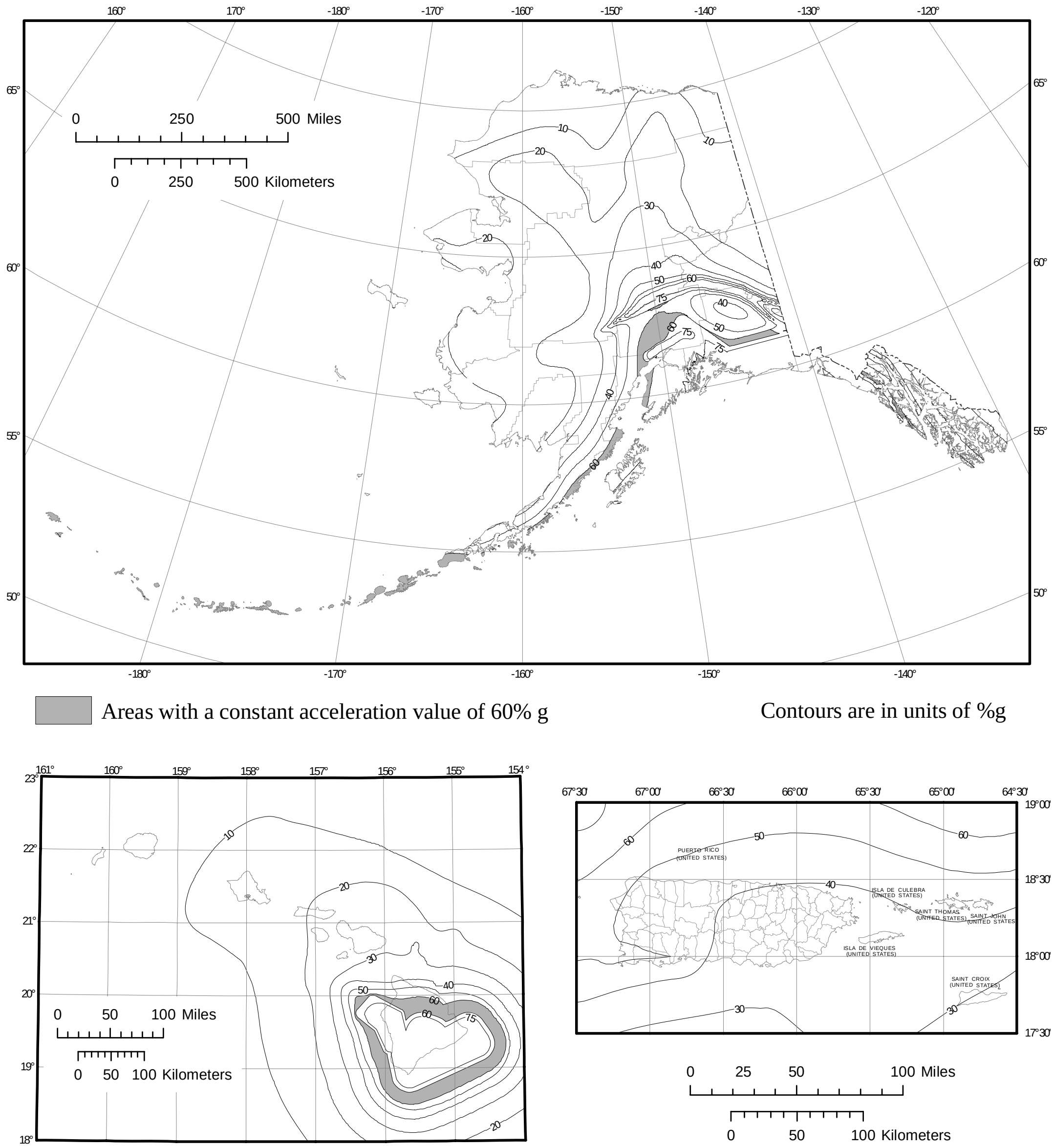


Figure 1613.5(2) Maximum Considered Earthquake (MCE) Ground Motion of 1 s Spectral Response Acceleration (5% of Critical Damping), Site Class B



Notes:

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 s spectral response acceleration. As such, they are different from those on the uniform-hazard 2008 USGS National Seismic Hazard Maps 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) be used to determine the mapped value for a specified location.

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: The purpose of this proposal is to make the 2012 edition of the IBC consistent with the 2010 edition of ASCE 7. The 2010 edition of ASCE 7 adopted new spectral response seismic design maps and maps of the transition period for the long-period portion of a design response spectrum. These maps were prepared by the United States Geological Survey (USGS) through the support and encouragement of the Federal Emergency Management Agency (FEMA) and in collaboration with the ASCE 7 Seismic Subcommittee and the Building Seismic Safety Council (BSSC) Seismic Design Procedures Reassessment Group. The maps are being balloted upon for their adoption into the 2010 edition of ASCE 7. In preparing these maps, the USGS updated their seismic hazard data, incorporating new information on earthquake sources and ground motion prediction equations, such as the new Next Generation Attenuation (NGA) relations.

Another important change incorporated in these maps is the adoption of a uniform risk, as opposed to uniform hazard basis, eliminating inequities in the treatment of different regions of the conterminous U.S. inherent in the prior generation of maps. The affect of this new basis is to modestly reduce design ground motions in much of the United States, consistent with present scientific understanding of the risk associated with structures constructed in these regions. The new maps will result in a modst reduction in the cost of construction for seismic resistance in most parts of the U.S.

The USGS has also developed a companion software program that calculates spectral values for a specific site based on a site's longitude, latitude, and site soil classification. The calculated values are based on the data used to prepare these maps. The spectral values may be adjusted for Site Class effects using the Site Classifications Procedure contained in this section as well as in ASCE 7-10. The software program should be used for establishing spectral values for design because the maps proposed for adoption herein, and those found in ASCE 7, are at too large a scale to provide accurate spectral values for most sites. The software program be accessed at the USGS Web site at http://earthquake.usgs.gov/designmaps

These 2008 maps supersede versions released in 1996 and 2002. Updating the maps involved interactions with hundreds of scientists and engineers at regional and topical workshops. USGS also solicited advice from working groups, expert panels, State geological surveys, Federal agencies, and hazard experts from industry and academia. The Pacific Earthquake Engineering Research Center developed new "Next Generation Attenuation" (NGA) crustal ground-motion models; the Working Group on California Earthquake Probabilities revised the California earthquake rate model; the Western States Seismic Policy Council submitted recommendations for the Intermountain West; and three expert panels were assembled to provide advice on best available science.

The most significant changes to the 2008 maps fall into two categories, as follows:

1. Changes to earthquake source and occurrence rate models:
   In California, the source model was updated to account for new scientific information on faults. For example, models for the southern San Andreas Fault System were modified to incorporate new geologic data. The source model was also modified to better match the historical rate of magnitude 6.5 to 7 earthquakes.
   The Cascadia Subduction Zone lying offshore of northern California, Oregon, and Washington was modeled using a distribution of large earthquakes between magnitude 8 and 9. Additional weight was given to the possibility for a catastrophic magnitude-9 earthquake that ruptures, on average, every 500 years from northern California to Washington, compared to a model that allows for smaller ruptures.
   The Wasatch fault in Utah was modeled to include the possibility of rupture from magnitude 7.4 earthquakes on the fault.
   Fault steepness estimates were modified based on global observations of normal faults.
   Several new faults were included or revised in the Pacific Northwest, California, and the Intermountain West regions.
   The New Madrid Seismic Zone in the Central U.S. was revised to include updated fault geometry and earthquake information. In addition, the model was adjusted to include the possibility of several large earthquakes taking place within a few years or less, similar to the earthquake sequence of 1811–1812.
   Source models for the region near Charleston, S.C., have been modified to include offshore faults that are thought to be capable of generating earthquakes.
   A broader range of earthquake magnitudes was used for the Central and Eastern U.S.
   Earthquake catalogs and seismicity parameters were updated.

2. Changes to models of ground shaking (that show how ground motion decays with distance from an earthquake's source) for different parts of the U.S., based on new published studies:
   New NGA ground-motion prediction models developed by the Pacific Earthquake Engineering Research Center were adopted for crustal earthquakes beneath the Western U.S. These new models use shaking records from 173 global shallow crustal earthquakes to better constrain ground motion in western States.
   Several new and updated ground-shaking models for earthquakes in the Central and Eastern U.S. were implemented in the maps. One of the new ground-shaking models accounts for the possibility that ground motion decays more rapidly from the earthquake source than was previously considered.
   New ground-motion models were applied for earthquake sources along the Cascadia Subduction Zone.

The new National Seismic Hazard Maps show, with some exceptions, similar or lower ground motion compared with the 2002 edition. For example, ground motion in the Central and Eastern U.S. has been generally lowered by about 10–25 percent due to the modifications of the ground-motion models. Ground motion in the Western U.S. is as much as 30 percent lower for shaking caused by long-period (1-second) seismic waves, and ground motion is similar (within 10–20 percent) for shaking caused by short-period (0.2-second) waves.

The new 2008 maps represent the best available science as determined by the USGS from an extensive information-gathering and review process. Changes will be made in future versions of the maps as new information on earthquake sources and resulting ground motion is gathered and processed.

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

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

ICCFILENAME: ROSSBERG-S11-1613.5.1

S97–09/10

1613.5.1, Figure 1613.5(1) - Figure 1613.5(14); IRC Figure R301.2(2)

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

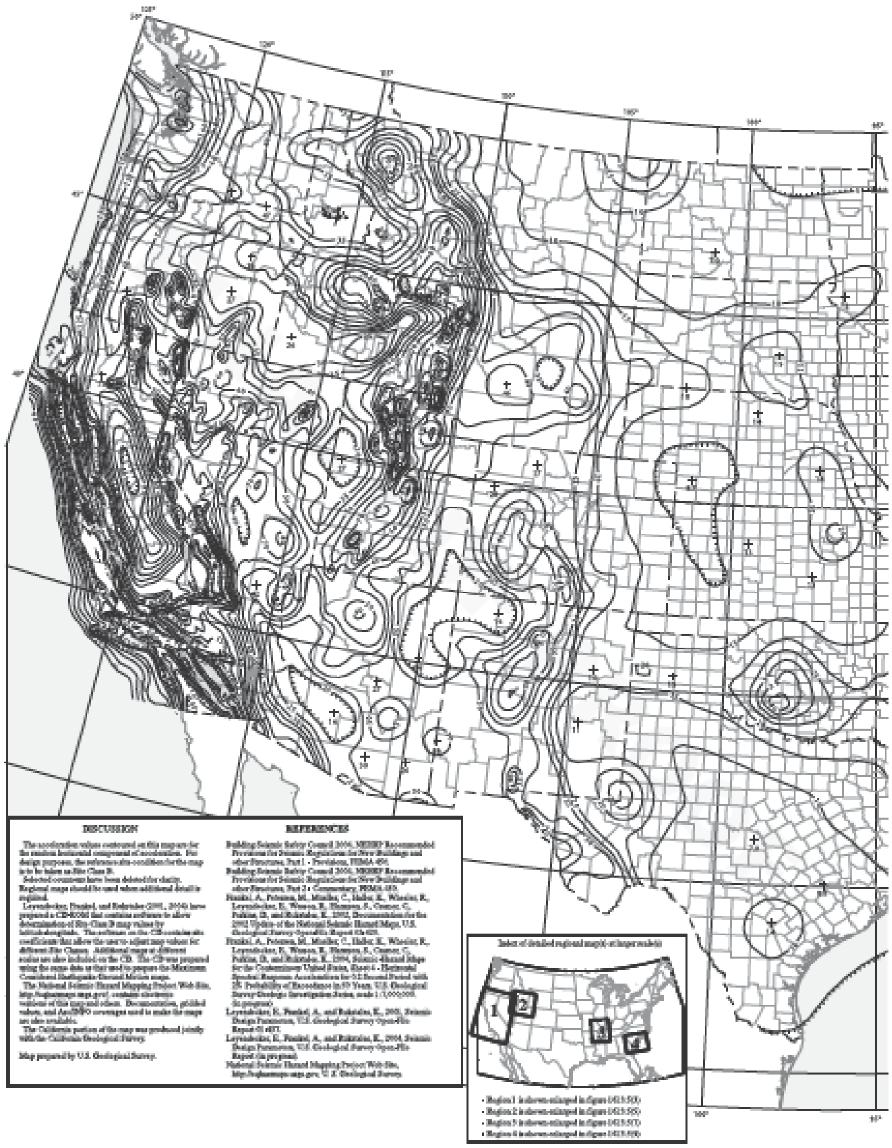
THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC-STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

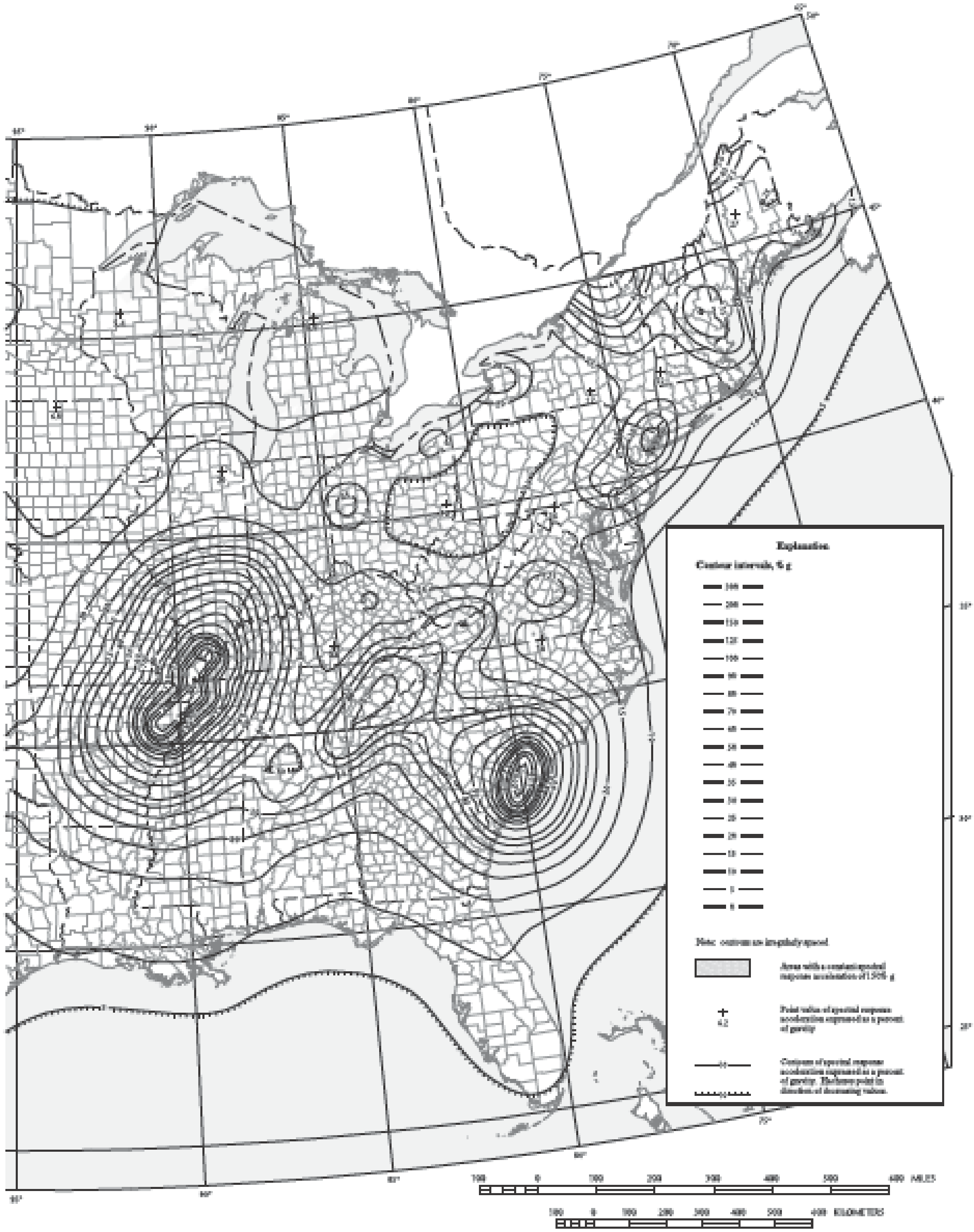
1613.5.1 Mapped Acceleration Parameters. The parameters S_S and S_1 shall be determined from the 0.2 and 1 s spectral response accelerations shown on Figures 1613.5(1) and 1613.5(2) ~~through 1613.5(14), respectively~~. Where S_1 is less than or equal to 0.04 and S_S is less than or equal to 0.15, the 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***~~

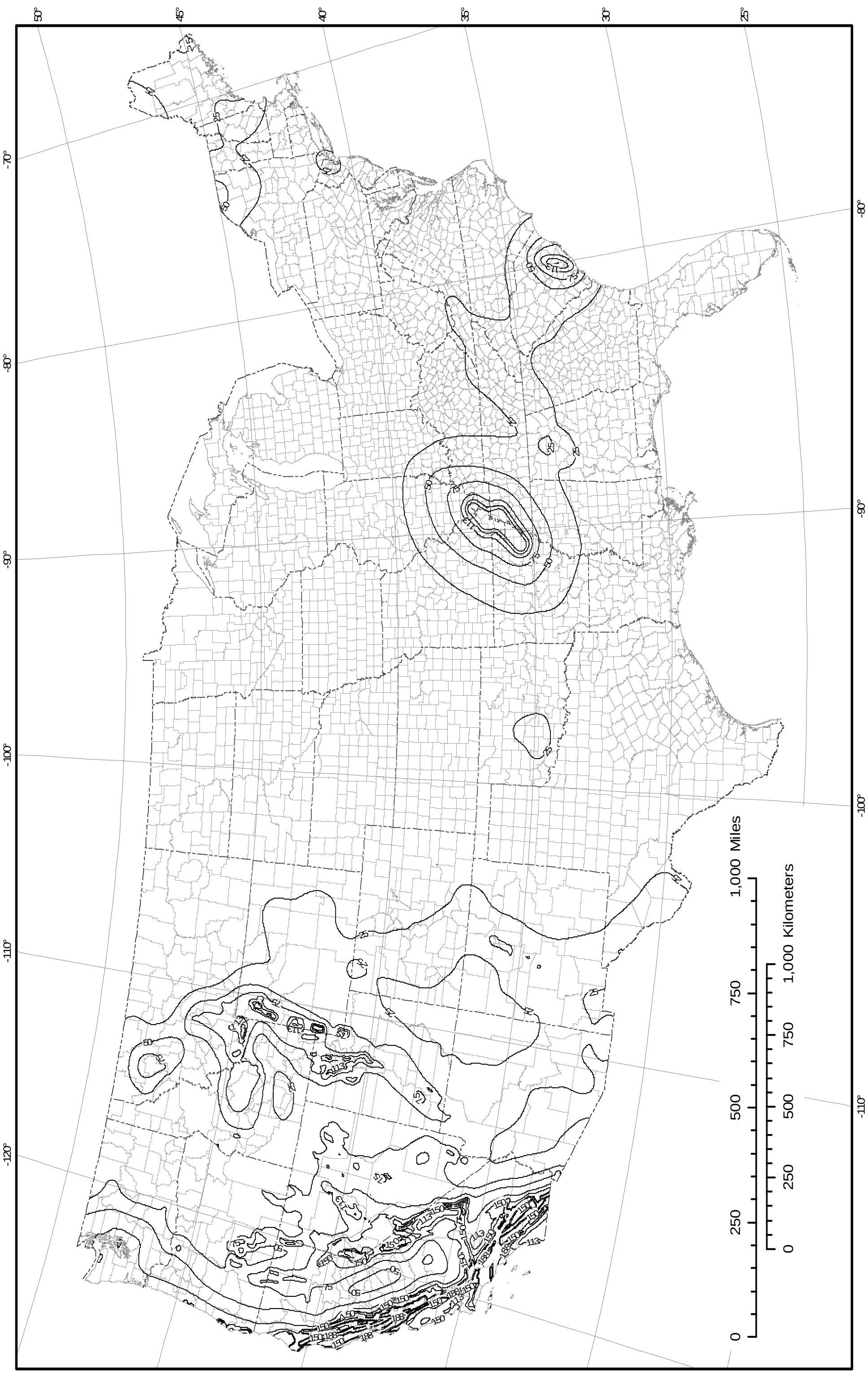
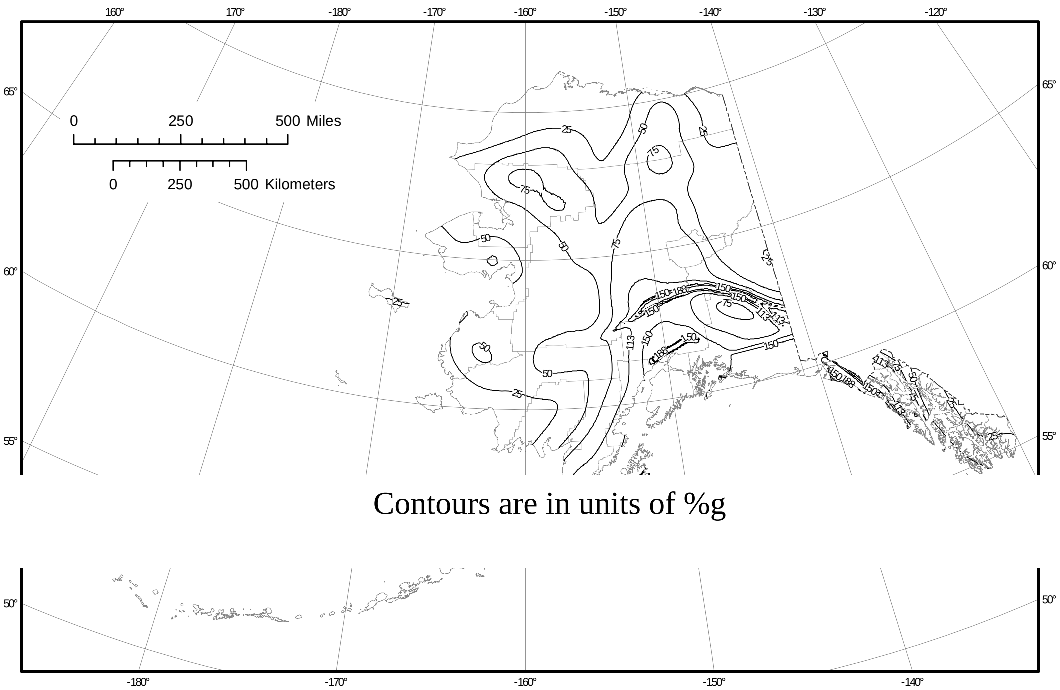
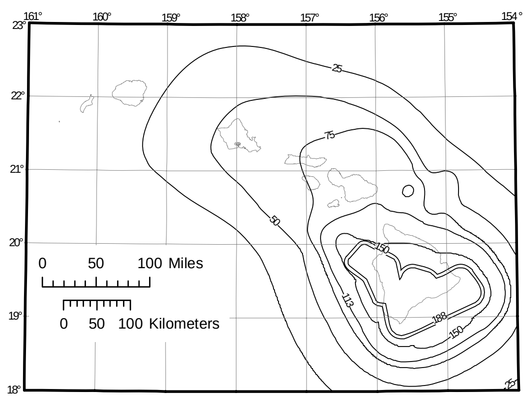
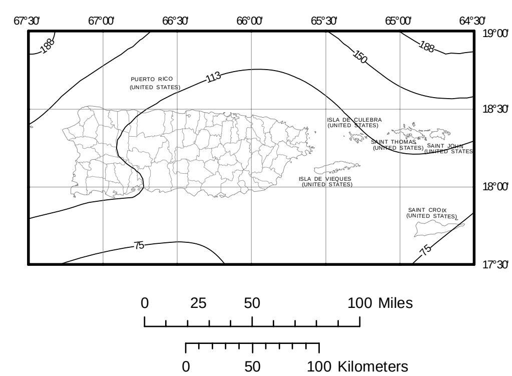


FIGURE 1613.5(1)
MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION OF 0.2 S SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B







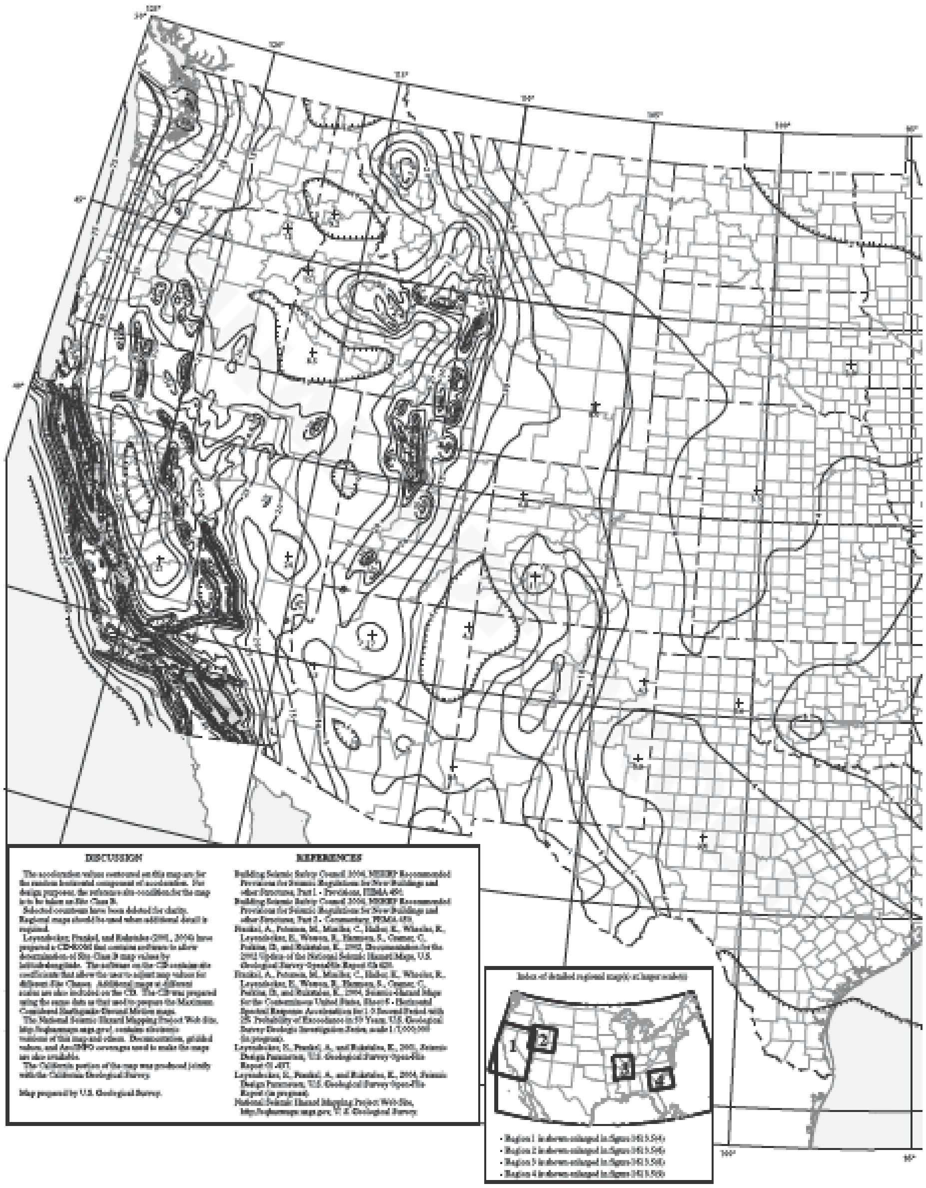
Notes:

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.

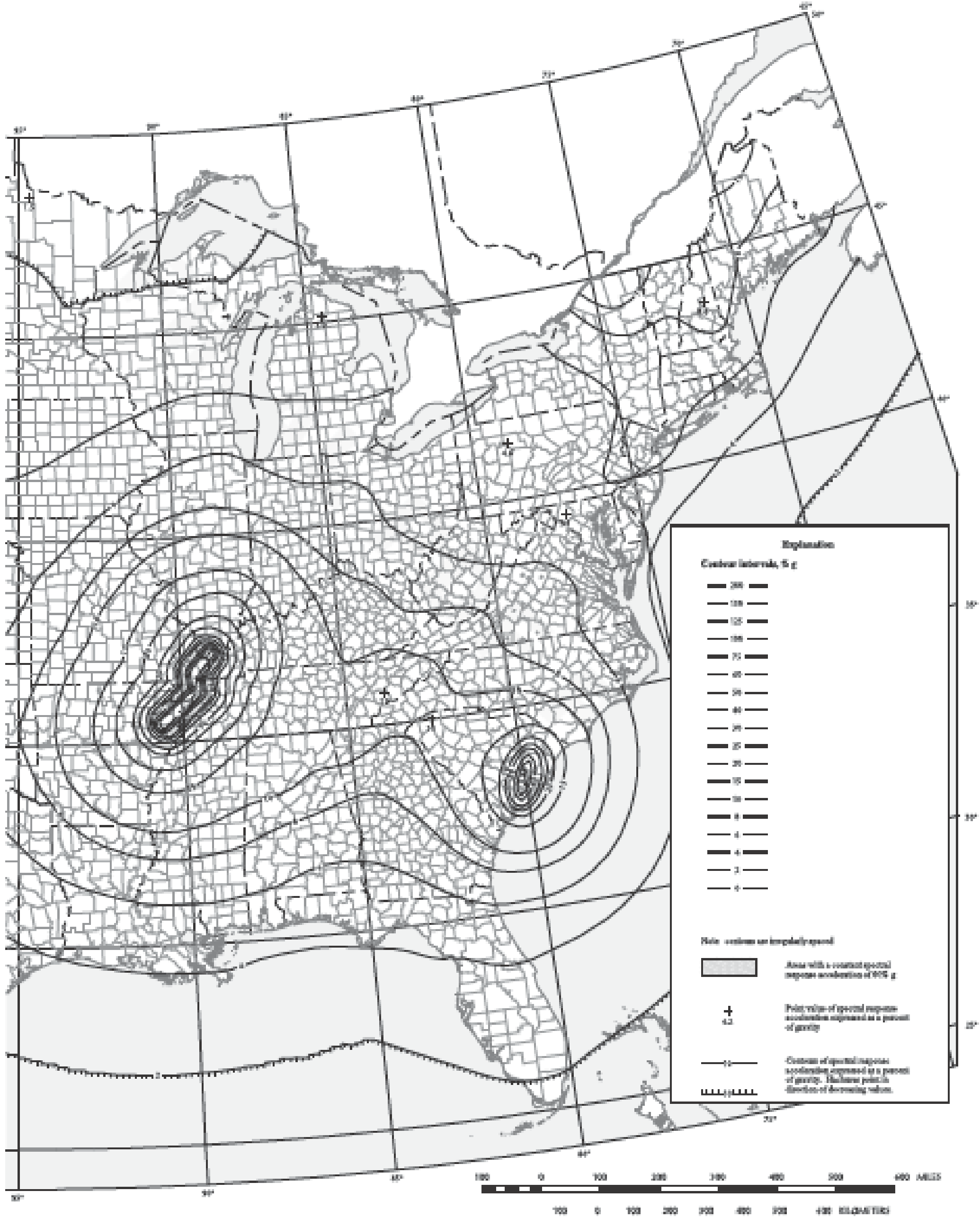
Ground motion values contoured on these maps incorporate risk-targeted and deterministic ground motions and a factor of 1.1 for the maximum direction of 0.2 s spectral response acceleration. As such, they are different from those on the uniform-hazard-based 2008 USGS National Seismic Hazard Maps posted 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**~~

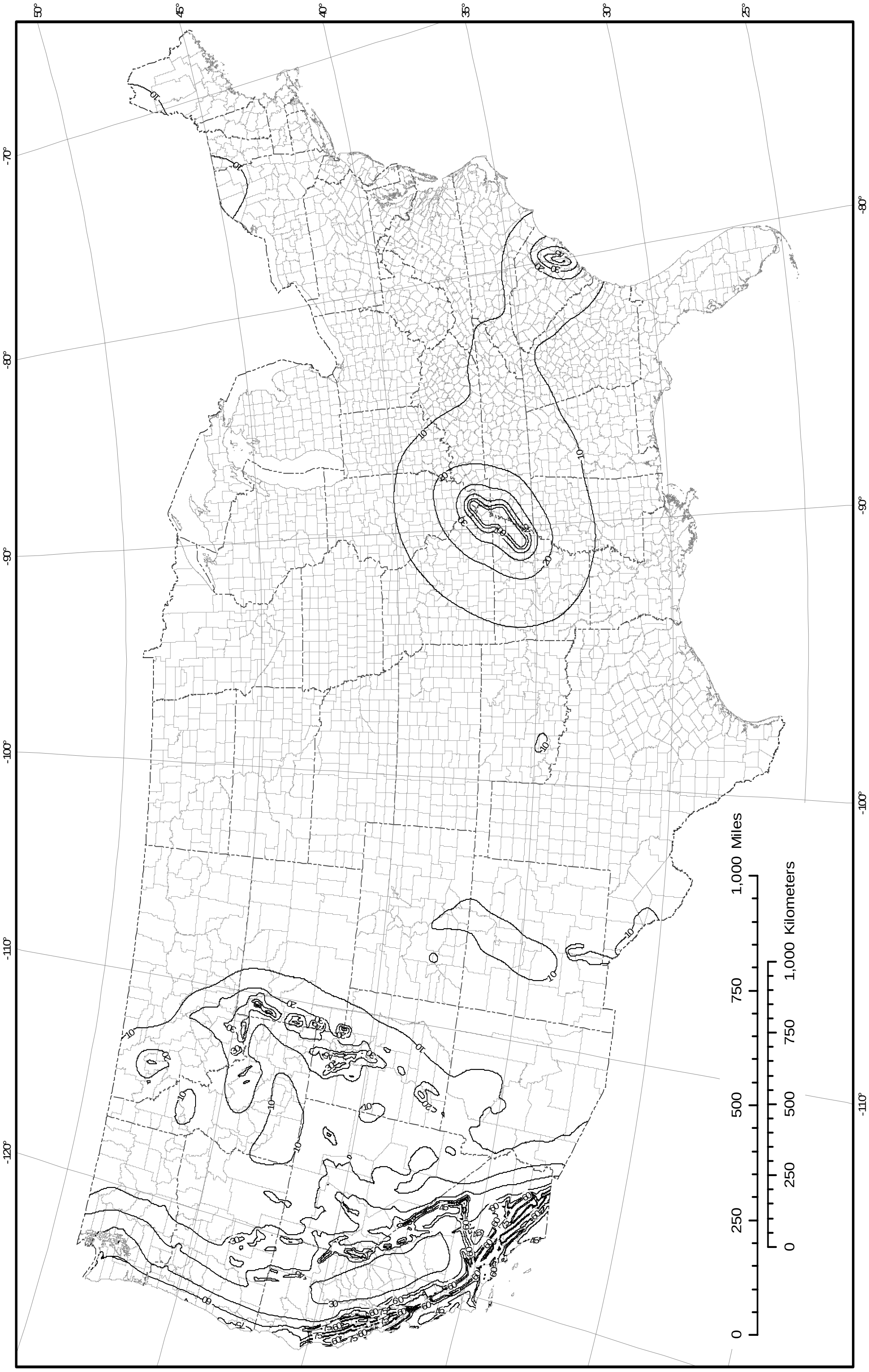
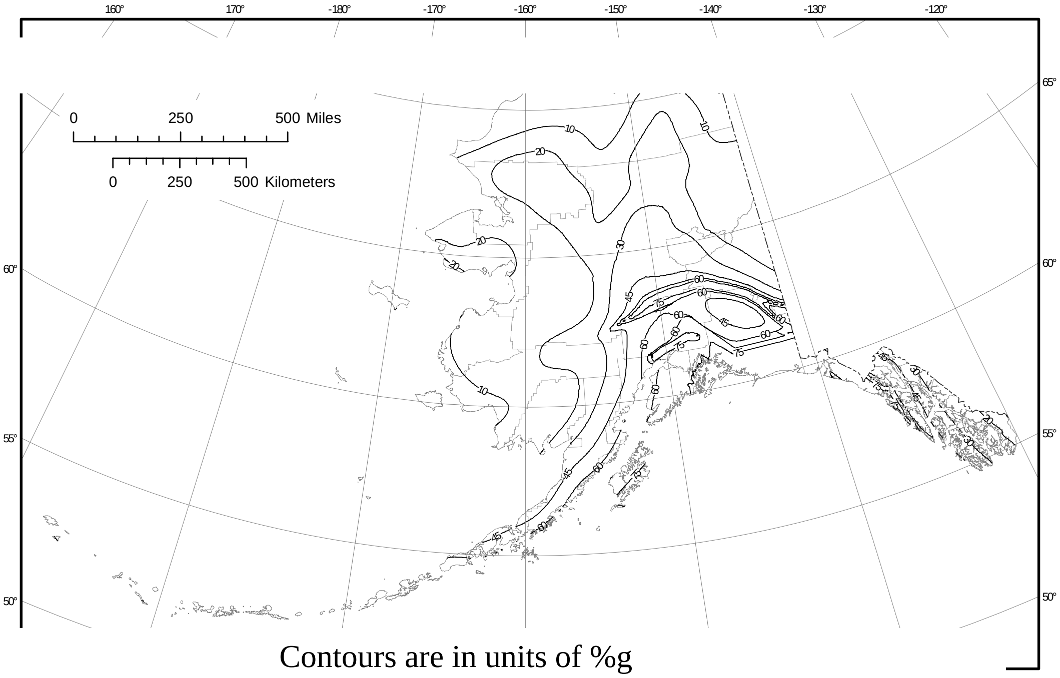
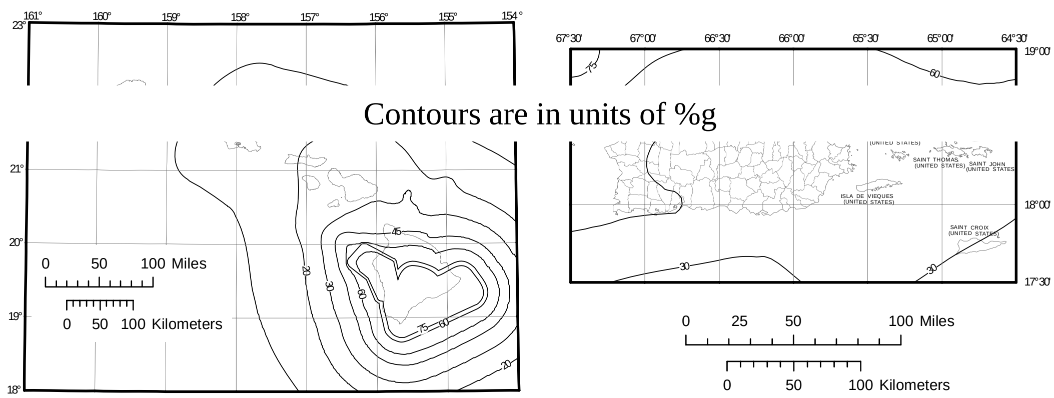


FIGURE 1613.5(2)
MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION OF 1 S SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



Contours are in units of %g



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 1.3 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.

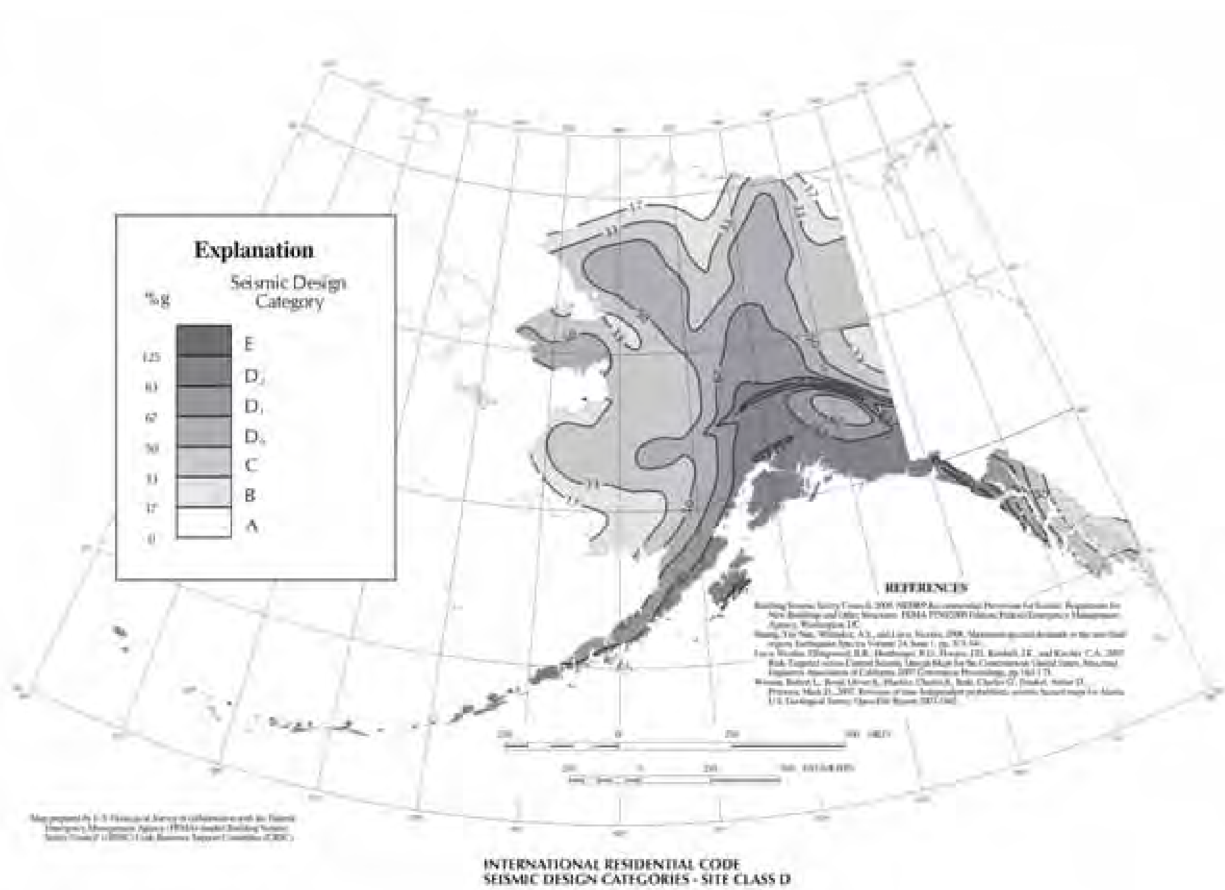
FIGURE 1613.5(2) (CONTINUED)
MAXIMUM CONSIDERED EARTHQUAKE (MCE) GROUND MOTION OF 1 S SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

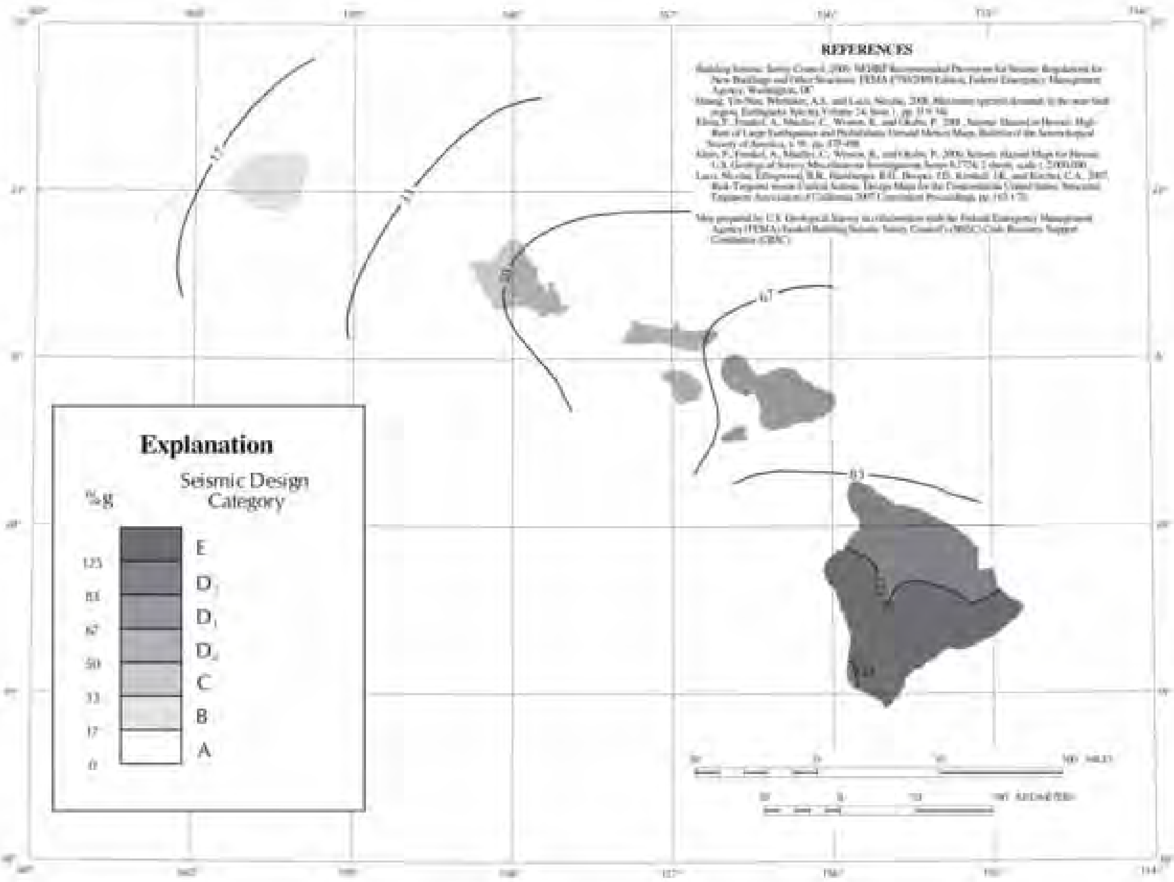
Delete Figures 1613.5(3) through 1613.5(14) without substitution.

PART II – IRC BUILDING/ENERGY

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

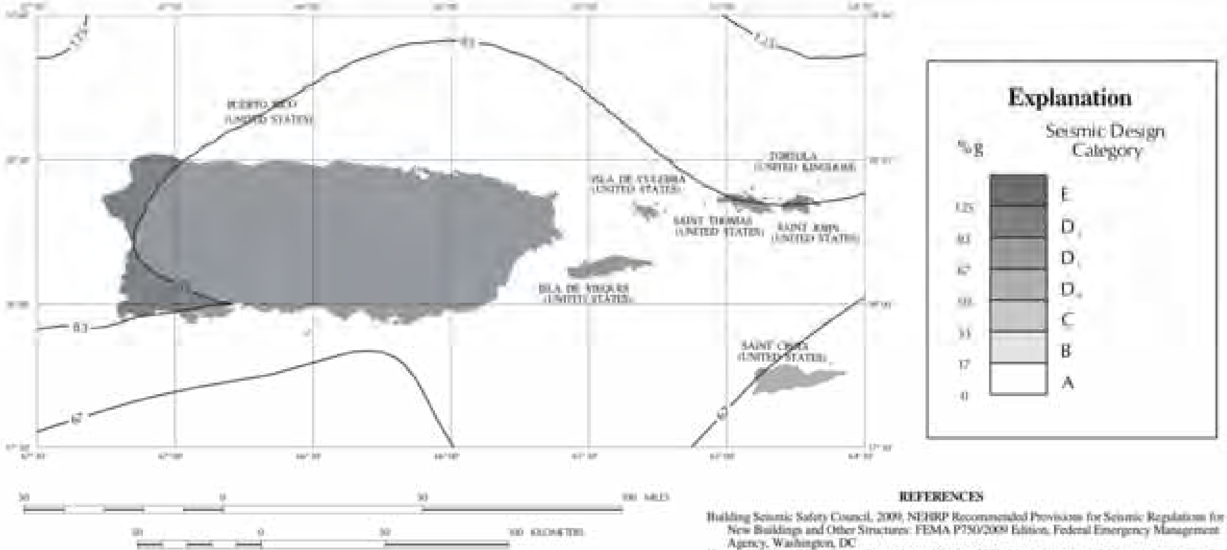
FIGURE R301.2(2)
SEISMIC DESIGN CATEGORIES -- SITE CLASS D





INTERNATIONAL RESIDENTIAL CODE
SEISMIC DESIGN CATEGORIES - SITE CLASS D

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



REFERENCES

Building Seismic Safety Council, 2009, NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures: FEMA P750/2009 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., Frankel, A., Petersen, M., and Leyendecker, E., 2003, Documentation for 2003 USGS Seismic Hazard Maps for Puerto Rico and the U.S. Virgin Islands, U.S. Geological Survey Open-File Report 03-379.

Map prepared by U.S. Geological Survey in collaboration with the Federal Emergency Management Agency (FEMA)-funded Building Seismic Safety Council's (BSSC) Code Resource Support Committee (CRSC).

INTERNATIONAL RESIDENTIAL CODE
SEISMIC DESIGN CATEGORIES - SITE CLASS D

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

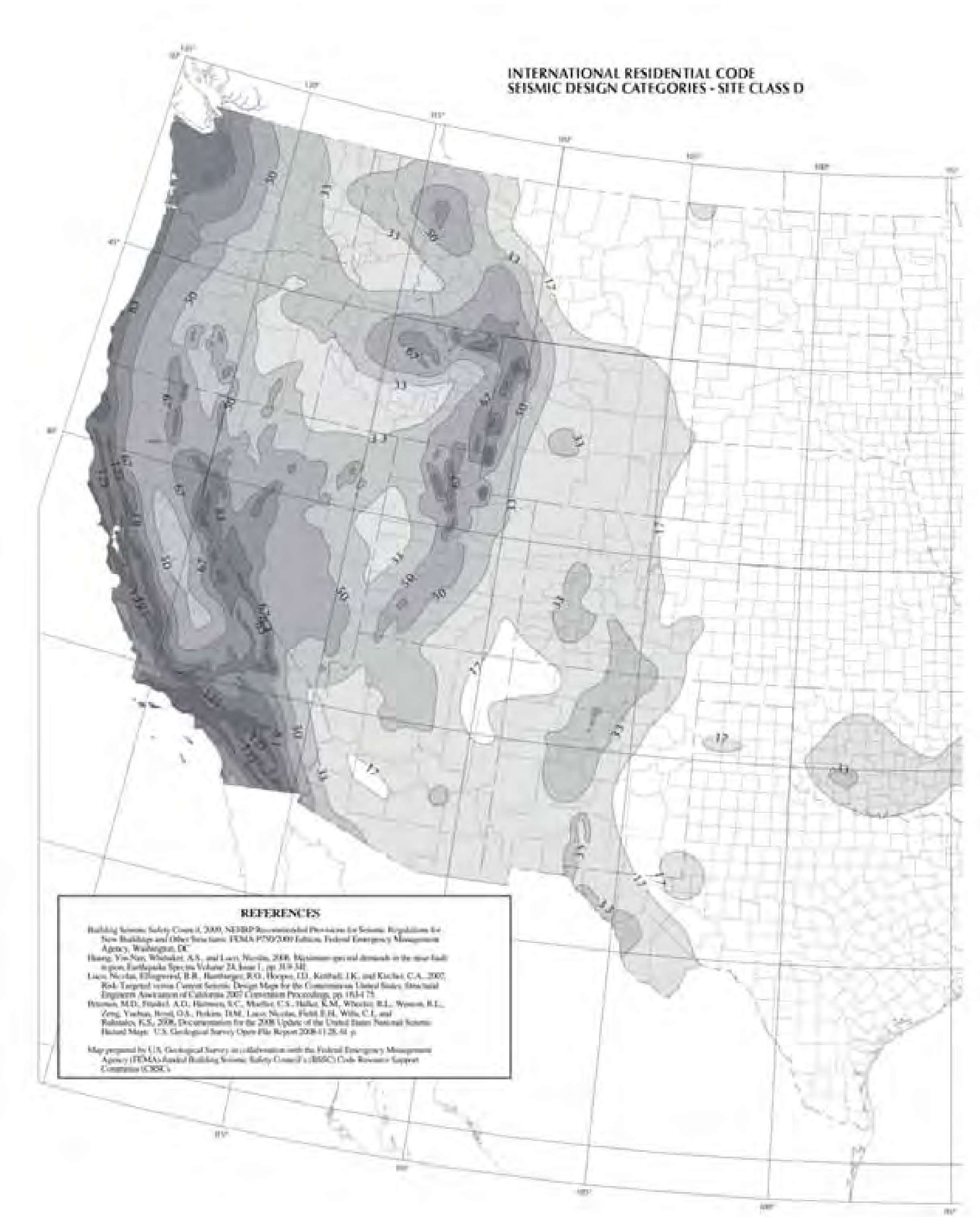


FIGURE R301.2(2) -- continued
SEISMIC DESIGN CATEGORIES -- SITE CLASS D

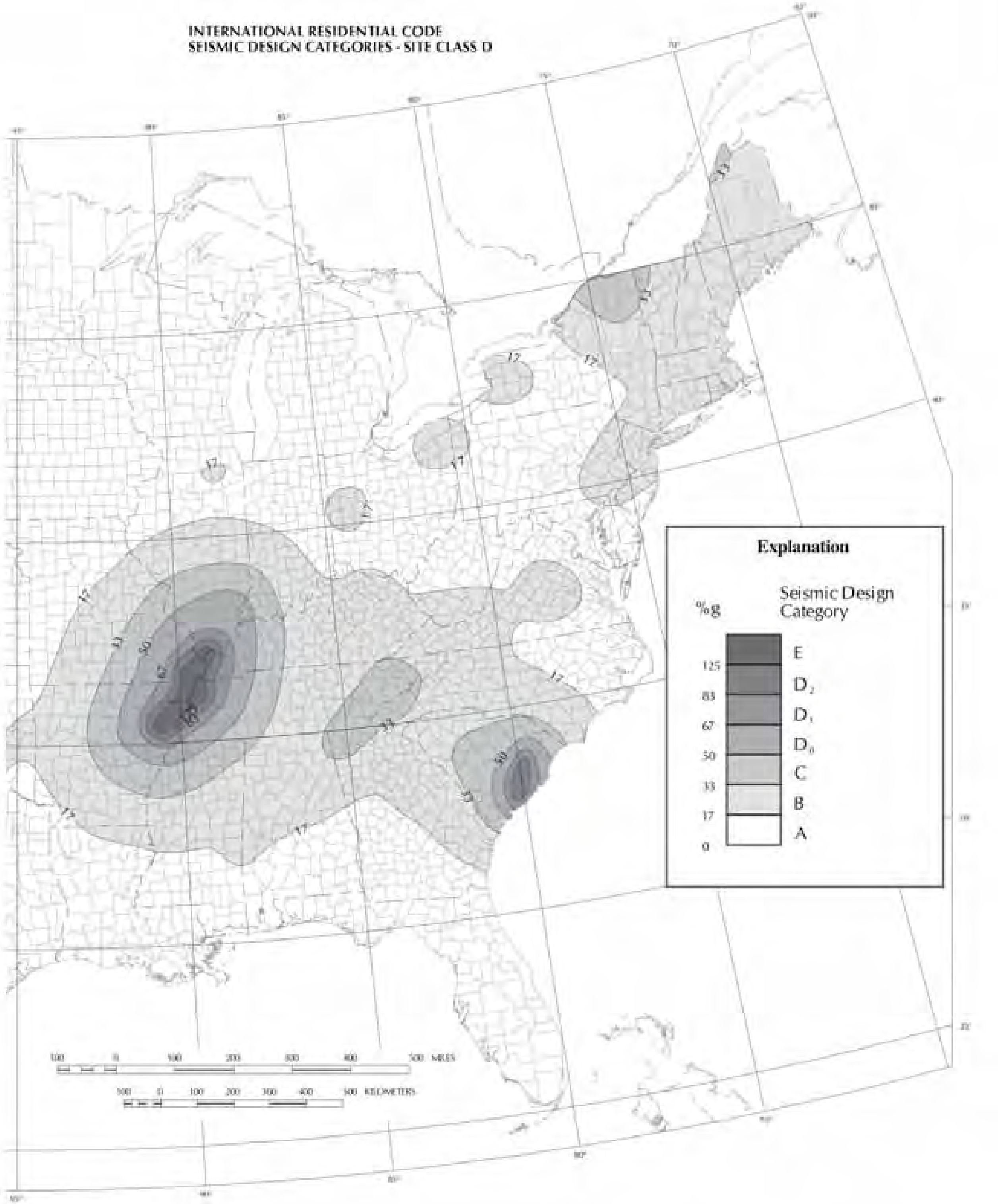


FIGURE R301.2(2) -- continued
SEISMIC DESIGN CATEGORIES -- SITE CLASS D

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 to 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.*

PART II- 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: (IBC) The new maps may lower costs in some locations but may increase them in others. (IRC) This proposal will not increase the cost of construction and will reduce the cost in some regions.

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: WINKEL-COBEEN-S2-1613.5.1

S98–09/10

1613.5.2, 1613.5.5, 1613.5.5.1, Table 1613.5.2, Table 1613.5.5

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

1. Revise as follows:

1613.5.2 Site class definitions. Based on the site soil properties, the site shall be classified as ~~either~~ Site Class A, B, C, D, E or F in accordance with ~~Table 1613.5.2~~ Chapter 20 of ASCE 7. ~~When~~ Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be used unless the building official or geotechnical data determines ~~that~~ Site Class E or F soils ~~is likely to be~~ are present at the site.

2. Delete without substitution:

~~TABLE 1613.5.2~~
~~SITE CLASS DEFINITIONS~~

~~SITE CLASS~~	~~SOIL PROFILE NAME~~	~~AVERAGE PROPERTIES IN TOP 100 feet, SEE SECTION 1613.5.5~~		
		~~Soil shear wave velocity, $\bar{v}_s$, (ft/s)~~	~~Standard penetration resistance, $\bar{N}$~~	~~Soil undrained shear strength, $\bar{s}_u$, (psf)~~
~~A~~	~~Hard rock~~	~~$\bar{v}_s > 5{,}000$~~	~~N/A~~	~~N/A~~
~~B~~	~~Rock~~	~~$2{,}500 < \bar{v}_s \leq 5{,}000$~~	~~N/A~~	~~N/A~~
~~C~~	~~Very dense soil and soft rock~~	~~$1{,}200 < \bar{v}_s \leq 2{,}500$~~	~~$\bar{N} > 50$~~	~~$\bar{s}_u \geq 2{,}000$~~
~~D~~	~~Stiff soil profile~~	~~$600 \leq \bar{v}_s \leq 1{,}200$~~	~~$15 \leq \bar{N} \leq 50$~~	~~$1{,}000 \leq \bar{s}_u \leq 2{,}000$~~
~~E~~	~~Soft soil profile~~	~~$\bar{v}_s < 600$~~	~~$\bar{N} < 15$~~	~~$\bar{s}_u < 1{,}000$~~
~~E~~	—	~~Any profile with more than 10 feet of soil having the following characteristics:~~ ~~1. Plasticity index $PI > 20$,~~ ~~2. Moisture content $w \geq 40\%$, and~~ ~~3. Undrained shear strength $\bar{s}_u < 500$ psf~~		
~~F~~	—	~~Any profile containing soils having one or more of the following characteristics:~~ ~~1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.~~ ~~2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil)~~ ~~3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$)~~ ~~4. Very thick soft/medium stiff clays ($H > 120$ feet)~~		

~~For SI: 1 foot = 304.8 mm, 1 square foot = 0.0929m2, 1 pound per square foot = 0.0479 kPa. N/A = Not applicable~~

~~**1613.5.5 Site classification for seismic design.** Site classification for Site Class C, D or E shall be determined from Table 1613.5.~~

~~The notations presented below apply to the upper 100 feet (30 480 mm) of the site profile. Profiles containing distinctly different soil and/or rock layers shall be subdivided into those layers designated by a number that ranges from 1 to *n* at the bottom where there is a total of *n* distinct layers in the upper 100 feet (30 480 mm). The symbol *i* then refers to any one of the layers between 1 and *n*.~~

~~where:~~

~~v_{si} = The shear wave velocity in feet per second (m/s).~~
~~d_i = The thickness of any layer between 0 and 100 feet (30 480 mm).~~

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

~~(Equation 16-40)~~

$$\sum_{i=1}^{n} d_i$$

~~N_i is the Standard Penetration Resistance (ASTM D 1586) not to exceed 100 blows/foot (328 blows/m) as directly measured in the field without corrections. When refusal is met for a rock layer, N_i shall be taken as 100 blows/foot (328 blows/m).~~

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

~~**(Equation 16-41)**~~

~~where N_i and d_i in Equation 16-41 are for cohesionless soil, cohesive soil and rock layers.~~

$$\bar{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}}$$

~~**(Equation 16-42)**~~

~~where:~~

$$\sum_{i=1}^{m} d_i = d_s$$

~~Use d_i and N_i for cohesionless soils layers only in Equation 16-42.~~

~~d_s = The total thickness of cohesionless soil layers in the top 100 feet (30 480 mm).~~

~~m = The number of cohesionless soil layers in the top 100 feet (30 480 mm).~~

~~s_{ui} = The undrained shear strength in psf (kPa), not to exceed 5,000 psf (240 kPa), ASTM D 2166 or D 2850.~~

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}}$$

~~**(Equation 16-43)**~~

~~where:~~

$$\sum_{i=1}^{k} d_i = d_c$$

~~d_c = The total thickness of cohesive soil layers in the top 100 feet (30 480 mm).~~

~~k = The number of cohesive soil layers in the top 100 feet (30 480 mm).~~

~~PI = The plasticity index, ASTM D 4318.~~

~~w = The moisture content in percent, ASTM D 2216.~~

~~Where a site does not qualify under the criteria for Site Class F, and there is a total thickness of soft clay greater than 10 feet (3,048 mm) where a soft clay layer is defined by: s_u < 500 psf (25 kPa), w ≥ 40 percent and PI > 20, it shall be classified as Site Class E.~~

~~The shear wave velocity for rock, Site Class B, shall be either measured on site or estimated by a geotechnical engineer or engineering geologist/seismologist for competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Site Class C.~~

~~The hard rock category, Site Class A, shall be supported by shear wave velocity measurements either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 100 feet (30 480 mm), surficial shear wave velocity measurements are permitted to be extrapolated to assess v_s.~~

~~The rock categories, Site Classes A and B, shall not be used if there is more than 10 feet (3048 mm) of soil between the rock surface and the bottom of the spread footing or mat foundation.~~

~~**TABLE 1613.5.5**~~
~~**SITE CLASSIFICATION** [a]~~

~~SITE CLASS~~	~~v_s~~	~~N or N_{ch}~~	~~s_u~~
~~E~~	~~< 600 ft/s~~	~~< 15~~	~~< 1,000 psf~~
~~D~~	~~600 to 1,200 ft/s~~	~~15 to 50~~	~~1,000 to 2,000 psf~~
~~C~~	~~1,200 to 2,500 ft/s~~	~~> 50~~	~~> 2,000~~

~~For SI: 1 foot per second = 304.8 mm per second, 1 pound per square foot = 0.0479 kN/m².~~

~~a. If the s_u method is used and the N_{CH} and s_u criteria differ, select the category with the softer soils (for example, use Site Class E instead of D).~~

~~**1613.5.5.1 Steps for classifying a site.**~~

~~1. Check for the four categories of Site Class F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific evaluation.~~
~~2. Check for the existence of a total thickness of soft clay > 10 feet (3048 mm) where a soft clay layer is defined by: s_u < 500 psf (25 kPa), $w \geq$ 40 percent, and PI > 20. If these criteria are satisfied, classify the site as Site Class E.~~
~~3. Categorize the site using one of the following three methods with $\bar{v}_s$, $\bar{N}$, and $\bar{s}_u$ computed in all cases as specified.~~
    ~~3.1. $\bar{v}_s$ for the top 100 feet (30 480 mm) ($\bar{v}_s$ method).~~
    ~~3.2. $\bar{N}$ for the top 100 feet (30 480 mm) ($\bar{N}$ method).~~
    ~~3.3. $\bar{N}_{ch}$ for cohesionless soil layers (PI < 20) in the top 100 feet (30 480 mm) and average, $\bar{s}_u$, for cohesive soil layers (PI > 20) in the top 100 feet (30 480mm) ($\bar{s}_u$ method).~~

Reason: The purpose for this proposal is to delete text from the IBC related to site class definitions and site classifications in favor of referencing the applicable text in ASCE 7. This will have the benefit of eliminating from the IBC text that is duplicated in ASCE 7 and may also conflict with the corresponding text in ASCE 7.

IBC Section 1613.5.2 relies on Table 16.13.5.2 for classifying sites by site class based on the types of soils present and their properties. In ASCE 7-05 (and 7-10), however, this is accomplished by Chapter 20. This chapter includes Table 20.3-1, which corresponds roughly to IBC Table 16.13.5.2, but also contains additional information not contained in the IBC table. This information is found in Section 20.3 of ASCE 7-05. In Section 20.3.1 for Site Class F, there is an exception to Item #1, which is not found in IBC Table 1613.5.2. Also in Section 20.3.1, there is a second condition for Item 4 (s_u < 1000psf), that is not found in IBC Table 1613.5.2. Sections 20.3.2 and 20.3.4 on Site Classes E and B, respectively, contain criteria for classifying a site by site class but this information is not found in IBC Table 1613.5.2. The text from Section 20.3.3 on Site Classes C, D and E is not found in the IBC. Finally, Table 20.3-1 of ASCE 7-05 lists Site Class E soil as "soft clay soil" and includes N and N_{ch} for penetration resistance but IBC Table 1613.5.2 lists Site Class E soil as "soft soil" and only includes N.

IBC Section 1613.5.2 requires classification of sites based on the types of soils present and their properties as Site Class A, B, C, D, E or F in accordance with Table 1613.5.2. Section 161.3.5.5, however, requires classification of sites for Site Class C, D or E to be determined from Table 1613.5.5, which is a simplified version of Table 1613.5.2. The charging text of Section 1613.5.5 appears to limit the provisions in the section to Site Classes C, D and E but there are paragraphs at the end of Section 1613.5.5 for Site Classes A and B.

The classification of sites into site classes based on soil properties demands the expertise of a registered design professional qualified to practice the profession of geotechnical engineering and that profession is not well served by duplicative and potentially conflicting provisions in the IBC and ASCE 7.

In IBC Section 1613.5.2, editorial changes are also made for consistency with Section 11.4.2 of ASCE 7-10, which is the source of the provisions in the section.

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

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

ICCFILENAME: Brazil-S12-1613.5.2

S99–09/10
1613.5.5, Chapter 35

Proponent: Ronald J. Ebelhar, HC NUTTING I A Terracon Company, representing self

1. Revise as follows:

1613.5.5 Site classification for seismic design. Site classification for Site Class C, D or E shall be determined from Table 1613.5.5. The notations presented below apply to the upper 100 feet (30 480 mm) of the site profile. Profiles containing distinctly different soil and/or rock layers shall be subdivided into those layers designated by a number that ranges from 1 to n at the bottom where there is a total of n distinct layers in the upper 100 feet (30 480 mm). The symbol i then refers to any one of the layers between 1 and n.

where:

v_{si} = The shear wave velocity in feet per second (m/s)<u>, in accordance with ASTM D 4428 or ASTM D 7400.</u>
d_i = The thickness of any layer between 0 and 100 feet (30 480 mm)

where:

$$v_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$ **(Equation 16-40)**

$$\sum_{i=1}^{n} d_i = 100 \text{ feet } (30\ 480\text{mm})$$

(Remainder of section unchanged)

2. Add new standard to Chapter 35 as follows:

ASTM

D 4428/D 4428M-07	Standard Test Methods for Crosshole Seismic Testing
D 7400-08	Standard Test Methods for Downhole Seismic Testing

Reason: There is currently no guidance in the IBC for example methods or standards to measure the shear wave velocity in the field. There is guidance for measuring the Standard Penetration Resistance and the undrained shear strength.

Cost Impact: The code change proposal may potentially have a limited impact on the cost of construction.

Analysis: A review of the standard(s) proposed for inclusion in the code, ACSE49-09, 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.

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

ICCFILENAME: EBELHAR-S1-1613.5.5

S100–09/10
1613.6.1, 1602.1

Proponent: Jim Rossberg, SEI of ASCE, representing self

1. Delete without substitution:

~~**1613.6.1 Assumption of flexible diaphragm**. Add the following text at the end of Section 12.3.1.1 of ASCE 7.~~

~~Diaphragms constructed of wood structural panels or untopped steel decking shall also be permitted to be idealized as flexible, provided all of the following conditions are met:~~

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

(Renumber subsequent sections)

2. Revise as follows:

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

Diaphragm flexible. A diaphragm is flexible for the purpose of distribution of story shear and torsional moment where so indicated in Section 12.3.1 of ASCE 7~~, as modified in Section 1613.6.1~~.

(Definitions not shown are unchanged)

Reason: This provision has been considered and approved by the Seismic Subcommittee of ASCE 7 for inclusion into the 2010 edition of ASCE 7 hence with the adoption of ASCE 7-10 by reference this provision becomes duplicative. As of the submission date of this code change, the ASCE 7 Standards Committee is completing the committee balloting portion of the 2010 edition of ASCE/SEI 7. The document is designated ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* and it is expected that it will be completed and available for purchase prior to the ICC Final Action Hearings in May of 2010 . Any person interested in obtaining a public comment copy of ASCE/SEI 7-10 may do so by contacting the proponent at jrossberg@asce.org.

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

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

ICCFILENAME: ROSSBERG-S10-1613.6.1

S101–09/10
1613.6.3

Proponent: Jim Rossberg, SEI of ASCE, representing self

Delete without substitution:

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

Reason: This provision has been considered and approved by the Seismic Subcommittee of ASCE 7 for inclusion into the 2010 edition of ASCE 7 hence with the adoption of ASCE 7-10 by reference this provision becomes duplicative. As of the submission date of this code change, the ASCE 7 Standards Committee is completing the committee balloting portion of the 2010 edition of ASCE/SEI 7. The document is designated ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* and it is expected that it will be completed and available for purchase prior to the ICC Final Action Hearings in May of 2010 . Any person interested in obtaining a public comment copy of ASCE/SEI 7-10 may do so by contacting the proponent at jrossberg@asce.org.

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

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

ICCFILENAME: ROSSBERG-S9-1613.6.3

S102–09/10
1613.6.4

Proponent: Philip Brazil, Reid Middleton, Inc. representing self
Jason Thompson, National Concrete Masonry Association, representing the Masonry Alliance for Codes and Standards, Phil Samblanet, The Masonry Society
Jim Rossberg, SEI of ASCE, representing self

Delete without substitution:

~~**1613.6.4 Autoclaved aerated concrete (AAC) masonry shear wall design coefficients and system limitations**. Add the following text at the end of Section 12.2.1 of ASCE 7:~~

~~For ordinary reinforced AAC masonry shear walls used in the seismic force-resisting system of structures, the response modification factor, R, shall be permitted to be taken as 2, the deflection application factor, C_d, shall be permitted to be taken as 2, and the system overstrength factor, Ω_o, shall be permitted to be taken as 2-1/2. Ordinary reinforced AAC masonry shear walls shall not be limited in height for buildings assigned to Seismic Design Category B , shall be limited in height to 35 feet (10 668 mm) for buildings assigned to Seismic Design Category C, and are not permitted for buildings assigned to Seismic Design Categories D, E and F.~~

~~For ordinary plain (unreinforced) AAC masonry shear walls used in the seismic-force-resisting system of structures, the response modification factor, R, shall be permitted to be taken as 1½, the deflection application factor, C_d, shall be permitted to be taken as 1-1/2, and the system overstrength factor, Ω_o, shall be permitted to be taken as 2-1/2. Ordinary plain (unreinforced) AAC masonry shear walls shall not be limited in height for buildings assigned to Seismic Design Category B and are not permitted for buildings assigned to Seismic Design Categories C, D, E and F.~~

(Renumber remaining sections)

Reason:
(Brazil)- The purpose for this proposal is to delete a revision to ASCE 7-05 that will no longer be needed because the revision will have been incorporated into the 2010 edition of ASCE 7. This is being accomplished by ASCE 7 Proposal TC-5-CH14-12, which is being balloted by the Main Committee (Item #12 of the Eighth Main Committee Ballot on Seismic Provisions). It is expected that the Main Committee will approve the proposal.
(Thompson-Samblanet) -This modification to ASCE 7-05 was added because ASCE 7 did not have time to adequately consider seismic design coefficients for Autoclaved Aerated Concrete masonry during its last revision cycle. Since that time, the Building Seismic Safety Council has approved similar seismic design coefficients and requirements for AAC masonry. Revisions to the 2010 edition of ASCE 7 that are technically identical to the modifications proposed for deletion here are under review by the ASCE 7 Committee. If adopted into ASCE 7-10, these modifications will no longer be necessary and as such are proposed for deletion from the IBC.
(Rossberg)- This provision is being considered for approval by the Seismic Subcommittee of ASCE 7 for inclusion into the 2010 edition of ASCE 7 hence with the adoption of ASCE 7-10 by reference this provision becomes duplicative. As of the submission date of this code change, the ASCE 7 Standards Committee is completing the committee balloting portion of the 2010 edition of ASCE/SEI 7. The document is designated ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* and it is expected that it will be completed and available for purchase prior to the ICC Final Action Hearings in May of 2010 . Any person interested in obtaining a public comment copy of ASCE/SEI 7-10 may do so by contacting the proponent at jrossberg@asce.org.

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

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

ICCFILENAME: BRAZIL-ROSSBERG-THOMPSON-SAMBLANET-S1-1613.6.4

S103–09/10
1613.6.5

Proponent: Jim Rossberg, SEI of ASCE, representing self

Delete without substitution:

~~**1613.6.5 Seismic controls for elevators.** Seismic switches in accordance with Section 8.4.10 of ASME A17.1 shall be deemed to comply with Section 13.6.10.3 of ASCE 7.~~

Reason: This provision has been considered and approved by the Seismic Subcommittee of ASCE 7 for inclusion into the 2010 edition of ASCE 7 hence with the adoption of ASCE 7-10 by reference this provision becomes duplicative. As of the submission date of this code change, the ASCE 7 Standards Committee is completing the committee balloting portion of the 2010 edition of ASCE/SEI 7. The document is designated ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* and it is expected that it will be completed and available for purchase prior to the ICC Final Action Hearings in May of 2010 . Any person interested in obtaining a public comment copy of ASCE/SEI 7-10 may do so by contacting the proponent at jrossberg@asce.org.

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

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

ICCFILENAME: ROSSBERG-S7-1613.6.5

S104–09/10
1613.6.6

Proponent: Bonnie Manley, representing American Iron and Steel Institute,
Jim Rossberg, SEI of ASCE

Delete without substitution as follows:

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

> ~~**12.2.5.4 Increased building height limit for steel-braced frames, special steel plate shear walls and special reinforced concrete shear walls.** The height limits in Table 12.2-1 are permitted to be increased from 160 feet (48 768 mm) to 240 feet (75 152 mm) for structures assigned to *Seismic Design Category* D or E and from 100 feet (30 480 mm) to 160 feet (48 768 mm) for structures assigned to *Seismic Design Category* F that have steel-braced frames, special steel plate shear walls or special reinforced concrete cast-in-place shear walls and that meet both of the following requirements:~~

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

Reason:
(MANLEY) This section was added to the 2009 IBC (Proposal S94-07/08) to correct an oversight in the development of ASCE 7-05. The correction has been made to Section 12.2.5.4 of the 2010 edition of ASCE 7, so this modification is no longer necessary.
(ROSSBERG) This provision has been considered and approved by the Seismic Subcommittee of ASCE 7 for inclusion into the 2010 edition of ASCE 7 hence with the adoption of ASCE 7-10 by reference this provision becomes duplicative. As of the submission date of this code change, the ASCE 7 Standards Committee is completing the committee balloting portion of the 2010 edition of ASCE/SEI 7. The document is designated ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* and it is expected that it will be completed and available for purchase prior to the ICC Final Action Hearings in May of 2010 . Any person interested in obtaining a public comment copy of ASCE/SEI 7-10 may do so by contacting the proponent at jrossberg@asce.org.

Cost Impact:
(MANLEY)-There is no anticipated impact on the cost of construction.
(ROSSBERG)- The code change proposal will not increase the cost of construction.

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

ICCFILENAME: Manley- Rossberg-S12-1613.6.6

S105–09/10
1613.6.7

Proponent: Jim Rossberg, SEI of ASCE, representing self

Delete without substitution:

~~**1613.6.7 Minimum distance for building separation.** All buildings and structures shall be separated from adjoining structures. Separations shall allow for the maximum inelastic response displacement (δ_M). δ_M shall be determined at critical locations with consideration for both translational and torsional displacements of the structure using Equation 16-44.~~

$$\delta_M = \frac{C_d \delta_{max}}{I}$$ ~~**(Equation 16-44)**~~

~~Where~~

~~C_d = Deflection amplification factor in Table 12.2-1 of ASCE 7.~~
~~δ_{max} = Maximum displacement defined in Section 12.8.4.3 of ASCE 7.~~
~~I = Importance factor in accordance with Section 11.5.1 of ASCE 7~~

~~Adjacent buildings on the same property shall be separated by a distance not less than δ_{MT} , determined by Equation 16-45.~~

$$\delta_{MT} = \sqrt{(\delta_{M1})^2 + (\delta_{M2})^2}$$ ~~**(Equation 16-45)**~~

~~where~~

~~δ_{M1} , δ_{M2} = The maximum inelastic response displacements of the adjacent buildings in accordance with Equation 16-44.~~

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

~~**Exceptions:**~~

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

Reason: This provision has been considered and approved by the Seismic Subcommittee of ASCE 7 for inclusion into the 2010 edition of ASCE 7 hence with the adoption of ASCE 7-10 by reference this provision becomes duplicative. As of the submission date of this code change, the ASCE 7 Standards Committee is completing the committee balloting portion of the 2010 edition of ASCE/SEI 7. The document is designated ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* and it is expected that it will be completed and available for purchase prior to the ICC Final Action Hearings in May of 2010 . Any person interested in obtaining a public comment copy of ASCE/SEI 7-10 may do so by contacting the proponent at jrossberg@asce.org.

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

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

ICCFILENAME: ROSSBERG-S5-1613.6.7

S106–09/10
1613.6.8

Proponent: Jim Rossberg, SEI of ASCE, representing self

Delete without substitution:

~~**1613.6.8 HVAC Ductwork with I_p = 1.5.** Seismic supports are not required for HVAC ductwork with I_p = 1.5 if either of the following conditions is met for the full length of each duct run:~~

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

Reason: This provision has been considered and approved by the Seismic Subcommittee of ASCE 7 for inclusion into the 2010 edition of ASCE 7 hence with the adoption of ASCE 7-10 by reference this provision becomes duplicative. As of the submission date of this code change, the ASCE 7 Standards Committee is completing the committee balloting portion of the 2010 edition of ASCE/SEI 7. The document is designated ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* and it is expected that it will be completed and available for purchase prior to the ICC Final Action Hearings in May of 2010 . Any person interested in obtaining a public comment copy of ASCE/SEI 7-10 may do so by contacting the proponent at jrossberg@asce.org.

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

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

ICCFILENAME: ROSSBERG-S4-1613.6.8

S107–09/10
1613.6.9 (New), CHAPTER 35

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

1. Add new text as follows:

<u>**1613.6.9 Seismic design parameters for cold-formed steel special bolted moment frames.** Add a new line, #C12 in ASCE 7, Table 12.2-1 for “Cold-formed Steel – Special Bolted Moment Frame” as follows:</u>

Seismic Force Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R	System Overstrength Factor,	Deflection Amplification Factor, C_d	Structural System Limitations and Building Height (ft) Limit				
					Seismic Design Category				
					B	C	D	E	F
<u>C12. Cold-Formed Steel – Special Bolted Moment Frame[b]</u>	<u>14.1</u>	<u>3.5</u>	<u>3.0[a]</u>	<u>3.5</u>	<u>35</u>	<u>35</u>	<u>35</u>	<u>35</u>	<u>35</u>

<u>a. Alternatively, the seismic load effect with overstrength, E_{mh}, can be based on the expected strength determined in accordance with AISI S110.</u>
<u>b. Cold-formed steel – special bolted moment frames shall be limited to one-story in height in accordance with AISI S110.</u>

2. Add standard to Chapter 35 as follows:

AISI
<u>S110-07</u> <u>Standard for Seismic Design Of Cold-Formed Steel Structural Systems – Special Bolted Moment Frames.</u>

Reason: This proposal introduces the seismic design parameters for a new system, "Cold-formed Steel – Special Bolted Moment Frame" or CFS-SBMFs. 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). Processing for ASCE 7-10 is not yet complete and, because of its shorten cycle, the ASCE 7 Main Committee may not be able to complete its action on the proposal. Consequently, the necessary modifications to ASCE 7 are being introduced here. If action is taken on Proposal TC-6-CH12-102-R3 in time for inclusion in the 2010 edition of ASCE 7, then this proposal will be withdrawn.

Please note, this proposal serves as a companion proposal to an IBC Chapter 22 modification which introduces a reference to the first edition of AISI S110, *Standard For Seismic Design Of Cold- Formed Steel Structural Systems – Special Bolted Moment Frames*. This document is based upon research conducted by Drs. Uang and Sato at UCSD (2007). Specifically, the CFS-SBMF 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 capacity design principles are provided in AISI S110 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.

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, , 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.

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.

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

ICCFILENAME: Manley-S1-1613.6.9

S108–09/10

1613.8 (New), Appendix L (New)

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 m^2) 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.

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

ICCFILENAME: Bachman-S1-1613.8-App L

S109–09/10
1602.1, 1605.2.2, 1605.3.1.2, 1614 (New)

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

1. Add new definition as follows:

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

ICE-SENSITIVE STRUCTURE: A structure for which the effect of an atmospheric ice load governs the design of a structure or portion thereof. This includes, but is not limited to, lattice structures, guyed masts, overhead lines, light suspension and cable-stayed bridges, aerial cable systems (e.g., for ski lifts or logging operations), amusement rides, open catwalks and platforms, flagpoles and signs.

2. Revise as follows:

NOTATIONS.

D = Dead load.
D_i = Weight of ice in accordance with Chapter 10 of ASCE 7.
E = Combined effect of horizontal and vertical earthquake induced forces as defined in Section 12.4.2 of ASCE 7.
F = Load due to fluids with well-defined pressures and maximum heights.
F_a = Flood load in accordance with Chapter 5 of ASCE 7.
H = Load due to lateral earth pressures, ground water pressure or pressure of bulk materials.
L = Live load, except roof live load, including any permitted live load reduction.
L_r = Roof live load including any permitted live load reduction.
R = Rain load.
S = Snow load.
T = Self-straining force arising from contraction or expansion resulting from temperature change, shrinkage, moisture change, creep in component materials, movement due to differential settlement or combinations thereof.
W = Load due to wind pressure.
W_i = Wind-on-ice in accordance with Chapter 10 of ASCE 7.

1605.2.2 ~~Flood~~ Other loads. Where flood loads, F_a, are to be considered in the design, the load combinations of Section 2.3.3 of ASCE 7 shall be used. Where an ice-sensitive structure is subjected to loads due to atmospheric icing, the load combinations of Section 2.3.4 of ASCE 7 shall be considered.

1605.3.1.2 ~~Flood~~ Other loads. Where flood loads, F_a, are to be considered in design, the load combinations of Section 2.4.2 of ASCE 7 shall be used. Where an ice-sensitive structure is subjected to loads due to atmospheric icing, the load combinations of Section 2.4.3 of ASCE 7 shall be considered.

3. Add new text as follows:

SECTION 1614
ATMOSPHERIC ICE LOADS

1614.1 General. Ice-sensitive structures shall be designed for atmospheric ice loads in accordance with Chapter 10 of ASCE 7.

(Renumber remaining sections)

Reason: The purpose for the proposal is to provide charging text in the IBC in conjunction with technical provisions of ASCE 7 for atmospheric ice loads. It is not expected that the ongoing balloting for the 2010 edition of ASCE 7 will affect this proposal.

Note that the statements being added to Sections 1605.2.2 and 1605.3.1.2 for ice-sensitive structures do not constitute charging text. They are cross references to Section 2.3.5 of ASCE 7-10 for Strength/LRFD load combinations and Section 2.4.4 of ASCE 7 for ASD load combinations, which only serve to modify the load combinations for strength design in Section 2.3.2 of ASCE 7-10 and the load combinations for allowable stress design in Section 2.4.4 of ASCE 7.

The statements in Sections 1605.2.2 and 1605.3.1.2 on flood loads, F_a, also do not constitute charging text but Section 1612.1 serves as the charging text for flood loads in the IBC in that it requires buildings, structures and portions thereof to be designed and constructed to resist the effects of flood loads in flood hazard areas. In that regard, Section 1614.1 is added to provide charging text for ice-sensitive structures in the same manner as is provided for flood load.

Section 10.1 of ASCE 7-10 requires atmospheric ice loads to be considered in the design of ice-sensitive structures. An “ice-sensitive structure” is defined in Section 10.2 of ASCE 7-10 and this definition is being added to the IBC. Having a definition of “ice-sensitive structure” in the IBC will provide a technical basis for determining which structures are ice-sensitive structures and, thus, are required to meet the technical provisions of ASCE 7-10 for them. All structures other than ice-sensitive structures are not required to meet these technical provisions.
Without the definition, the IBC will be without effective charging text. IBC Section 1614.1 relies on the determination of which structures are ice-sensitive structures in order to determine the need to comply with the applicable provisions of ASCE 7. That determination is not possible unless a definition of “ice-sensitive structure” is included in the IBC.

Note that this proposal does not contain similar changes to the alternative basic load combinations in Section 1605.3.2. The references to the load combinations in Sections 2.3.5 and 2.4.4 of ASCE 7 are an essential part of this proposal but these sections are not compatible with the alternative basic load combinations of the IBC. Revisions to the alternative basic load combinations for compatibility with the requirements for ice-sensitive structures in ASCE 7-10 should be pursued by others. The deletion of the alternative basic load combinations is the subject of a separate proposal.

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

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

ICCFILENAME: Brazil-S48-1605.2.2

S110–09/10
1614.3.1, 1614.4.1

Proponent: Matthew Senecal, PE, representing American Concrete Institute

Revise as follows:

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

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

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

Reason: ACI is in the process of completely reorganizing ACI 318, *Building Code Requirements for Structural Concrete*. This process was formally initiated in the spring of 2008 and was scheduled to be completed by January 1, 2014 so that it would be available for reference by the 2015 IBC. This 2015 IBC schedule was assumed to be the same as traditional ICC meeting schedules.

The reorganized ACI 318 will be significantly different in structure than 318-08. The code will include several new chapters that are based on member design. The 318-08 chapters will be significantly reworked to support the member chapters. Some provisions have been divided sometimes into several chapters and at other provisions combined.

The new schedules released in February 2009 by the ICC require that reorganized ACI 318 be completed by January 3, 2012 for changes to the IBC (Group A) that require textual changes to the body of the IBC. ACI committee 318 will likely not be done with the revisions by that time and thus not have a revised standard to submit for the 2015 IBC if it remains in Group A. ICC's CP 28-05 states that, if no textual changes are required and that the change to the code is in reference only, the revision may be considered by the administrative code committee prior to the Final Action Hearing in Group B.

In this code change proposal, ACI proposes to remove all references to specific sections in ACI 318 from the IBC, but not change the technical intent of any provision. This removal would allow ACI to submit an administrative change in Group B and give ACI Committee 318 one more year to complete their task of reorganization.

In some cases, the referenced section numbers that are removed are replaced with language from ACI 318 (usually the section heading) that will allow the user to easily locate the appropriate section. In some cases, the referenced information in ACI 318 is transcribed into the IBC. The use of words in place of section numbers may require some editorial revisions in the 2012/2013 ICC cycle, if the headings are changed through the reorganization.

The following explanations are given to for the revised Sections stated above:

1614.3.1 and 1614.4.1	The list of sections is the majority of the structural integrity requirements but not all that are related to these topics. A singular reference to ACI 318 is a more accurate statement. In addition, the provisions in ACI 318 apply to all concrete buildings; whereas, the scope of Section 1614 is limited to a few buildings. Therefore, some may interpret from Sections 1614.3.1 and 1614.4.1 as presently worded that the sections of ACI 318 that are cited only apply to the buildings within the scope of Section 1614. Deletion of the references to these specific sections will eliminate this potential incorrect interpretation.

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

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

ICCFILENAME: SENECAL-S1-1614.3.1

S111–09/10
1702.1

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 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 inspection 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.

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

ICCFILENAME: Brazil-S41-1702.1

S112–09/10
1702

Proponent: John England, MCO, England Enterprises Inc. representing self

Add new text as follows:

<u>**STATEMENT OF SPECIAL INSPECTIONS.** A separate document presented to the building official at time of plan submittal, detailing the special inspections required on the project, with details on how and when the inspections are to be carried out and to what extent. The Building Official has the authority to approve, reject or ask for more information on any part of the *statement of special inspections*.</u>

Reason: The code relates to the *statement of special inspections* but never defines it. A separate document from the specifications on large projects would make it easier for an inspection in the field to figure out who is doing what.

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

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

ICCFILENAME: England-S1-1702

S113–09/10
1702.1

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.

<u>**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.</u>

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.

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

ICCFILENAME: Harman-S1-1702.1

S114-09/10
1702.1

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

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 work <u>items that require</u> ~~requiring~~ special inspection<u> during their construction or installation process</u> by an approved special inspector<u>, in accordance with the statement of special inspections.</u> ~~who is present in the area where the work is being performed.~~

SPECIAL INSPECTION, PERIODIC. The ~~part-time or~~ intermittent observation of work items that can be observed during or after installation that require ~~requiring~~ *special inspection* by an *approved* special inspector, in accordance with the statement of special inspections. ~~who is present in the area where the work has been or is being performed and at the completion of the work.~~

Reason: The current definitions are unclear and do not convey the intent of the difference between continuous and periodic special inspection. This proposal is intended to align the definitions with the requirements of the statement of special inspections. Continuous inspections are required for only those construction items that must be observed during the process of installation, such as multiple pass welds, or concrete placement. Periodic inspections are appropriate for most items that may or must be observed after they are in place, such as reinforcing placement or beam locations.

Also, it does not seem appropriate that the code should require the special inspector to just be "present in the area where the work is being performed". The revisions rely on the definition of *special inspection* and the requirements of Chapter 17 to define the activities of the s*pecial inspector*.

The word "items" is added ("work items") because the word work alone is too broad and there are many individual items that require different levels of inspection in what may be construed as the "work".

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

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

FILENAME: Harman-S5-1702.1-2

S115–09/10

202, 1702, 1703, 1704.17 (New), 1708.1, 1708.4, 1708.4.1 (New), 1708.6 (New), Chapter 35

Proponent: James A Carlson, Seismic Source Company

1. Revise as follows:

SECTION 202
DEFINITIONS

LABEL. An identification applied on a product by the manufacturer or certificate of compliance that contains the name of the manufacturer, the function and performance characteristics of the product or material, and the name and identification of an accredited ~~approved~~ agency ~~and~~ that indicates that the representative sample of the product or material has been ~~tested~~ qualified by test or analysis and evaluated to industry standards and the manufacturers quality program is accepted by an accredited ~~approved~~ agency ~~(see Section 1703.5 and "Inspection certificate," "Manufacturer's designation" and "Mark")~~(A logo is used as a label on documents and equipment information name plate tags).

2. Delete without substitution:

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

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

3. Add new definitions as follows:

SECTION 1702
DEFINITIONS

ACCREDITED AGENCY. Accreditation refers to the formal recognition by a specialized body – an accreditation body.

ACCREDITED SPECIAL INSPECTIONS AGENCY. Agencies that perform field inspections of a building under construction.

ACCREDITED INSPECTION BODIES. Agencies that provide review of the manufacturers' quality process which entails a review of the quality program and onsite verification that records confirm the quality program is being implemented.

4. Delete without substitution:

~~**APPROVED AGENCY.** An established and recognized agency regularly engaged in conducting tests or furnishing inspection services, when such agency has been approved.~~

~~**APPROVED FABRICATOR.** An established and qualified person, firm or corporation approved by the building official pursuant to Chapter 17 of this code.~~

~~**INSPECTION CERTIFICATE.** An identification applied on a product by an approved agency containing the name of the manufacturer, the function and performance characteristics, and the name and identification of an approved agency that indicates that the product or material has been inspected and evaluated by an approved agency (see Section 1703.5 and "Label," "Manufacturer's designation" and "Mark").~~

5. Delete and substitute as follows:

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

<u>**1703.1 Building official approval.** Material, products, and reports are approved by the building official.</u>

6. Delete without substitution:

~~**1703.1.1 Independent.** An approved agency shall be objective and competent. The agency shall also disclose possible conflicts of interest so that objectivity can be confirmed.~~

~~**1703.1.2 Equipment.** An approved agency shall have adequate equipment to perform required tests. The equipment shall be periodically calibrated.~~

~~**1703.1.3 Personnel.** An approved agency shall employ experienced personnel educated in conducting, supervising and evaluating tests and/or inspections.~~

7. Revise as follows:

1703.2 Written approval. Any material, appliance, equipment, system or method of construction meeting the requirements of this code shall be <u>submitted for written approval by the building official during the plan check process</u> ~~approved in writing after satisfactory completion of the required tests and submission of required test reports~~.

1703.4 Performance. Specific information ~~consisting of test reports conducted by an approved testing agency in accordance with standards referenced in Chapter 35, or other such information as necessary,~~ shall be provided for the building official to determine that the material meets the applicable code requirements <u>and for approval</u>.

8. Delete without substitution:

~~**1703.4.1 Research and investigation.** Sufficient technical data shall be submitted to the building official to substantiate the proposed use of any material or assembly. If it is determined that the evidence submitted is satisfactory proof of performance for the use intended, the building official shall approve the use of the material or assembly subject to the requirements of this code. The costs, reports and investigations required under these provisions shall be paid by the permit applicant.~~

~~**1703.4.2 Research reports.** Supporting data, where necessary to assist in the approval of materials or assemblies not specifically provided for in this code, shall consist of valid research reports from approved sources.~~

9. Revise as follows:

1703.5 Labeling. Where materials or assemblies are required by this code to be labeled, such materials and assemblies shall be labeled by an <u>accredited</u> ~~approved~~ agency ~~in accordance with Section 1703~~. Products and materials required to be labeled shall be labeled in accordance with the procedures set forth in Sections 1703.5.1 through 1703.5.3.

1703.5.1 Testing. ~~An approved agency~~ <u>A test lab</u> shall ~~test~~ <u>verify</u> a representative sample of the product or material being labeled to the relevant standard or standards. ~~The approved agency shall maintain a record of the tests performed.~~ The record shall provide sufficient detail to verify compliance with the test standard.

1703.5.2 Inspection and identification. The <u>accredited</u> ~~approved~~ agency shall periodically perform an inspection, which shall be in-plant if necessary, of the product or material that is to be labeled. The inspection shall verify that the labeled product or material is representative of the product or material tested.

1703.5.3 Label information. The label shall contain the manufacturer's or distributor's identification, model number, serial number or definitive information describing the product or material's performance characteristics and <u>accredited</u> ~~approved~~ agency's identification.

10. Add new text as follows:

<u>**1704.17 Special Inspection for force-resisting-systems and designated seismic systems.** Special inspections shall be performed for force-resisting systems as identified in Sections 1705.3 and 1707.8 required by the building official. Special inspections shall be performed for designated seismic system as identified in Section 1707.9 as required by the building official. Inspection frequency shall be in accordance with Table 1704.17.</u>

<u>**1704.17.1 Qualifications.** Special inspection agencies shall have expertise in seismic restraint design as required by Chapter 13 of ASCE 7, knowledge of shake table testing qualification of equipment, and obtain special inspection accreditation.</u>

<u>**TABLE 1704.17**</u>
<u>**REQUIRED VERIFICATION AND INSPECTION OF NONSTRUCTURAL COMPONENTS**</u>

<u>**VERIFICATION AND INSPECTION**</u>	<u>**CONTINUOUS**</u>	<u>**PERIODIC**</u>	<u>**REFERENCE STANDARDS**</u>	<u>**IBC REFERENCE**</u>
1. <u>Verification of seismic supports</u>	<u>-</u>	<u>X</u>	<u>-</u>	<u>1707.7</u>
2. <u>Verification of equipment anchorage and certification of compliance for designated seismic systems</u>	<u>-</u>	<u>X</u>	<u>-</u>	<u>1707.8</u>
3. <u>Verification of isolation systems</u>	<u>-</u>	<u>X</u>	<u>-</u>	<u>1707.9</u>

11. Revise as follows:

1708.1 Testing and qualification for seismic resistance. The testing and qualification specified in Sections 1708.2 through 1708.5, unless exempted from *special inspections* by the exceptions of Section 1704.1, 1705.3 or 1705.3.1 are required as follows:

1. The seismic-force-resisting systems in structures assigned to *Seismic Design Category* C, D, E or F, as determined in Section 1613 shall meet the requirements of Sections 1708.2 and 1708.3, as applicable. <u>Testing of systems or system components shall be performed in accordance with ASHRAE Standard 171 or standards approved by the building official.</u>
2. Designated seismic systems in structures assigned to *Seismic Design Category* C, D, E or F subject to the special certification requirements of ASCE 7 Section 13.2.2 are required to be tested in accordance with Section 1708.4.
3. Architectural, mechanical and electrical components in structures assigned to *Seismic Design Category* C, D, E or F with an *Ip* = 1.0 are required to be tested in accordance with Section 1708.4 where the general design requirements of ASCE 7 Section 13.2.1, Item 2 for manufacturer's certification are satisfied by testing.
4. The seismic isolation system in seismically isolated structures shall meet the testing requirements of Section 1708.5

1708.4 Seismic certification of nonstructural components. The *registered design professional* shall state the applicable seismic certification requirements for nonstructural components and designated seismic systems on the *construction documents*.

1. The manufacturer of each designated seismic system component subject to the provisions of ASCE 7 Section 13.2.2 shall test or analyze the component and its mounting system or anchorage and submit a *certificate of compliance* for review and acceptance by the *registered design professional* responsible for the design of the designated seismic system and for approval by the *building official*. Certification shall be based on an actual

test on a shake table, by three-dimensional shock tests, by an analytical method using dynamic characteristics and forces, by the use of experience data (i.e., historical data demonstrating acceptable seismic performance) or by more rigorous analysis providing for equivalent safety. Use of analysis for active or energized components is not permitted unless a comparison is made to components that have been otherwise deemed as rugged. For complex components, testing or experience may be the only practical way to ensure that the equipment will be operable following a design earthquake. Active simple products such as dampers of fans shall be permitted to be certified by analysis.

2. Manufacturer's certification of compliance for the general design requirements of ASCE 7 Section 13.2.1 shall be based on analysis, testing or experience data.

12. Add new text as follows:

1708.4.1 Hazardous applications. Components with hazardous contents shall be certified by the manufacturer as maintaining containment following the design earthquake by (1) analysis, (2) approved shake table testing, or (3) experience data in accordance. Evidence demonstrating compliance of this requirement shall be submitted to the building official after review and acceptance by the registered design professional.

1708.6 Anchors. For required anchorage testing, see ASCE 7.

13. Add reference standard to Chapter 35 as follows:

ASHRAE
Standard 171-08 Method of Testing Seismic Restraint Devices for HVAC&R Equipment

Reason: (***1702 Definitions)*** Definitions need to be changed to correlate with the current philosophy associated with the implementation of code requirements. In the late 90's, shop drawings used to be stamped with approval by the registered design professional. Now the stamp says reviewed by the design professional and the shop drawings are then approved by the building official. The term "approved agency" has been replaced with "accredited agency" as defined in the ISO standards. The building officials have approved the design and equipment to be installed in buildings, but the building official does not approve design agencies. Even the ES criteria have been updated to delete any reference to "approved agency".
I have performed significant research on this subject. I have yet to find an "approved agency". I have a temporary accreditation as an "accredited" Inspection Body by IAS awaiting final payment. IAS does not accredit fabrication agencies. I am pursuing the idea of becoming an accredited IBC special inspector to fulfill inspection requirements by New York City. This will require significant insurance implications. I am however, providing a listing service for seismic qualified equipment supported by the accredited inspection body services that review the QA program of the manufacturer and is accepted by OSHPD.

Based on the above discussion the following additional changes to the definitions are required.

Delete inspection certificate. The only references are from other definitions. Any certificate reference in the body of text in chapter 17 refers to a certificate of compliance. Special inspections are required and the end product of special inspection is a report not a certificate. Inspection certificate has no use.

Label has been modified to clearly discuss the current process for a certificate of compliance and approval by the building official. Testing of the product is not required by AC 156 to be an accredited test lab. There is an option to use an unaccredited test lab if the test is witnessed by an accredited inspection body. Also, analysis may be an acceptable method to qualify the product. So my listing agency does list Certified Seismic Qualification Agencies that perform analysis or arrange for testing by a subcontractor. My IB services verify that their quality system is acceptable to ISO 9001 standards to control the qualification of equipment. I certify the qualification agencies management system as defined by ISO, see appendix A.

Manufacturers' designation and mark definitions are no longer valid.

(1703 Approvals) All of 1703.1 deals with approved agency. From the discussion above, approved agency has been removed from chapter 17. Since this section is approvals, that 1703.1 was replaced with building official approval. This is a simple definition and can be expanded in later revisions of the IBC.
1703.2 and 1703.4 have been edited to reflect the current plan check process and approved by the building official. 1703.4 shall be combined into one paragraph. 1703.4.2 is no longer required. Research report requirements are provided in industry standards (AC 156, ASHRAE 171, and AHRI Seismic Qualification Standard - draft. I have a contract with AHRI to prepare the standard. A 90% draft is in review and will be completed by October 2009. These new standards reflect the state of art and replace the need for the code to provide criteria.

1703.5 has been updated to reflect the new term accredited agency.

1704 did not contain any requirements for non structural components. I added Section 1704.15 to address this issue and refer to 1707 where requirements are defined. Actually the requirements for special inspections of non structural components are included in 1705, 1707 and 1708. These requirements are duplicate in nature and need thorough review and update. But at a minimum, the addition of 1704.15 is the first step in the revision process. I would be happy to further evaluate this issue in detail and provide further proposals.

1708.1 and 1708.4 have been updated to use the language developed for ASCE 7. I am a member of the Seismic Subcommittee of ASCE 7 and specifically TS-8. I have successfully provided many changes to ASCE 7 that make the requirements clear and concise to resolve industry issues. One change represented changes to the requirements for equipment qualification. The changes to 1708 reflect the current accepted language changes to ASCE 7.

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

Analysis: New wording in Section 1708.4 "For complex components, testing or experience may be the only practical way to ensure that the equipment will be operable following a design earthquake" should be rewritten as a mandatory requirement. A review of the standard(s) proposed for inclusion in the code, ASHRAE 171-08, 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.

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

ICCFILENAME: CARLSON-S1-1702

S116-09/10
1704, 1705, 1706, 1707, 1708, 1709, 1710

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

Revise as follows:

1703.6.1 Follow-up inspection. The applicant shall provide for *special inspections* of fabricated items in accordance with Section ~~1704.2~~ 1704.2.3.

SECTION 1704
SPECIAL INSPECTIONS, CONTRACTOR RESPONSIBILITY AND STRUCTURAL OBSERVATIONS

1704.1 General. This section provides minimum requirements for special inspections, contractor responsibility and structural observations.

~~**1704.1 General.**~~ **1704.2 Special inspections.** 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~~ 1705. 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.

1704.2.1 Special inspector qualifications. 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.

~~**1704.1.1**~~ **1704.2.2 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~~ 1704.3.

Exceptions:

1. A statement of *special inspections* is not required for structures designed and constructed in accordance with the conventional 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*.

~~1704.1.2~~ 1704.2.3 Report requirement. Special inspectors shall keep records of inspections. The special inspector shall furnish inspection reports to the *building official*, and to the *registered design professional in responsible charge*. Reports shall indicate that work inspected was or was not completed in conformance to *approved construction documents*. Discrepancies shall be brought to the immediate attention of the contractor for correction. If they are not corrected, the discrepancies shall be brought to the attention of the *building official* and to the *registered design professional in responsible charge* prior to the completion of that phase of the work. A final report documenting required *special inspections* and correction of any discrepancies noted in the inspections shall be submitted at a point in time agreed upon prior to the start of work by the applicant and the *building official*.

~~1704.2~~ 1704.2.4 Inspection of fabricators. Where fabrication of structural load-bearing members and assemblies is being performed on the premises of a fabricator's shop, *special inspection* of the fabricated items shall be required by this section and as required elsewhere in this code.

~~1704.2.1~~ 1704.2.4.1 Fabrication and implementation procedures. The special inspector shall verify that the fabricator maintains detailed fabrication and quality control procedures that provide a basis for inspection control of the workmanship and the fabricator's ability to conform to *approved construction documents* and referenced standards. The special inspector shall review the procedures for completeness and adequacy relative to the code requirements for the fabricator's scope of work.

Exception: *Special inspections* as required by Section ~~1704.2~~ 1704.2.4 shall not be required where the fabricator is *approved* in accordance with Section ~~1704.2.2~~ 1704.2.4.2.

~~1704.2.2~~ 1704.2.4.2 Fabricator approval. *Special inspections* required by Section ~~1704~~ 1705 are not required where the work is done on the premises of a fabricator registered and *approved* to perform such work without *special inspection*. Approval shall be based upon review of the fabricator's written procedural and quality control manuals and periodic auditing of fabrication practices by an *approved special inspection* agency. At completion of fabrication, the *approved* fabricator shall submit a *certificate of compliance* to the *building official* stating that the work was performed in accordance with the *approved construction documents*.

~~SECTION 1705~~
~~STATEMENT OF SPECIAL INSPECTIONS~~

~~1705.1 General~~. 1704.3 Statement of Special Inspections. Where *special inspection* or testing is required by Section 1704, ~~1707~~ 1705.11 or ~~1708~~ 1705.12, the *registered design professional in responsible charge* shall prepare a statement of special inspections in accordance with Section ~~1705~~ 1704.3 for submittal by the applicant (see Section ~~1704.1.1~~ 1704.2.1).

~~1705.2~~ 1704.3.1Content of statement of special inspections. The statement of special inspections shall identify the following:

1. The materials, systems, components and work required to have *special inspection* or testing by the *building official* or by the *registered design professional* responsible for each portion of the work.
2. The type and extent of each *special inspection*.
3. The type and extent of each test.   .
4. Additional requirements for *special inspection* or testing for seismic or wind resistance as specified in Section ~~1705.3~~ 1704.3.2, ~~1705.4~~ 1704.3.3, ~~1707~~ 1705.11or ~~1708~~1705.12.4.
5. For each type of *special inspection*, identification as to whether it will be continuous *special inspection* or periodic *special inspection*.

~~1705.3~~ 1704.3.2 Seismic resistance. The statement of special inspections shall include seismic requirements for cases covered in Sections ~~1705.3.1~~ 1704.3.2.1 through ~~1705.3.5~~ 1704.3.2.5.

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

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

~~1705.3.1~~ 1704.3.2.1 Seismic-force-resisting systems. The seismic force- resisting systems in structures assigned to *Seismic Design Category* C, D, E or F, in accordance with Section 1613.

Exception: Requirements for the seismic-force-resisting system are permitted to be excluded from the statement of special inspections for steel systems in structures assigned to *Seismic Design Category* C that are not specifically detailed for seismic resistance, with a response modification coefficient, *R*, of 3 or less, excluding cantilever column systems.

~~1705.3.2~~ 1704.3.2.2 Designated seismic systems. Designated seismic systems in structures assigned to *Seismic Design Category* D, E or F.

~~1705.3.3~~ 1704.3.2.3 Seismic Design Category C. The following additional systems and components in structures assigned to *Seismic Design Category* C:

1. Heating, ventilating and air-conditioning (HVAC) ductwork containing hazardous materials and anchorage of such ductwork.
2. Piping systems and mechanical units containing flammable, combustible or highly *toxic* materials.
3. Anchorage of electrical equipment used for emergency or standby power systems.

~~1705.3.4~~ 1704.3.2.4 Seismic Design Category D. The following additional systems and components in structures assigned to *Seismic Design Category* D:

1. Systems required for *Seismic Design Category* C.
2. Exterior wall panels and their anchorage.
3. Suspended ceiling systems and their anchorage.
4. Access floors and their anchorage.
5. Steel storage racks and their anchorage, where the importance factor is equal to 1.5 in accordance with Section 15.5.3 of ASCE 7.

~~1705.3.5~~ 1704.3.2.5 Seismic Design Category E or F. The following additional systems and components in structures assigned to *Seismic Design Category* E or F:

1. Systems required for *Seismic Design Categories* C and D.
2. Electrical equipment.

~~1705.3.6~~ 1704.3.2.6 Seismic requirements in the statement of special inspections. When Sections ~~1705.3~~ 1704.3.2 through ~~1705.3.5~~ 1704.3.2.5 specify that seismic requirements be included, the statement of special inspections shall identify the following:

1. The designated seismic systems and seismic force- resisting systems that are subject to *special inspections* in accordance with Sections ~~1705.3~~ 1704.3.2 through ~~1705.3.5~~ 1704.3.2.5.
2. The additional *special inspections* and testing to be provided as required by Sections ~~1707~~ 1705.11 and ~~1708~~ 1705.12 and other applicable sections of this code, including the applicable standards referenced by this code.

~~**1705.4**~~ **1704.3.3 Wind resistance.** The statement of special inspections shall include wind requirements for structures constructed in the following areas:

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

~~**1705.4.1**~~ **1704.3.3.1 Wind requirements in the statement of special inspections.** When Section ~~1705.4~~ 1704.3.3 specifies that wind requirements be included, the statement of special inspections shall identify the main wind-force-resisting systems and wind-resisting components subject to *special inspections* as specified in Section ~~1705.4.2~~ 1704.3.3.2.

~~**1705.4.2**~~ **1704.3.3.2 Detailed requirements.** The statement of special inspections shall include at least the following systems and components:

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

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

~~SECTION 1709~~
~~CONTRACTOR RESPONSIBILITY~~

~~**1709.1**~~ **1704.4 Contractor responsibility.** Each contractor responsible for the construction of a main wind- or seismic-force-resisting system, designated seismic system or a wind- or seismic-resisting component listed in the statement of special inspections shall submit a written statement of responsibility to the *building official* and the owner prior to the commencement of work on the system or component. The contractor's statement of responsibility shall contain acknowledgement of awareness of the special requirements contained in the statement of *special inspection*.

~~SECTION 1710~~
~~STRUCTURAL OBSERVATIONS~~

~~**1710.1 General.**~~ **1704.5 Structural observations.** Where required by the provisions of Section ~~1710.2~~ 1704.5.1 or ~~1710.3~~ 1704.5.2, the owner shall employ a *registered design professional* to perform structural observations as defined in Section 1702.

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

~~**1710.2**~~ **1704.5.1 Structural observations for seismic resistance.** Structural observations shall be provided for those structures assigned to *Seismic Design Category* D, E or F, as determined in Section 1613, where one or more of the following conditions exist:

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

~~1710.3~~ 1704.5.2 Structural observations for wind requirements. Structural observations shall be provided for those structures sited where the basic wind speed exceeds 110 mph (49 m/sec) determined from Figure 1609, where one or more of the following conditions exist:

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

SECTION 1705
REQUIRED VERIFICATION AND INSPECTION

1705.1 General. Verification and inspection of elements of buildings and structures shall be as required by this section.

~~1704.15~~ 1705.1.1 Special cases. *Special inspections* shall be required for proposed work that is, in the opinion of the *building official*, unusual in its nature, such as, but not limited to, the following examples:

1. Construction materials and systems that are alternatives to materials and systems prescribed by this code.
2. Unusual design applications of materials described in this code.
3. Materials and systems required to be installed in accordance with additional manufacturer's instructions that prescribe requirements not contained in this code or in standards referenced by this code.

~~1704.3~~ 1705.2 Steel construction. The *special inspections* for steel elements of buildings and structures shall be as required by Section ~~1704.3~~ 1705.2 and Table ~~1704.3~~ 1705.2.

Exceptions:

1. *Special inspection* of the steel fabrication process shall not be required where the fabricator does not perform any welding, thermal cutting or heating operation of any kind as part of the fabrication process. In such cases, the fabricator shall be required to submit a detailed procedure for material control that demonstrates the fabricator's ability to maintain suitable records and procedures such that, at any time during the fabrication process, the material specification, grade and mill test reports for the main stress-carrying elements are capable of being determined.
2. The special inspector need not be continuously present during welding of the following items, provided the materials, welding procedures and qualifications of welders are verified prior to the start of the work; periodic inspections are made of the work in progress and a visual inspection of all welds is made prior to completion or prior to shipment of shop welding.
   - 2.1. Single-pass fillet welds not exceeding 5/16 inch (7.9 mm) in size.
   - 2.2. Floor and roof deck welding.
   - 2.3. Welded studs when used for structural diaphragm.
   - 2.4. Welded sheet steel for cold-formed steel members.
   - 2.5. Welding of stairs and railing systems.

~~1704.3.1~~ 1705.2.1 Welding. Welding inspection and welding inspector qualification shall be in accordance with this section.

~~1704.3.1.1~~ 1705.2.1.1 Structural steel. Welding inspection and welding inspector qualification for structural steel shall be in accordance with AWS D1.1.

~~1704.3.1.2~~ 1705.2.1.2 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~~ 1705.2.1.3 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~~ 1705.2.2 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~~ 1705.2.3 **High-strength bolts.** Installation of high-strength bolts shall be inspected in accordance with AISC 360.

~~1704.3.3.1~~ 1705.2.3.1**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~~ 1705.2.3.2 **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~~ 1705.2.3.3 **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.

~~1704.3.4~~ 1705.2.4 **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~~ 1705.2
REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]	IBC REFERENCE
2. Inspection of high-strength bolting:				
a.Snug-tight joints.	—	X	AISC 360, Section M2.5	~~1704.3.3~~ 1705.2.3
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	—		
5. 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~~ 1705.2.1
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	
b.Reinforcing steel:				
1) Verification of weldability of reinforcing steel other than ASTM A 706.	—	X	AWS D1.4 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 structural walls of concrete and shear reinforcement.	X	—		

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARDa	IBC REFERENCE
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~~ 1705.2.2
b. Member locations.		—	X	
c. Application of joint details at each connection.		—	X	

(Portions of table not shown do not change)

a. Where applicable, see also Section ~~1707.1~~ 1705.11, Special inspection for seismic resistance.

~~1704.4~~ 1705.3 Concrete construction. The *special inspections* and verifications for concrete construction shall be as required by this section and Table ~~1704.4~~ 1705.3.

Exception: *Special inspections* shall not be required for:

1. Isolated spread concrete footings of buildings three stories or less above *grade plane* that are fully supported on earth or rock.
2. Continuous concrete footings supporting walls of buildings three stories or less above *grade plane* that are fully supported on earth or rock where:
   2.1. The footings support walls of light-frame construction;
   2.2. The footings are designed in accordance with Table 1809.7; or
   2.3. The structural design of the footing is based on a specified compressive strength, f'_c, no greater than 2,500 pounds per square inch (psi) (17.2 Mpa), regardless of the compressive strength specified in the *construction documents* or used in the footing construction.
3. Nonstructural concrete slabs supported directly on the ground, including prestressed slabs on grade, where the effective prestress in the concrete is less than 150 psi (1.03 Mpa).
4. Concrete foundation walls constructed in accordance with Table 1807.1.6.2.
5. Concrete patios, driveways and sidewalks, on grade.

~~1704.4.1~~ 1705.3.1 Materials. In the absence of sufficient data or documentation providing evidence of conformance to quality standards for materials in Chapter 3 of ACI 318, the building official shall require testing of materials in accordance with the appropriate standards and criteria for the material in Chapter 3 of ACI 318. Weldability of reinforcement, except that which conforms to ASTM A 706, shall be determined in accordance with the requirements of Section 3.5.2 of ACI 318.

TABLE ~~1704.4~~ 1705.3
REQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION
(No Change to Table)

a. Where applicable, see also Section ~~1707.1~~ 1705.11, Special inspection for seismic resistance.

~~1704.5~~ 1705.4 Masonry construction. Masonry construction shall be inspected and verified in accordance with the requirements of Sections ~~1704.5.1~~ 1705.4.1 through ~~1704.5.2~~ 1705.4.2, depending on the *occupancy category* of the building or structure.

Exception: *Special inspections* shall not be required for:

1. Empirically designed masonry, glass unit masonry or masonry veneer designed by Section 2109, 2110 or Chapter 14, respectively, or by Chapter 5, 7 or 6 of TMS 402/ACI 530/ASCE 5, respectively, when they are part of structures classified as *Occupancy Category* I, II or III in accordance with Section 1604.5.
2. Masonry foundation walls constructed in accordance with Table 1807.1.6.3(1), 1807.1.6.3(2),1807.1.6.3(3) or 1807.1.6.3(4).
3. Masonry fireplaces, masonry heaters or masonry chimneys installed or constructed in accordance with Section 2111, 2112 or 2113, respectively.

~~1704.5.1~~ <u>1705.4.1</u> **Empirically designed masonry, glass unit masonry and masonry veneer in Occupancy Category IV.** The minimum *special inspection* program for empirically designed masonry, glass unit masonry or masonry veneer designed by Section 2109, 2110 or Chapter 14, respectively, or by Chapter 5, 7 or 6 of TMS 402/ACI ASCE 5, respectively, in structures classified as *Occupancy Category* IV, in accordance with Section 1604.5, shall comply with Table ~~1704.5.1~~ <u>1705.4.1</u>.

~~1704.5.2~~ <u>1705.4.2</u> **Engineered masonry in Occupancy Category I, II or III.** The minimum *special inspection* program for masonry designed by Section 2107 or 2108 or by chapters other than Chapter 5, 6 or 7 of TMS402/ACI 530/ASCE 5 in structures classified as *Occupancy Category* I, II or III, in accordance with Section 1604.5, shall comply with Table ~~1704.5.1~~ <u>1705.4.1</u>.

~~1704.5.3~~ <u>1705.4.3</u> **Engineered masonry in Occupancy Category IV.** The minimum *special inspection* program for masonry designed by Section 2107 or 2108 or by chapters other than Chapter 5, 6 or 7 of TMS402/ACI 530/ASCE 5 in structures classified as *Occupancy Category* IV, in accordance with Section 1604.5, shall comply with Table ~~1704.5.3~~ <u>1705.4.3</u>.

~~1704.11~~ <u>1705.4.4</u> **Vertical masonry foundation elements.** *Special inspection* shall be performed in accordance with Section ~~1704.5~~ <u>1705.4</u> for vertical masonry foundation elements.

TABLE ~~1704.5.1~~ <u>1705.4.1</u>
LEVEL 1 REQUIRED VERIFICATION AND INSPECTION OF MASONRY CONSTRUCTION
(No Change to Table)

TABLE ~~1704.5.3~~ <u>1705.4.3</u>
LEVEL 2 REQUIRED VERIFICATION AND INSPECTION OF MASONRY CONSTRUCTION
(No Change to Table)

~~1704.6~~ <u>1705.5</u> **Wood construction.** *Special inspections* of the fabrication process of prefabricated wood structural elements and assemblies shall be in accordance with Section ~~1704.2~~ <u>1704.2.3</u>. *Special inspections* of site-built assemblies shall be in accordance with this section.

~~1704.6.1~~ <u>1705.5.1</u> **High-load diaphragms.** High-load diaphragms designed in accordance with Table 2306.2.1(2) shall be installed with *special inspections* as indicated in Section ~~1704.1~~ <u>1704.2</u>. The special inspector shall inspect the wood structural panel sheathing to ascertain whether it is of the grade and thickness shown on the *approved* building plans. Additionally, the special inspector must verify the nominal size of framing members at adjoining panel edges, the nail or staple diameter and length, the number of fastener lines and that the spacing between fasteners in each line and at edge margins agrees with the *approved* building plans.

~~1704.6.2~~ <u>1705.5.2</u> **Metal-plate-connected wood trusses spanning 60 feet or greater.** Where a 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.

~~1704.7~~ <u>1705.6</u> **Soils.** *Special inspections* for existing site soil conditions, fill placement and load-bearing requirements shall be as required by this section and Table ~~1704.7~~ <u>1705.6</u>. The *approved* geotechnical report, and the *construction documents* prepared by the *registered design professionals* shall be used to determine compliance. During fill placement, the special inspector shall determine that proper materials and procedures are used in accordance with the provisions of the *approved* geotechnical report.

> **Exception:** Where Section 1803 does not require reporting of materials and procedures for fill placement, the special inspector shall verify that the in-place dry density of the compacted fill is not less than 90 percent of the maximum dry density at optimum moisture content determined in accordance with ASTM D 1557.

~~1704.8~~ <u>1705.7</u> **Driven deep foundations.** *Special inspections* shall be performed during installation and testing of driven deep foundation elements as required by Table ~~1704.8~~ <u>1705.7</u>. The *approved* geotechnical report, and the *construction documents* prepared by the *registered design professionals*, shall be used to determine compliance.

~~1704.9~~ <u>1705.8</u> **Cast-in-place deep foundations.** *Special inspections* shall be performed during installation and testing of cast-in-place deep foundation elements as required by Table ~~1704.9~~ <u>1705.8</u>. The *approved* geotechnical report, and the *construction documents* prepared by the *registered design professionals*, shall be used to determine compliance.

~~1704.10~~ __1705.9__ **Helical pile foundations.** *Special inspections* shall be performed continuously during installation of helical pile foundations. The information recorded shall include installation equipment used, pile dimensions, tip elevations, final depth, final installation torque and other pertinent installation data as required by the *registered design professional in responsible charge*. The *approved* geotechnical report and the documents prepared by the *registered design professional* shall be used to determine compliance.

TABLE ~~1704.7~~ __1705.6__
REQUIRED VERIFICATION AND INSPECTION OF SOILS
(No Change to Table)

TABLE ~~1704.8~~ __1705.7__
REQUIRED VERIFICATION AND INSPECTION OF DRIVEN DEEP FOUNDATION ELEMENTS

VERIFICATION AND INSPECTION TASK	CONTINUOUS DURING TASK LISTED	PERIODICALLY DURING TASK LISTED
5. For steel elements, perform additional inspections in accordance with Section ~~1704.3~~ __1705.2__.	—	—
6. For concrete elements and concrete-filled elements, perform additional inspections in accordance with Section ~~1704.4~~ __1705.3__.	—	—

(Portions of table not shown are unchanged)

TABLE ~~1704.9~~ __1705.8__
REQUIRED VERIFICATION AND INSPECTION OF CAST-IN-PLACE DEEP FOUNDATION ELEMENTS

VERIFICATION AND INSPECTION TASK	CONTINUOUS DURING TASK LISTED	PERIODICALLY DURING TASK LISTED
3. For concrete elements, perform additional inspections in accordance with Section ~~1704.4~~ __1705.3__.	—	—

(Portions of table not shown are unchanged)

~~SECTION 1706~~
~~SPECIAL INSPECTIONS FOR WIND REQUIREMENTS~~

~~1706.1~~ __1705.10__ **Special inspections for wind ~~requirements~~ __resistance__.** *Special inspections* itemized in Sections ~~1706.2~~ __1705.10.1__ through ~~1706.4~~ __1705.10.3__, unless exempted by the exceptions to Section ~~1704.1~~ __1704.2__, are required for buildings and structures constructed in the following areas:

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

~~1706.2~~ __1705.10.1__ **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.

~~1706.3~~ __1705.10.2__ **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.).

~~1706.4~~ 1705.10.3 Wind-resisting components. Periodic special inspection is required for the following systems and components:

1. Roof cladding.
2. Wall cladding.

~~SECTION 1707~~
~~SPECIAL INSPECTIONS FOR SEISMIC RESISTANCE~~

~~1707.1~~ 1705.11 Special inspections for seismic resistance. *Special inspections* itemized in Sections ~~1707.2~~ 1705.11.1 through ~~1707.9~~ 1705.11.8, unless exempted by the exceptions of Section ~~1704.1~~ 1704.2, ~~1705.3~~ 1704.3.2, or ~~1705.3.1~~ 1704.3.2.1, are required for the following:

1. The seismic-force-resisting systems in structures assigned to *Seismic Design Category* C, D, E or F, as determined in Section 1613.
2. Designated seismic systems in structures assigned to *Seismic Design Category* D, E or F.
3. Architectural, mechanical and electrical components in structures assigned to *Seismic Design Category* C, D, E or F that are required in Sections ~~1707.6~~ 1705.11.5 and ~~1707.7~~ 1705.11.6.

~~1707.2~~ 1705.11.1 Structural steel. *Special inspection* for structural steel shall be in accordance with the quality assurance plan requirements of AISC 341.

Exceptions:

1. *Special inspections* of structural steel in structures assigned to *Seismic Design Category* C that are not specifically detailed for seismic resistance, with a response modification coefficient, *R*, of 3 or less, excluding cantilever column systems.
2. For ordinary moment frames, ultrasonic and magnetic particle testing of complete joint penetration groove welds are only required for demand critical welds.

~~1707.3~~ 1705.11.2 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.).

~~1707.4~~ 1705.11.3 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.

~~1707.5~~ 1705.11.4 Storage racks and access floors. Periodic *special inspection* is required during the anchorage of access floors and storage racks 8 feet (2438 mm) or greater in height in structures assigned to *Seismic Design Category* D, E or F.

~~1707.6~~ **<u>1705.11.5</u> Architectural components.** Periodic *special inspection* during the erection and fastening of exterior cladding, interior and exterior nonbearing walls and interior and exterior veneer in structures assigned to *Seismic Design Category* D, E or F.

Exceptions:

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

~~1707.7~~ **<u>1705.11.6</u> Mechanical and electrical components.** *Special inspection* for mechanical and electrical equipment shall be as follows:

1. Periodic special inspection is required during the anchorage of electrical equipment for emergency or standby power systems in structures assigned to *Seismic Design Category* C, D, E or F;
2. Periodic special inspection is required during the installation of anchorage of other electrical equipment in structures assigned to *Seismic Design Category* E or F;
3. Periodic special inspection is required during installation of piping systems intended to carry flammable, combustible or *highly toxic* contents and their associated mechanical units in structures assigned to *Seismic Design Category* C, D, E or F;
4. Periodic special inspection is required during the installation of HVAC ductwork that will contain hazardous materials in structures assigned to *Seismic Design Category* C, D, E or F; and
5. Periodic special inspection is required during the installation of vibration isolation systems in structures assigned to *Seismic Design Category* C, D, E or F where the *construction documents* require a nominal clearance of 1/4 inch (6.4 mm) or less between the equipment support frame and restraint.

~~1707.8~~ **<u>1705.11.7</u> Designated seismic system verifications.** The special inspector shall examine designated seismic systems requiring seismic qualification in accordance with Section ~~1708.5~~ <u>1705.12.4</u> and verify that the *label*, anchorage or mounting conforms to the *certificate of compliance*.

~~1707.9~~ **<u>1705.11.8</u> Seismic isolation system.** Periodic special inspection is required during the fabrication and installation of isolator units and energy dissipation devices that are part of the seismic isolation system.

~~SECTION 1708~~
~~STRUCTURAL TESTING FOR SEISMIC RESISTANCE~~

~~1708.1~~ **<u>1705.12</u> Testing and qualification for seismic resistance.** The testing and qualification specified in Sections ~~1708.2~~ <u>1705.12.1</u> through ~~1708.5~~ <u>1705.12.4</u>, unless exempted from *special inspections* by the exceptions of Section ~~1704.1~~ <u>1704.2</u>, ~~1705.3~~ <u>1704.3.2</u>, or ~~1705.3.1~~ <u>1704.3.2.1</u> are required as follows:

1. The seismic-force-resisting systems in structures assigned to *Seismic Design Category* C, D, E or F, as determined in Section 1613 shall meet the requirements of Sections ~~1708.2~~ <u>1705.12.1</u> and ~~1708.~~3 <u>1705.12.2</u>, as applicable.
2. Designated seismic systems in structures assigned to *Seismic Design Category* C, D, E or F subject to the special certification requirements of ASCE 7 Section 13.2.2 are required to be tested in accordance with Section ~~1708.4~~ <u>1705.12.3</u>.
3. Architectural, mechanical and electrical components in structures assigned to *Seismic Design Category* C, D, E or F with an *Ip* = 1.0 are required to be tested in accordance with Section ~~1708.4~~ <u>1705.12.3</u> where the general design requirements of ASCE 7 Section 13.2.1, Item 2 for manufacturer's certification are satisfied by testing.
4. The seismic isolation system in seismically isolated structures shall meet the testing requirements of Section ~~1708.5~~ <u>1705.12.4</u>.

~~1708.2~~ **<u>1705.12.1</u> Concrete reinforcement.** Where reinforcement complying with ASTM A 615 is used to resist earthquake-induced flexural and axial forces in special moment frames, special structural walls and coupling beams connecting special structural walls, in structures assigned to *Seismic Design Category* B, C, D, E or F as determined in Section 1613, the reinforcement shall comply with Section 21.1.5.2 of ACI 318. Certified mill test reports shall be provided for each shipment of such reinforcement. Where reinforcement complying with ASTM A 615 is to be welded, chemical tests shall be performed to determine weldability in accordance with Section 3.5.2 of ACI 318.

~~**1708.3**~~ **<u>1705.12.2</u> Structural steel.** Testing for structural steel shall be in accordance with the quality assurance plan requirements of AISC 341.

Exceptions:

1. Testing for structural steel in structures assigned to *Seismic Design Category* C that are not specifically detailed for seismic resistance, with a response modification coefficient, *R*, of 3 or less, excluding cantilever column systems.
2. For ordinary moment frames, ultrasonic and magnetic particle testing of complete joint penetration groove welds are only required for demand critical welds.

~~**1708.4**~~ **<u>1705.12.3</u> Seismic certification of nonstructural components.** The *registered design professional* shall state the applicable seismic certification requirements for nonstructural components and designated seismic systems on the *construction documents*.

1. The manufacturer of each designated seismic system component subject to the provisions of ASCE 7 Section 13.2.2 shall test or analyze the component and its mounting system or anchorage and submit a *certificate of compliance* for review and acceptance by the *registered design professional* responsible for the design of the designated seismic system and for approval by the *building official*. Certification shall be based on an actual test on a shake table, by three-dimensional shock tests, by an analytical method using dynamic characteristics and forces, by the use of experience data (i.e., historical data demonstrating acceptable seismic performance) or by more rigorous analysis providing for equivalent safety.
2. Manufacturer's certification of compliance for the general design requirements of ASCE 7 Section 13.2.1 shall be based on analysis, testing or experience data.

~~**1708.5**~~ **<u>1705.12.4</u> Seismically isolated structures.** For required system tests, see Section 17.8 of ASCE 7.

~~**1704.12**~~ **<u>1705.13</u> Sprayed fire-resistant materials.** *Special inspections* for sprayed fire-resistant materials applied to floor, roof and wall assemblies and structural members shall be in accordance with Sections ~~1704.12.1~~ <u>1705.13.1</u> through ~~1704.12.6~~ <u>1705.13.6</u>. *Special inspections* shall be based on the fire-resistance design as designated in the *approved construction documents*. The tests set forth in this section shall be based on samplings from specific floor, roof and wall assemblies and structural members. *Special inspections* shall be performed after the rough installation of electrical, automatic sprinkler, mechanical and plumbing systems and suspension systems for ceilings, where applicable.

~~**1704.12.1**~~ **<u>1705.13.1</u> Physical and visual tests.** The *special inspections* shall include the following tests and observations to demonstrate compliance with the listing and the fire-resistance rating:

1. Condition of substrates.
2. Thickness of application.
3. Density in pounds per cubic foot (kg/m3).
4. Bond strength adhesion/cohesion.
5. Condition of finished application.

~~**1704.12.2**~~ **<u>1705.13.2</u> Structural member surface conditions.** The surfaces shall be prepared in accordance with the *approved* fire-resistance design and the written instructions of *approved* manufacturers. The prepared surface of structural members to be sprayed shall be inspected before the application of the sprayed fire-resistant material.

~~**1704.12.3**~~ **<u>1705.13.3</u> Application.** The substrate shall have a minimum ambient temperature before and after application as specified in the written instructions of *approved* manufacturers. The area for application shall be ventilated during and after application as required by the written instructions of *approved* manufacturers.

~~**1704.12.4**~~ **<u>1705.13.4</u> Thickness.** No more than 10 percent of the thickness measurements of the sprayed fire-resistant materials applied to floor, roof and wall assemblies and structural members shall be less than the thickness required by the *approved* fire-resistance design, but in no case less than the minimum allowable thickness required by Section ~~1704.12.4.1~~ <u>1705.13.4.1</u>.

~~**1704.12.4.1**~~ **<u>1705.13.4.1</u> Minimum allowable thickness.** For design thicknesses 1 inch (25 mm) or greater, the minimum allowable individual thickness shall be the design thickness minus 1/4 inch (6.4 mm). For design thicknesses less than 1 inch (25 mm), the minimum allowable individual thickness shall be the design thickness minus 25 percent.

Thickness shall be determined in accordance with ASTM E 605. Samples of the sprayed fire-resistant materials shall be selected in accordance with Sections ~~1704.12.4.2~~ 1705.13.4.2 and ~~1704.12.4.3~~ 1705.13.4.5.

~~704.12.4.2~~ 1705.13.4.2 Floor, roof and wall assemblies. The thickness of the sprayed fire-resistant material applied to floor, roof and wall assemblies shall be determined in accordance with ASTM E 605, making not less than four measurements for each 1,000 square feet (93 m2) of the sprayed area in each *story* or portion thereof.

~~1704.12.4.2.1~~ 1705.13.4.3 Cellular decks. Thickness measurements shall be selected from a square area, 12 inches by 12 inches (305 mm by 305 mm) in size. A minimum of four measurements shall be made, located symmetrically within the square area.

~~1704.12.4.2.2~~ 1705.13.4.4 Fluted decks. Thickness measurements shall be selected from a square area, 12 inches by 12 inches (305 mm by 305 mm) in size. A minimum of four measurements shall be made, located symmetrically within the square area, including one each of the following: valley, crest and sides. The average of the measurements shall be reported.

~~1704.12.4.3~~ 1705.13.4.5 Structural members. The thickness of the sprayed fire-resistant material applied to structural members shall be determined in accordance with ASTM E 605. Thickness testing shall be performed on not less than 25 percent of the structural members on each floor.

~~1704.12.4.3.1~~ 1705.13.4.6 Beams and girders. At beams and girders thickness measurements shall be made at nine locations around the beam or girder at each end of a 12-inch (305 mm) length.

~~1704.12.4.3.2~~ 1705.13.4.7 Joists and trusses. At joists and trusses, thickness measurements shall be made at seven locations around the joist or truss at each end of a 12-inch (305 mm) length.

~~1704.12.4.3.3~~ 1705.13.4.8 Wide-flanged columns. At wide-flanged columns, thickness measurements shall be made at 12 locations around the column at each end of a 12-inch (305 mm) length.

~~1704.12.4.3.4~~ 1705.13.4.9 Hollow structural section and pipe columns. At hollow structural section and pipe columns, thickness measurements shall be made at a minimum of four locations around the column at each end of a 12-inch (305 mm) length.

~~1704.12.5~~ 1705.13.5 Density. The density of the sprayed fire-resistant material shall not be less than the density specified in the *approved* fire-resistance design. Density of the sprayed fire-resistant material shall be determined in accordance with ASTM E 605. The test samples for determining the density of the sprayed fire-resistant materials shall be selected as follows:

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

~~1704.12.6~~ 1705.13.6 Bond strength. The cohesive/adhesive bond strength of the cured sprayed fire-resistant material applied to floor, roof and wall assemblies and structural members shall not be less than 150 pounds per square foot (psf) (7.18 kN/m^2). The cohesive/adhesive bond strength shall be determined in accordance with the field test specified in ASTM E 736 by testing in-place samples of the sprayed fire-resistant material selected in accordance with Sections ~~1704.12.6.1~~ 1705.13.6.1 through ~~1704.12.6.3~~ 1705.13.6.3.

~~1704.12.6.1~~ 1705.13.6.1 Floor, roof and wall assemblies. The test samples for determining the cohesive/adhesive bond strength of the sprayed fire-resistant materials shall be selected from each floor, roof and wall assembly at the rate of not less than one sample for every 2,500 square feet (232 m^2) of the sprayed area in each *story* or portion thereof.

~~1704.12.6.2~~ 1705.13.6.2 Structural members. The test samples for determining the cohesive/adhesive bond strength of the sprayed fire-resistant materials shall be selected from beams, girders, trusses, columns and other structural members at the rate of not less than one sample for each type of structural member for each 2,500 square feet (232 m^2) of floor area or portion thereof in each *story*.

~~1704.12.6.3~~ 1705.13.6.3 Primer, paint and encapsulant bond tests. Bond tests to qualify a primer, paint or encapsulant shall be conducted when the sprayed fire-resistant material is applied to a primed, painted or encapsulated surface for which acceptable bond-strength performance between these coatings and the fire-resistant material has not been determined. A bonding agent *approved* by the SFRM manufacturer shall be applied to a primed, painted or encapsulated surface where the bond strengths are found to be less than required values.

~~1704.13~~ 1705.14 Mastic and intumescent fire-resistant coatings. *Special inspections* for mastic and intumescent fire-resistant coatings applied to structural elements and decks shall be in accordance with AWCI 12-B. *Special inspections* shall be based on the fire-resistance design as designated in the *approved construction documents*.

~~1704.14~~ 1705.15 Exterior insulation and finish systems (EIFS). *Special inspections* shall be required for all EIFS applications.

Exceptions:

1. *Special inspections* shall not be required for EIFS applications installed over a *water-resistive barrier* with a means of draining moisture to the exterior.
2. *Special inspections* shall not be required for EIFS applications installed over masonry or concrete walls.

~~1704.14.1~~ 1705.15.1 Water-resistive barrier coating. A *water-resistive barrier* coating complying with ASTM E 2570 requires *special inspection* of the *water-resistive barrier* coating when installed over a sheathing substrate.

[F] ~~1704.16~~ 1705.16 Special inspection for smoke control. Smoke control systems shall be tested by a special inspector.

[F] ~~1704.16.1~~ 1705.16.1 Testing scope. The test scope shall be as follows:

1. During erection of ductwork and prior to concealment for the purposes of leakage testing and recording of device location.
2. Prior to occupancy and after sufficient completion for the purposes of pressure difference testing, flow measurements and detection and control verification.

[F] ~~1704.16.2~~ 1705.16.2 Qualifications. *Special inspection* agencies for smoke control shall have expertise in fire protection engineering, mechanical engineering and certification as air balancers.

SECTION ~~1711~~ 1706
DESIGN STRENGTHS OF MATERIALS

~~1711.1~~ 1706.1 Conformance to standards. The design strengths and permissible stresses of any structural material that are identified by a manufacturer's designation as to manufacture and grade by mill tests, or the strength and stress grade is otherwise confirmed to the satisfaction of the *building official*, shall conform to the specifications and methods of design of accepted engineering practice or the *approved* rules in the absence of applicable standards.

~~1711.2~~ 1706.2 New materials. For materials that are not specifically provided for in this code, the design strengths and permissible stresses shall be established by tests as provided for in Section ~~1712~~ 1707.

SECTION ~~1712~~ 1707
ALTERNATIVE TEST PROCEDURE

~~1712.1~~ 1707.1 General. In the absence of *approved* rules or other *approved* standards, the *building official* shall make, or cause to be made, the necessary tests and investigations; or the *building official* shall accept duly authenticated reports from *approved agencies* in respect to the quality and manner of use of new materials or assemblies as provided for in Section 104.11. The cost of all tests and other investigations required under the provisions of this code shall be borne by the applicant.

SECTION ~~1713~~ 1708
TEST SAFE LOAD

~~1713.1~~ 1708.1 Where required. Where proposed construction is not capable of being designed by *approved* engineering analysis, or where proposed construction design method does not comply with the applicable material design standard, the system of construction or the structural unit and the connections shall be subjected to the tests prescribed in Section ~~1715~~ 1710. The *building official* shall accept certified reports of such tests conducted by an *approved* testing agency, provided that such tests meet the requirements of this code and *approved* procedures.

SECTION ~~1714~~ 1709
IN-SITU LOAD TESTS

~~**1714.1**~~ **1709.1 General.** Whenever there is a reasonable doubt as to the stability or load-bearing capacity of a completed building, structure or portion thereof for the expected loads, an engineering assessment shall be required. The engineering assessment shall involve either a structural analysis or an in-situ load test, or both. The structural analysis shall be based on actual material properties and other as-built conditions that affect stability or load-bearing capacity, and shall be conducted in accordance with the applicable design standard. If the structural assessment determines that the load-bearing capacity is less than that required by the code, load tests shall be conducted in accordance with Section ~~1714.2~~ 1709.2. If the building, structure or portion thereof is found to have inadequate stability or load-bearing capacity for the expected loads, modifications to ensure structural adequacy or the removal of the inadequate construction shall be required.

~~**1714.2**~~ **1709.2 Test standards.** Structural components and assemblies shall be tested in accordance with the appropriate material standards listed in Chapter 35. In the absence of a standard that contains an applicable load test procedure, the test procedure shall be developed by a *registered design professional* and *approved*. The test procedure shall simulate loads and conditions of application that the completed structure or portion thereof will be subjected to in normal use.

~~**1714.3**~~ **1709.3 In-situ load tests.** In-situ load tests shall be conducted in accordance with Section ~~1714.3.1~~ 1709.3.1 or ~~1714.3.2~~ 1709.3.2 and shall be supervised by a *registered design professional*. The test shall simulate the applicable loading conditions specified in Chapter 16 as necessary to address the concerns regarding structural stability of the building, structure or portion thereof.

~~**1714.3.1**~~ **1709.3.1 Load test procedure specified.** Where a standard listed in Chapter 35 contains an applicable load test procedure and acceptance criteria, the test procedure and acceptance criteria in the standard shall apply. In the absence of specific load factors or acceptance criteria, the load factors and acceptance criteria in Section ~~1714.3.2~~ 1709.3.2 shall apply.

~~**1714.3.2**~~ **1709.3.2 Load test procedure not specified.** In the absence of applicable load test procedures contained within a standard referenced by this code or acceptance criteria for a specific material or method of construction, such *existing structure* shall be subjected to a test procedure developed by a *registered design professional* that simulates applicable loading and deformation conditions. For components that are not a part of the seismic-load-resisting system, the test load shall be equal to two times the unfactored design loads. The test load shall be left in place for a period of 24 hours. The structure shall be considered to have successfully met the test requirements where the following criteria are satisfied:

1. Under the design load, the deflection shall not exceed the limitations specified in Section 1604.3.
2. Within 24 hours after removal of the test load, the structure shall have recovered not less than 75 percent of the maximum deflection.
3. During and immediately after the test, the structure shall not show evidence of failure.

SECTION ~~1715~~ 1710
PRECONSTRUCTION LOAD TESTS

~~**1715.1**~~ **1710.1 General.** In evaluating the physical properties of materials and methods of construction that are not capable of being designed by *approved* engineering analysis or do not comply with applicable material design standards listed in Chapter 35, the structural adequacy shall be predetermined based on the load test criteria established in this section.

~~**1715.2**~~ **1710.2 Load test procedures specified.** Where specific load test procedures, load factors and acceptance criteria are included in the applicable design standards listed in Chapter 35, such test procedures, load factors and acceptance criteria shall apply. In the absence of specific test procedures, load factors or acceptance criteria, the corresponding provisions in Section ~~1715.3~~ 1710.3 shall apply.

~~**1715.3**~~ **1710.3 Load test procedures not specified.** Where load test procedures are not specified in the applicable design standards listed in Chapter 35, the load-bearing and deformation capacity of structural components and assemblies shall be determined on the basis of a test procedure developed by a *registered design professional* that simulates applicable loading and deformation conditions. For components and assemblies that are not a part of the seismic-force-resisting system, the test shall be as specified in Section ~~1715.3.1~~ 1710.3.1. Load tests shall simulate the applicable loading conditions specified in Chapter 16.

~~**1715.3.1**~~ **1710.3.1 Test procedure.** The test assembly shall be subjected to an increasing superimposed load equal to not less than two times the superimposed design load. The test load shall be left in place for a period of 24 hours. The tested assembly shall be considered to have successfully met the test requirements if the assembly recovers not less than 75 percent of the maximum deflection within 24 hours after the removal of the test load. The test assembly shall then be reloaded and subjected to an increasing superimposed load until either structural failure occurs or the superimposed load is equal to two and one-half times the load at which the deflection limitations specified in Section ~~1715.3.2~~ 1710.3.2 were reached, or the load is equal to two and one-half times the superimposed design load. In the case of structural components and assemblies for which deflection limitations are not specified in Section ~~1715.3.2~~ 1710.3.2, the test specimen shall be subjected to an increasing superimposed load until structural failure occurs or the load is equal to two and one-half times the desired superimposed design load. The allowable superimposed design load shall be taken as the lesser of:

1. The load at the deflection limitation given in Section ~~1715.3.2~~ 1710.3.2.
2. The failure load divided by 2.5.
3. The maximum load applied divided by 2.5.

~~**1715.3.2**~~ **1710.3.2 Deflection.** The deflection of structural members under the design load shall not exceed the limitations in Section 1604.3.

~~**1715.4**~~ **1710.4 Wall and partition assemblies.** *Load-bearing wall* and partition assemblies shall sustain the test load both with and without window framing. The test load shall include all design load components. Wall and partition assemblies shall be tested both with and without door and window framing.

~~**1715.5**~~ **1710.5 Exterior window and door assemblies.** The design pressure rating of exterior windows and doors in buildings shall be determined in accordance with Section ~~1715.5.1~~ 1710.5.1 or ~~1715.5.2~~ 1710.5.2.

> **Exception:** Structural wind load design pressures for window units smaller than the size tested in accordance with Section ~~1715.5.1~~ 1710.5.1 or ~~1715.5.2~~ 1710.5.2 shall be permitted to be higher than the design value of the tested unit provided such higher pressures are determined by accepted engineering analysis. All components of the small unit shall be the same as the tested unit. Where such calculated design pressures are used, they shall be validated by an additional test of the window unit having the highest allowable design pressure.

~~**1715.5.1**~~ **1710.5.1 Exterior windows and doors.** Exterior windows and sliding doors shall be tested and labeled as conforming to AAMA/WDMA/CSA101/I.S.2/A440. The *label* shall state the name of the manufacturer, the *approved* labeling agency and the product designation as specified in AAMA/ WDMA/CSA101/I.S.2/A440. Exterior side-hinged doors shall be tested and *labeled* as conforming to AAMA/WDMA/CSA101/I.S.2/A440 or comply with Section ~~1715.5.2~~ 1710.5.2. Products tested and labeled as conforming to AAMA/WDMA/CSA 101/I.S.2/A440 shall not be subject to the requirements of Sections 2403.2 and 2403.3.

~~**1715.5.2**~~ **1710.5.2 Exterior windows and door assemblies not provided for in Section ~~1715.5.1~~ 1710.5.1.** Exterior window and door assemblies shall be tested in accordance with ASTM E 330. Structural performance of garage doors shall be determined in accordance with either ASTM E 330 or ANSI/DASMA 108, and shall meet the acceptance criteria of ANSI/DASMA 108. Exterior window and door assemblies containing glass shall comply with Section 2403. The design pressure for testing shall be calculated in accordance with Chapter 16. Each assembly shall be tested for 10 seconds at a load equal to 1.5 times the design pressure.

~~**1715.6**~~ **1710.6 Test specimens.** Test specimens and construction shall be representative of the materials, workmanship and details normally used in practice. The properties of the materials used to construct the test assembly shall be determined on the basis of tests on samples taken from the load assembly or on representative samples of the materials used to construct the load test assembly. Required tests shall be conducted or witnessed by an *approved agency*.

SECTION ~~1716~~ 1711
MATERIAL AND TEST STANDARDS

~~**1716.1**~~ **1711.1 Test standards for joist hangers and connectors.**

~~**1716.1.1**~~ **1711.1.1 Test standards for joist hangers.** The vertical load-bearing capacity, torsional moment capacity and deflection characteristics of joist hangers shall be determined in accordance with ASTM D 1761 using lumber having a specific gravity of 0.49 or greater, but not greater than 0.55, as determined in accordance with AF&PA NDS for the joist and headers.

Exception: The joist length shall not be required to exceed 24 inches (610 mm).

~~**1716.1.2**~~ **1711.1.2 Vertical load capacity for joist hangers.** The vertical load capacity for the joist hanger shall be determined by testing a minimum of three joist hanger assemblies as specified in ASTM D 1761. If the ultimate vertical load for any one of the tests varies more than 20 percent from the average ultimate vertical load, at least three additional tests shall be conducted. The allowable vertical load of the joist hanger shall be the lowest value determined from the following:

1. The lowest ultimate vertical load for a single hanger from any test divided by three (where three tests are conducted and each ultimate vertical load does not vary more than 20 percent from the average ultimate vertical load).
2. The average ultimate vertical load for a single hanger from all tests divided by three (where six or more tests are conducted).
3. The average from all tests of the vertical loads that produce a vertical movement of the joist with respect to the header of 1/8 inch (3.2 mm).
4. The sum of the allowable design loads for nails or other fasteners utilized to secure the joist hanger to the wood members and allowable bearing loads that contribute to the capacity of the hanger.
5. The allowable design load for the wood members forming the connection.

~~**1716.1.3**~~ **1711.1.3 Torsional moment capacity for joist hangers.** The torsional moment capacity for the joist hanger shall be determined by testing at least three joist hanger assemblies as specified in ASTM D 1761. The allowable torsional moment of the joist hanger shall be the average torsional moment at which the lateral movement of the top or bottom of the joist with respect to the original position of the joist is 1/8 inch (3.2 mm).

~~**1716.1.4**~~ **1711.1.4 Design value modifications for joist hangers.** Allowable design values for joist hangers that are determined by Item 4 or 5 in Section ~~1716.1.2~~ 1711.1.2 shall be permitted to be modified by the appropriate duration of loading factors as specified in AF&PA NDS but shall not exceed the direct loads as determined by Item 1, 2 or 3 in Section ~~1716.1.2~~ 1711.1.2. Allowable design values determined by Item 1, 2 or 3 in Section ~~1716.1.2~~ 1711.1.2 shall not be modified by duration of loading factors.

~~**1716.2**~~ **1711.2 Concrete and clay roof tiles.**

~~**1716.2.1**~~ **1711.2.1 Overturning resistance.** Concrete and clay roof tiles shall be tested to determine their resistance to overturning due to wind in accordance with SBCCI SSTD 11 and Chapter 15.

~~**1716.2.2**~~ **1711.2.2 Wind tunnel testing.** When roof tiles do not satisfy the limitations in Chapter 16 for rigid tile, a wind tunnel test shall be used to determine the wind characteristics of the concrete or clay tile roof covering in accordance with SBCCI SSTD 11 and Chapter 15.

Reason: The current chapter organization includes actual construction material inspection requirements (e.g. concrete inspection) sandwiched between sections related to the statement of special inspection and other procedural requirements. The result is difficulty in implementing a project specific special inspection program and potential for misinterpretation. This proposal relocates provisions contained in sections 1705, 1709, and 1710 into 1704 to increase clarity of intent and provide more ease of use. Note that current sections 1709 (Contractor Responsibility) and 1710 (Structural Observations) are included in the new 1704 since they are part of the overall program to assure that construction is in accordance with the design intent. The reorganization renames two sections to reflect updated content within the sections and renumbers sections following 1704, but does not make changes to tables or overall chapter content. This reorganization follows a more logical and consistent order as follows:

1701 GENERAL – unchanged
1702 DEFINITIONS – unchanged
1703 APPROVALS – unchanged
1704 SPECIAL INSPECTIONS, CONTRACTOR RESPONSIBILITY AND STRUCTURAL OBSERVATIONS – outlines the requirements for special inspection, the Statement of Special Inspections, Contractor Responsibility and Structural Observation, prior to the listing of the actual special inspections required by material. This Section now includes portions of old 1704, and all of old 1705, 1709 and 1710
1705 – REQUIRED VERIFICATION AND INSPECTION – outlines the material-specific inspections and testing requirements. It will include most of old Section1704 as well as all of old Sections 1706, 1707 and 1708.
1706 – DESIGN STRENGTHS OF MATERIALS – old Section 1711 unchanged
1707 – ALTERNATIVE TESTING PROCEDURE – old Section 1712 unchanged
1708 – TEST SAFE LOAD – old Section 1713 unchanged
1709 – IN-SITU LOAD TESTS - old Section 1714 unchanged
1710 - PRECONSTRUCTION LOAD TESTS - old Section 1715 unchanged
1711 – MATERIAL AND TEST STANDARDS - old Section 1716 unchanged

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

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

FILENAME: HARMAN-S6-CHAPTER 17

S117–09/10
1704.1

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

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 specified in Section 109.

The special inspector shall be a qualified person who shall demonstrate competence, to the satisfaction of the building official, for the inspection of the particular type of construction or operation requiring special inspection. The registered design professional in responsible charge and engineers of record involved in the design of the project are permitted to act as the approved agency and their personnel are permitted to 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 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 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~~2. 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.

.

Reason: Exception #2 of Section 1704.1 exempts from special inspection building components not designed by a registered design professional such as an architect or structural engineer. The exception is being deleted because "building component" is not defined, its meaning is vague and its application is subject to a wide variation in interpretation. If an exception for building components was warranted for which justification is not apparent, it would appear that building components designed by a registered design professional would be exempt due to the expertise provided by the registered design professional in its design; building components designed by other than a registered design professional would not be exempt. The need for special inspection of a building component should be determined based on the current requirements for special inspection in Sections 1704, 1706, 1707 and 1708. Special inspection for a building component may not be warranted but that determination should be made by applying Exception #1 of Section 1704.1 for work of a minor nature or as warranted by local conditions.

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

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

ICCFILENAME: Brazil-S22-1704.1

S118–09/10
1704.1

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. <u>Special inspections are not required for portions of structures designed and constructed in accordance with the conventional light-frame construction provisions of Section 2308.</u>

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.

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

ICCFILENAME: Ehrlich-S1-1704 1

S119–09/10
1704.1.1 (New)

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

Add new text as follows:

<u>**1704.1.1 Access for special inspection.** The construction or work for which special inspection is required shall remain accessible and exposed for special inspection purposes until completion of the required special inspections.</u>

(Renumber subsequent sections)

Reason: Section 110.1 requires construction or work requiring inspection to remain accessible and exposed for inspection purposes until approved. The IBC does not have a comparable statement for special inspections and this proposal provides it.

For inspections by the building official, the requirement that the construction or work remain accessible and exposed until approved is derived from the powers and duties entrusted to building officials by the jurisdictions they serve and which are specified in Section 104. A special inspector, however, is typically not a public official but a private individual providing third-party special inspection services to an owner or owner's authorized agent. A special inspector is typically not authorized to approve construction or work and relies on the building official for that authority. The lack of the authority by the special inspector to approve construction or work is why the requirement for construction or work to remain accessible and exposed is being proposed.

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

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

ICCFILENAME: Brazil-S26-1704.1.1

S120–09/10

1704.1.1, 1705.1

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

Revise as follows:

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 permit issuance. This statement shall be in accordance with Section 1705.

Exception~~s~~:

~~1.~~ A statement of special inspections is not required for structures designed and constructed in accordance with the conventional 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.~~

1705.1 General. Where special inspection or testing is required by Section 1704, 1707 or 1708, the registered design professional in responsible charge shall prepare a statement of special inspections in accordance with Section ~~1705~~ 1705.2 for submittal by the applicant ~~(see~~ in accordance with Section 1704.1.1~~)~~.

Exception: 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.

Reason: The purpose for this proposal is to improve the scoping provisions applicable to the statement of special inspections. Section 1705.1 requires the statement of special inspections to be prepared by the registered design professional in responsible charge where special inspection or testing is required elsewhere in Chapter 17. Exception #2 of Section 1704.1.1, however, permits the statement to be prepared by a qualified person approved by the building official for construction not designed by a registered design professional. The charging text of Section 1704.1.1 requires submittal of a statement of special inspections prepared by the registered design professional in responsible charge "in accordance with Section 107.1." Section 107.1, however, contains requirements for submission of documents with each permit application.

The proposal deletes from the charging text of Section 1704.1.1 preparation of the statement of special inspections by the registered design professional in responsible charge because the requirements for preparation are in Section 1705 and Section 1704.1.1 references Section 1705. Section 1704.1.1, then, becomes a requirement for submittal of the statement of special inspections in accordance with Section 107.1. Exception #1 of Section 1704.1.1 is retained because it exempts structures designed and constructed in accordance with the conventional construction provisions of Section 2308 from the requirement for **submittal** (emphasis mine) of a statement of special inspections. Exception #2, however, is relocated to Section 1705.1 because it exempts structures not designed by a registered design professional from the requirement for **preparation** (emphasis mine) of a statement of special inspections by the registered design professional in responsible charge.

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

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

ICCFILENAME: Brazil-S32-1704.1.1

S121–09/10

1704.3-1704.3.3.3, Table 1704.3

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~~s~~:

~~1.~~ *Special inspection* of the steel fabrication process shall not be required where the fabricator does not perform any welding, thermal cutting or heating operation of any kind as part of the fabrication process. In such cases, the fabricator shall be required to submit a detailed procedure for material control that demonstrates the fabricator's ability to maintain suitable records and procedures such that, at any time during the fabrication process, the material specification~~,~~ 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 boltsrequiring 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~~	—		
1~~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, i~~Identification markings to conform to ASTM standards specified in the approved construction documents.	—	X	Applicable ASTM material standards	
b~~c~~. Manufacturers' certified test reports.	—	X		
~~4. Material verification of weld filler materials:~~				
~~a. Identification markings to conform to AWS specification in the approved construction documents.~~	~~—~~	~~X~~	~~AISC 360, Section A3.5 and Applicable AWS A5 documents~~	~~—~~
~~b. Manufacturer's certificate of compliance required.~~	—	~~X~~	—	—
2~~5~~. Inspection of welding: a. ~~Structural steel and c~~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~~		
1~~6~~) Floor and roof deck welds.	—	X	AWS D1.3	
b. Reinforcing steel:				

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]	IBC REFERENCE
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.

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

ICCFILENAME: Manley-S13-1704.3

S122–09/10
Table 1704.4

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

Revise as follows:

TABLE 1704.4
REQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION

VERIFICATION AND INSPECTION	CONTIN-UOUS	PERIODIC	REFERENCED STANDARD [a]	IBC REFERENCE
3. Inspection of ~~bolts to be installed in concrete prior to and during placement of~~ anchors cast in concrete where allowable loads have been increased or where strength design is used.	~~X~~ —	— X	ACI 318; 8.1.3, 21.2.8	1911.5, 1912.1
4. Inspection of anchors ~~installed in~~ post-installed in hardened concrete members and designed in accordance with Section 1912. [b]	X	~~X~~ —	ACI 318; 3.8.6, 8.1.3, 21.2.8	1912.1
5. Inspection of anchors post-installed in hardened concrete members and qualified for installation through Section 104.11.	Note b	Note b		

(Portions of table not shown do not change)

a. Where applicable, see also Section 1707.1, Special inspection for seismic resistance.
b. Special inspection of anchors qualified for installation through Section 104.11 shall be conducted in accordance with the requirements specified in the report of qualification, such as an evaluation report issued by the ICC Evaluation Service.

Reason: The purpose for this proposal is to adjust the required extent of special inspection for concrete anchors to be more consistent with the access available for special inspectors to perform their duties. Anchors that are cast into concrete are visible for inspection from their installation until the concrete is placed. In this regard, their inspection is similar to that of concrete reinforcement for which Table 1704.3 currently specifies periodic special inspection. It is sufficient for special inspectors to be present intermittently during installation of the cast-in anchors, provided they are able to perform their inspections prior to placement of the concrete after which effective inspection is severely diminished.

Anchors that are post-installed into hardened concrete, however, require the presence of the special inspector during their installation to verify that the anchors are installed in accordance with the construction documents, the project specifications and installation instructions by the manufacturer of the anchors. In this regard, their inspection is similar to that of concrete placement for which Table 1704.3 currently specifies continuous special inspection. They may not need to be present continuously during installation of all the post-installed anchors but their presence is needed during the installation of a certain percentage of them and the extent and frequency of the special inspection should be agreed upon beforehand by all affected parties, including, but not limited to, the owner or owner's representative, contractor, special inspector and the building official.

Item #3 on bolts to be installed in concrete currently applies to bolts designed using (1) allowable stress design procedures and increases in allowable loads, which is primarily intended for increases permitted by IBC Section 1911.5, and (2) strength design procedures for which IBC Section 1912.1 requires compliance with Appendix D of ACI 318 as modified therein. The references to IBC Sections 1911.5 and 1912.1 at the column of IBC references account for this. Section 1912.1 governs the "strength design of anchors installed in concrete for purposes of transmitting structural loads from one connected element to the other." Compliance with Appendix D of ACI 318 is required for anchors that are within its scope. For all other anchors, design "in accordance with an approved procedure" is required.

Item #3 is changed to "anchors cast in concrete" for consistency with similar terms used in IBC Sections 1911.5 and 1912.1 and Appendix D of ACI 318 and because it is seen as a more general term than "bolts to be installed in concrete." The requirement for continuous special inspection is changed to periodic special inspection as discussed in the first paragraph of this statement. Section D.1 in Appendix D of ACI 318 defines "cast-in anchor" as "a headed bolt, headed stud or hooked bolt installed before placing concrete" and Section D2.2 includes post-installed anchors in its scoping statement. Section D.1 of Appendix D also defines "anchor" as "a steel element either cast into concrete or post-installed into a hardened concrete member and used to transmit applied loads, including headed bolts, hooked bolts (J- or L-bolt), headed studs, expansion anchors or undercut anchors."

Item #4 on anchors installed in hardened concrete applies to anchors complying with IBC Section 1912 for which Section 1912.1 requires the use of strength design procedures in accordance with Appendix D of ACI 318 as modified therein. The reference to IBC Section 1912.1 at the column of IBC references accounts for this.

Item #4 is changed to "anchors post-installed in hardened concrete" for consistency with Appendix D of ACI 318 and to more clearly distinguish them from "anchors cast in concrete." The requirement for periodic special inspection is changed to continuous special inspection as discussed in the second paragraph of this statement. Section D.1 in Appendix D of ACI 318 defines "post-installed anchor" as "an anchor installed in hardened concrete" and Section D2.2 includes cast-in anchors in its scoping statement.

Item #4 is also revised by adding a reference to IBC Section 1912.1. Adding the reference would not be necessary if Section 1912.1 applied to all post-installed anchors designed with strength design procedures and continuous special inspection as defined in Section 1702.1 was sufficient to specify the frequency and extent to which special inspection is needed. This is considered necessary for anchors post-installed into hardened concrete that are designed and installed in accordance with IBC Section 1912.1 but it is not always considered necessary for anchors post-installed into hardened concrete that are qualified for use in accordance with Section 104.11 on alternative means and methods through nationally recognized evaluation services, such as the ICC Evaluation Service (ICC-ES).

The evaluation reports issued by ICC-ES frequently identify requirements for special inspection by referencing the requirements of IBC Chapter 17 for that purpose. Some evaluation reports, however, specify in detail what is required of a special inspector. Extent and frequency are often included. As long as this is limited to the evaluation report, it improves the process of special inspection. Many of these evaluation reports with these detailed requirements, however, also specify periodic or continuous special inspection and reference IBC Chapter 17 in the process. When this is done, compliance with applicable special inspection requirements in the report and in IBC Chapter 17 are assumed and, presumably, expected. The threshold between continuous and periodic special inspection in the IBC versus these evaluation reports, however, is different. IBC Chapter 17 relies on the definitions of "continuous special inspection" and "periodic special inspection" to distinguish between them. The evaluation reports that specify detailed special inspection requirements rely on these detailed requirements. Where these reports also specify periodic or continuous special inspection and reference IBC Chapter 17, however, they create conflicts between the report and the IBC if the threshold between continuous and periodic special inspection is different in the evaluation report than it would be if the report did not exist and the anchor was designed and installed for compliance with the IBC without the benefit of an evaluation report.

Ideally, where detailed requirements for special inspection are included in an ICC-ES evaluation report, they would be specified insufficient detail that there would be no mention of continuous or periodic special inspection. The detailed requirements would effectively serve the same purpose as the distinction between continuous and periodic special inspection serves in the IBC.

Conflicts now exist between post-installed concrete anchors qualified by Section 104.11 through an ICC-ES evaluation report versus such anchors designed and installed to meet the requirements of the IBC without the benefit of an evaluation report. Consider concrete adhesive anchors as an example. Based on the definitions in Section 1702.1, continuous special inspection is considered necessary because an inspector must be present during the installation of the anchor to effectively perform an inspection. The typical evaluation report for a concrete adhesive anchor, however, specifies periodic special inspection. Where special inspection for the anchor is conducted in accordance with the detailed requirements in such a report, effective inspection can occur. But if special inspection for the anchor is conducted in accordance with Table 1704.4 of the 2009 IBC based on the definition of periodic inspection in Section 1702.1, effective inspection can not occur because it may be too late to effectively inspect a concrete adhesive anchor once installation has begun.

Item #5 is added to (1) account for post-installed anchors qualified for use in accordance with Section 104.11 through nationally recognized evaluation services, such as the ICC Evaluation Service (ICC-ES), and to (2) distinguish between the requirements for special inspection of anchors designed to comply with the IBC alone versus those qualified for use through evaluation services such as ICC-ES.

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

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

ICCFILENAME: Brazil-S51-1704.4

S123-09/10
1704.5

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:

1704.5 Masonry construction. Masonry construction shall be inspected and verified in accordance with <u>TMS 402/ACI 530/ASCE 5 and TMS 602/ACI 530.1/ASCE 6 quality assurance program requirements</u>. ~~the requirements of Sections 1704.5.1 through 1704.5.3, depending on the occupancy category of the building or structure.~~

Exception: Special inspections shall not be required for:

1. Empirically designed masonry, glass unit masonry or masonry veneer designed by Section 2109, 2110 or Chapter 14, respectively, ~~or by Chapter 5, 7 or 6 of TMS 402/ACI 530/ASCE 5, respectively,~~ when they are part of structures classified as Occupancy Category I, II or III in accordance with Section 1604.5.
2. Masonry foundation walls constructed in accordance with Table 1807.1.6.3(1), 1807.1.6.3(2), 1807.1.6.3(3) or 1807.1.6.3(4).
3. Masonry fireplaces, masonry heaters or masonry chimneys installed or constructed in accordance with Section 2111, 2112 or 2113, respectively.

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

2. Delete without substitution:

~~**1704.5.2 Engineered masonry in Occupancy Category I, II or III.** The minimum special inspection program for masonry designed by Section 2107 or 2108 or by chapters other than Chapter 5, 6 or 7 of TMS402/ACI 530/ASCE 5 in structures classified as Occupancy Category I, II or III, in accordance with Section 1604.5, shall comply with Table 1704.5.1.~~

~~**1704.5.3 Engineered masonry in Occupancy Category IV.** The minimum special inspection program for masonry designed by Section 2107 or 2108 or by chapters other than Chapter 5, 6 or 7 of TMS402/ACI 530/ASCE 5 in structures classified as Occupancy Category IV, in accordance with Section 1604.5, shall comply with Table 1704.5.3.~~

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

~~VERIFICATION AND INSPECTION~~	~~FREQUENCY OF INSPECTION~~		~~REFERENCE FOR CRITERIA~~		
	~~CONTINUOUS~~	~~PERIODIC~~	~~IBC SECTION~~	~~TMS 402/ACI 530/ASCE 5a~~	~~TMS 602/ACI 530.1/ASCE 6a~~
~~1. Compliance with required inspection pro-visions of the construction documents and the approved submittals shall be verified.~~	—	~~X~~	—	—	~~Art. 1.5~~
~~2. Verification of f'_m and f'_{AAC} prior to con-struction except where specifically exempted by this code.~~	—	~~X~~	—	—	~~Art. 1.4B~~
~~3. Verification of slump flow and VSI as delivered to the site for self-consolidating grout.~~	~~X~~	—	—	—	~~Art. 1.5B.1.b.3~~
~~4. As masonry construction begins, the following shall be verified to ensure compliance:~~					
~~a. Proportions of site-prepared mortar.~~	—	~~X~~	—	—	~~Art. 2.6A~~
~~b. Construction of mortar joints.~~	—	~~X~~	—	—	~~Art. 3.3B~~
~~c. Location of reinforcement, connectors, prestressing tendons and anchorages.~~	—	~~X~~	—	—	~~Art. 3.4, 3.6A~~
~~d. Prestressing technique.~~	—	~~X~~	—	—	~~Art. 3.6B~~
~~e. Grade and size of prestressing tendons and anchorages.~~	—	~~X~~	—	—	~~Art. 2.4B, 2.4H~~
~~5. During construction the inspection pro-gram shall verify:~~					
~~a. Size and location of structural elements.~~	—	~~X~~	—	—	~~Art. 3.3F~~
~~b. Type, size and location of anchors, including other details of anchorage of masonry to structural members, frames or other construction.~~	—	~~X~~	—	~~Sec. 1.2.2(e), 1.16.1~~	—
~~c. Specified size, grade and type of reinforcement, anchor bolts, prestressing tendons and anchorages.~~	—	~~X~~	—	~~Sec. 1.15~~	~~Art. 2.4, 3.4~~
~~d. Welding of reinforcing bars.~~	~~X~~	—	—	~~Sec. 2.1.9.7.2, 3.3.3.4(b)~~	—

~~VERIFICATION AND INSPECTION~~	~~FREQUENCY OF INSPECTION~~		~~REFERENCE FOR CRITERIA~~		
	~~CONTINUOUS~~	~~PERIODIC~~	~~IBC SECTION~~	~~TMS 402/ACI 530/ASCE 5a~~	~~TMS 602/ACI 530.1/ASCE 6a~~
~~e. Preparation, construction and protection of masonry during cold weather (temperature below 40°F) or hot weather (temperature above 90°F).~~	—	~~X~~	~~Sec. 2104.3, 2104.4~~		
~~f. Application and measurement of prestressing force.~~	~~X~~	—	—	—	~~Art. 1.8C, 1.8D~~
~~6. Prior to grouting, the following shall be verified to ensure compliance:~~	~~X~~	—	—	—	~~Art. 3.6B~~
~~a. Grout space is clean.~~					—
~~b. Placement of reinforcement and connectors, and prestressing tendons and anchorages.~~	—	~~X~~	—	— ~~Sec. 1.13~~	~~Art. 3.2D~~ ~~Art. 3.4~~
~~c. Proportions of site-prepared grout and prestressing grout for bonded tendons.~~	—	~~X~~	—	—	~~Art. 2.6B~~
~~d. Construction of mortar joints.~~	—	~~X~~	—		
~~7. Grout placement shall be verified to ensure compliance:~~	~~X~~	—	—	—	~~Art. 3.3B~~
~~a. Grouting of prestressing bonded tendons.~~	~~X~~	—	—	—	~~Art. 3.5~~
~~8. Preparation of any required grout speci-mens, mortar specimens and/or prisms shall be observed.~~	—			—	~~Art. 3.6C~~

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

TABLE 1704.5.3
LEVEL 2 REQUIRED VERIFICATION AND INSPECTION OF MASONRY CONSTRUCTION

~~VERIFICATION AND INSPECTION~~	~~CONTINUOUS~~	~~PERIODIC~~	~~REFERENCE FOR CRITERIA~~		
			~~IBC SECTION~~	~~TMS 402/ACI 530/ASCE 5a~~	~~TMS 602/ACI 530.1/ASCE 6a~~
~~1. Compliance with required inspection provisions of the construction documents and the approved submittals.~~	—	~~X~~	—	—	~~Art. 1.5~~
~~2. Verification of f'_m and f'_{AAC} prior to construction and for every 5,000 square feet during construction.~~	—	~~X~~	—	—	~~Art. 1.4B~~
~~3. Verification of proportions of materials in premixed or preblended mortar and grout as delivered to the site.~~	—	~~X~~	—	—	~~Art. 1.5B~~
~~4. Verification of slump flow and VSI as delivered to the site for self-consolidating grout.~~	~~X~~	—	—	—	~~Art. 1.5B.1.b.3~~
~~5. The following shall be verified to ensure compliance:~~					
~~a. Proportions of site-prepared mortar, grout and prestressing grout for bonded tendons.~~	—	~~X~~	—	—	~~Art. 2.6A~~
~~b. Placement of masonry units and construction of mortar joints.~~	—	~~X~~	—	—	~~Art. 3.3B~~
~~c. Placement of reinforcement, connectors and prestressing tendons and anchorages.~~	—	~~X~~	—	~~Sec. 1.15~~	~~Art. 3.4, 3.6A~~
~~d. Grout space prior to grout.~~	~~X~~	—	—	—	~~Art. 3.2D~~
~~e. Placement of grout.~~	~~X~~	—	—	—	~~Art. 3.5~~
~~f. Placement of prestressing grout.~~	~~X~~	—	—	—	~~Art. 3.6C~~
~~g. Size and location of structural elements.~~	—	~~X~~	—	—	~~Art. 3.3F~~
~~h. Type, size and location of anchors, including other details of anchorage of masonry to structural members, frames or other construction.~~	~~X~~	—	—	~~Sec.1.2.2(e), 1.16.1~~	—
~~i. Specified size, grade and type of reinforcement, anchor bolts, prestressing tendons and anchorages.~~	—	~~X~~	—	~~Sec. 1.15~~	~~Art. 2.4, 3.4~~
~~j. Welding of reinforcing bars.~~	~~X~~	—	—	~~Sec. 2.1.9.7.2, 3.3.3.4 (b)~~	—
~~k. Preparation, construction and protection of masonry during cold weather (temperature below 40°F) or hot weather (temperature above 90°F).~~	—	~~X~~	~~Sec. 2104.3, 2104.4~~	—	~~Art. 1.8C, 1.8D~~
~~l. Application and measurement of prestressing force.~~	~~X~~	—	—	—	~~Art. 3.6B~~
~~6. Preparation of any required grout specimens and/or prisms shall be observed.~~	~~X~~	—	~~Sec. 2105.2.2, 2105.3~~	—	~~Art. 1.4~~

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

Reason:
Synopsis: Remove IBC code requirements that are already specified in standards. TMS 402/ACI 530/ASCE 5 and TMS 602/ACI 530.1/ASCE 6 Building Code Requirements for Masonry Structures provides requirements for quality assurance of masonry construction.

Detailed Discussion: The basis for design and construction of masonry structures as provided in Chapter 21 of IBC is TMS 402/ACI 530/ASCE 5 and TMS 602/ACI 530.1/ASCE 6, Building Code Requirements for Masonry Structures. Therefore, this proposal seeks to provide the special inspection provided in these standards which is consistent with the design and construction procedures in the code. This proposal coordinates the special inspections with the design procedures that were adopted in the last code development cycle.

1. Section 1.18 of TMS 402/ACI 530/ASCE 5 Building *Code Requirements for Masonry Structures* and Article 1.6 of TMS 602/ACI 530/ASCE 6 *Specification for Masonry Structures* includes requirements for tests, inspections and verifications of masonry constructions. The tests, inspections and verifications within TMS 402 are inclusive of those tests, inspections and verifications in the current International Building Code.
2. Deletion of “or by Chapter 5, 7 or 6 of TMS 402/ACI 530/ASCE 5, respectively,” referenced in the 1st exception and section 1704.5.1 is to avoid redundancy since any masonry designed in accordance with Chapters 5, 6 or 7 of TMS 402 is subject to a quality assuranceprogram specified in Section 1.18 of TMS 402.
3. Reference is made to Level B Quality Assurance requirements specified in TMS 402 for the list of tests, inspections and verifications required for masonry designed in accordance with IBC Sections 2109 2110 and Chapter 14.
4. Sections 1704.5.2 and 1704.5.3 are deleted in entirety since all masonry designed in accordance IBC Sections 2107 and 2108 must comply with Chapter 1 of TMS 402. TMS 402 Chapter 1 requires that masonry construction must be tested, inspected and verified.
5. Tables 1704.5.1 and 1704.5.3 are deleted because there is no reference to them and all tests, inspections and verifications are identified in TMS 402.

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

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

FILENAME: Harman-S3-1704.5

S124–09/10

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

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

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC
1. Verify that grade stamp on framing lumber, plywood and OSB panels conforms to the construction documents.		X
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.		X
3. Inspect details of wood framing including framing layout, member sizes, blocking, bridging and bearing lengths.		X
4. Inspect diaphragms and shear walls to verify that		X

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.		

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	–		

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERENCED STANDARD[a]	IBC REFERENCE
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		X	AWS D1.3	
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		X	Applicable ASTM material standards	

(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.

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

ICCFILENAME: Harman-S2-1704

S125–09/10
Table 1704.7

Proponent: Lori A. Simpson, Treadwell & Rollo, Inc., representing self

Revise as follows:

TABLE 1704.7
REQUIRED VERIFICATION AND INSPECTION OF SOILS

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

Reason: The amount of time spent verifying use of proper materials, densities and lift thickness during placement and compaction of controlled fill should be at the discretion of the Geotechnical Engineer. Periodic visits can be sufficient to confirm relative compaction achieved on each lift. The amount of time spent on site and the number of visits will depend on the amount of fill placed per day. It is not necessary to be on site full time during grading, as part-time observation and periodic tests can sufficiently confirm the adequacy of the compaction.

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

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

ICCFILENAME: SIMPSON-S1-TABLE 1704.7

S126–09/10
1704.15 (New), 1704.15.1 (New), 1704.15.2 (New), Chapter 35

Proponent: Tony Crimi, A.C. Consulting Solutions Inc., representing International Firestop Council

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

1. Add new text as follows:

1704.15 Fire-resistant penetrations and joints. Special inspections for through penetrations, membrane penetrations, joints, 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. Special inspections shall be based on the fire-resistance design or system as designated in the approved construction documents.

1704.15.1 Fire-resistant penetrations. Penetration firestop systems in fire-resistance-rated assemblies shall not be concealed from view until inspected and approved. Inspections of penetration firestop systems of the types specified in Sections 713.3.1.2 and 713.4.1.2 are permitted to be conducted by an approved inspection agency in accordance with ASTM E2174.

1704.15.2 Fire-resistive joints. Fire resistant joint systems within, or at the perimeter of, fire-resistance-rated assemblies shall not be concealed from view until inspected and approved. Inspection of joints of the types specified in Sections 714.3 and 714.4 are permitted to be conducted by an approved inspection agency in accordance with ASTM E2393.

2. Add standard to IBC Chapter 35 as follows.

ASTM
ASTM E2174-04 Standard Practice for On-site Inspection of Installed Fire Stops
ASTM E2393-04, Standard practice for On-Site Inspection of Installed Fire Resistive Joint Systems and Perimeter Fire Barriers

Reason: The purpose of this proposal adds a reference to two Consensus Standards developed at ASTM for inspection of installed penetration firestop systems, fire-resistive joints, and perimeter fire barriers. The Code already mandates proper installation of penetration firestops to maintain the integrity of vertical and horizontal fire or smoke separations. Section 1704 of the *International Building Code*® (IBC) provides for special inspection agencies. Under the IBC, final authority for recognition of special inspection agencies rests with the building official having jurisdiction. These Standards identify proper methods for the field inspection of these systems, and provides consistent procedures needed to conduct and document the on-site assessment of the installations.

Substantiation: Firestop and joint system designs and materials are increasing in number and variety. The current code relies heavily on Installers, Designers, and Code Officials to verify proper system selection and installation. In response to this reality, a standard practice was developed within the ASTM process to allow inspections of through-penetration firestops, joints, and perimeter fire barrier systems to be conducted in a thorough and consistent manner, with standardized report formats, regardless of the Trade or individual conducting the inspection. Part of the impetus for the development of that standard was the recognition that jurisdictions sometimes do not have sufficient resources themselves to ensure that all penetrations and joints are firestopped properly. In any project, the number of joints and penetrations can range from hundreds to a few thousand in a single building. The addition of these new Standards to the Code would provide and identify a means for authorities having jurisdiction to have effective tools to mandate standardized inspection thoroughness and quality third party inspection agencies are used for verification of these important systems. The inclusion of consensus standards would ensure that required inspections are conducted consistently, fairly, and adequately, while also standardizing inspection reports, so that they will be of a uniform high quality.

The proposed code change would provide the code official the option of having a third party (e.g. approved inspection agency) to conduct the inspection of joints and penetrations in conformance with these Standards, while preserving the option to utilize other policies and procedures consistent with the intent of the Code.

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

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM E2174-04 and ASTM E2393-04, 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.

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

ICCFILENAME:CRIMI-S1-1704.15

S127–09/10
1704.15 (New), Chapter 35

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

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

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 ½ 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 ASTME 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.

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

ICCFILENAME: KOFFELL-S3-1704.15 NEW

S128–09/10
1704.15 (New), Chapter 35

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

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

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 ½ 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 ASTME 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.

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

ICCFILENAME: KOFFELL-S4-1704.15 NEW

S129–09/10

1613.4, 1705.1, 1705.2, 1705.3-1705.3.6, 1705.4-1705.4.2, 1707.1, 1708.1

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

1. Revise as follows:

1705.1 General. Where special inspection or testing is required by Section 1704, 1706, 1707 or 1708, the registered design professional in responsible charge shall prepare a statement of special inspections in accordance with Section 1705 for submittal by the permit applicant (see Section 1704.1.1).

1705.2 Content of statement of special inspections. The statement of special inspections shall identify the following:

1. The materials, systems, components and work required to have special inspection or testing by the building official or by the registered design professional responsible for each portion of the work.
2. The type and extent of each special inspection.
3. The type and extent of each test.
4 Additional requirements for special inspection or testing for seismic or wind resistance as specified in Sections ~~1705.3, 1705.4~~ 1706, 1707~~,~~ ~~or~~ and 1708.
5. For each type of special inspection, identification as to whether it will be continuous special inspection or periodic special inspection.

2. Delete without substitution:

~~**1705.3 Seismic resistance.** The statement of special inspections shall include seismic requirements for cases covered in Sections 1705.3.1 through 1705.3.5.~~

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

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

~~**1705.3.1 Seismic-force-resisting systems.** The seismic-force-resisting systems in structures assigned to Seismic Design Category C, D, E or F, in accordance with Section 1613.~~

~~**Exception:** Requirements for the seismic-force resisting system are permitted to be excluded from the statement of special inspections for steel systems in structures assigned to Seismic Design Category C that are not specifically detailed for seismic resistance, with a response modification coefficient, *R*, of 3 or less, excluding cantilever column systems.~~

~~**1705.3.2 Designated seismic systems.** Designated seismic systems in structures assigned to Seismic Design Category D, E or F.~~

~~**1705.3.3 Seismic Design Category C.** The following additional systems and components in structures assigned to Seismic Design Category C:~~

~~1. Heating, ventilating and air-conditioning (HVAC) ductwork containing hazardous materials and anchorage of such ductwork.~~
~~2. Piping systems and mechanical units containing flammable, combustible or highly toxic materials.~~
~~3. Anchorage of electrical equipment used for emergency or standby power systems.~~

~~**1705.3.4 Seismic Design Category D.** The following additional systems and components in structures assigned to Seismic Design Category D:~~

~~1. Systems required for Seismic Design Category C.~~
~~2. Exterior wall panels and their anchorage.~~
~~3. Suspended ceiling systems and their anchorage.~~
~~4. Access floors and their anchorage.~~
~~5. Steel storage racks and their anchorage, where the importance factor is equal to 1.5 in accordance with Section 15.5.3 of ASCE 7.~~

~~**1705.3.5 Seismic Design Category E or F.** The following additional systems and components in structures assigned to Seismic Design Category E or F:~~

1. ~~Systems required for Seismic Design Categories C and D.~~
2. ~~Electrical equipment.~~

~~**1705.3.6 Seismic requirements in the statement of special inspections.** When Sections 1705.3 through 1705.3.6 specifies that seismic requirements be included, the statement of special inspections shall identify the following:~~

1. ~~The designated seismic systems and seismic-force-resisting systems that are subject to special inspections in accordance with Sections 1705.3 through 1705.3.6.~~
2. ~~The additional special inspections and testing to be provided as required by Sections 1707 and 1708 and other applicable sections of this code, including the applicable standards referenced by this code.~~

~~**1705.4 Wind resistance.** The statement of special inspections shall include wind requirements for all structures constructed in the following areas:~~

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

~~**1705.4.1 Wind requirements in the statement of special inspections.** When Section 1705.4 specifies that wind requirements be included, the statement of special inspections shall identify the main wind-force-resisting systems and wind-resisting components subject to special inspections as specified in Section 1705.4.2.~~

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

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

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

3. Revise as follows:

1707.1 Special inspections for seismic resistance. Special inspections itemized in Sections 1707.2 through 1707.9, unless exempted by the exceptions of Section 1704.1~~, 1705.3 or 1705.3.1~~, are required for the following:

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C, D, E or F, as determined in Section 1613.
2. Designated seismic systems in structures assigned to Seismic Design Category D, E or F.
3. Architectural, mechanical and electrical components in structures assigned to Seismic Design Category C, D, E or F that are required in Sections 1707.6 and 1707.7.

1708.1 Testing and qualification for seismic resistance. The testing and qualification specified in Sections 1708.2 through 1708.5, unless exempted from special inspection by the exceptions of Section 1704.1, ~~1705.3 or 1705.3.1~~ are required as follows:

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C, D, E or F, as determined in Section 1613 shall meet the requirements of Sections 1708.2 and 1708.3, as applicable.
2. Designated seismic systems in structures assigned to Seismic Design Category C, D, E or F subject to the special certification requirements of ASCE 7 Section 13.2.2 are required to be tested in accordance with Section 1708.4.

3. Architectural, mechanical and electrical components in structures assigned to Seismic Design Category C, D, E or F with an I_p = 1.0 are required to be tested in accordance with Section 1708.4 where the general design requirements of ASCE 7 Section 13.2.1, Item 2 for manufacturer's certification are satisfied by testing.
4. The seismic isolation system in seismically isolated structures shall meet the testing requirements of Section 1708.5.

4. Delete without substitution:

~~**1613.4 Special inspections**. Where required by Sections 1705.3 through 1705.3.5, the statement of special inspections shall include the special inspections required by Section 1705.3.6.~~

Reason: The purpose for this proposal is to remove superfluous and conflicting text from Chapter 17. Section 1705.1 requires the registered design professional in responsible charge to prepare a statement of special inspections where special inspections or tests are required elsewhere in Chapter 17. Section 1705.2 specifies the required content of the statement of special inspections, including but not limited to: (1) the type and extent of each special inspection, (2) the type and extent of each test, (3) the additional requirements for special inspections and tests for seismic and wind resistance, and (4) for each type of special inspection, whether it is to be continuous or periodic.

Section 1704 specifies the required special inspections and tests for all structures. Sections 1707 and 1708 specify additional required special inspections and tests for structures on sites of high seismic hazard (assigned to Seismic Design Category C, D, E or F. Section 1706 specifies additional required special inspections and tests for structures on sites of high wind hazard (basic wind speed, V_{3S}, of 110 mph or greater for Exposure Categories C and D and 120 mph for Exposure Category B).

With the requirements described above, the text in Sections 1705.3 through 1705.4.2 serves no purpose other than to repeat much of what is already specified elsewhere in Chapter 17 but these sections also contain numerous conflicts. Section 1705.2 requires the type and extent of each special inspection and test to be specified in the statement of special inspections. Sections 1705.3 through 1705.4.2, however, specify numerous thresholds that must be met before certain special inspections or tests are specified and exempt several that Section 1705.2 requires to be specified in the statement. Sections 1705.3 through 1705.4.2 also create numerous potential conflicts with the required inspections in Sections 1704. 1706, 1707 and 1708 by not requiring an otherwise required special inspection or test to be included in the statement of special inspections. There are several instances where a special inspection or test is required by Section 1704. 1706, 1707 or 1708 to be performed and is required by Section 1705.2 to be included in the statement of special inspections but is not required to be included in the statement of special inspections by Section 1705.3 or 1705.4. Examples are noted below.

The charging text of Sections 1705.3 (seismic) and 1705.4 (wind) each requires the statement of special inspections to include seismic requirements and wind requirements, respectively. This can be nothing more than a requirement for what is to be specified in the statement of special inspections. It can not be a requirement for the actual special inspections or tests, which are specified in Sections 1704, 1706, 1707 and 1708.

Section 1613.4 requires the statement of special inspections to include the special inspections required by Section 1705.3.6 and Item #2 of this section requires the additional special inspections and tests of Sections 1707 and 1708 to be specified in the statement but this is superfluous in that the requirement is already specified in Item #4 of Section 1705.2.

Note that this proposal corrects inadvertent errors by adding Section 1706 to Section 1705.1 and to Item 4 of Section 1705.2.

Although Section 1705.3 is being deleted by this proposal, modifications to this section are the subject of a separate proposal. Should both proposals be approved by the ICC membership, it is the intent of the proponent to modify this section in accordance with the separate proposal, not to delete it in accordance with this proposal.

Although Sections 1705.3.6 and 1705.4.1 are being deleted by this proposal, modifications to these sections are the subject of a separate proposal. Should both proposals be approved by the ICC membership, it is the intent of the proponent to modify these sections in accordance with the separate proposal, not to delete them in accordance with this proposal.

The following are examples of conflicts between thresholds in Section 1705.3 or 1705.4 and Section 1704. 1706, 1707 or 1708:

1. Section 1705.1, Exception 1, exempts the seismic requirements of structures of light-frame construction with a maximum S_{DS} of 0.5 and a maximum height above grade plane of 35 feet from being included in the statement but Sections 1707.1(1) and 1708.1(2) exempt the seismic-force-resisting systems of all structures assigned to SDC A and B from special inspection and testing, respectively.
2. Section 1705.1, Exception 2, exempts the seismic requirements of reinforced concrete and masonry structural systems of structures with a maximum S_{DS} of 0.5 and a maximum height above grade plane of 25 feet from being included in the statement but Sections 1707.1(1) and 1708.1(2) exempt the seismic-force-resisting systems of all structures assigned to SDC A and B from special inspection and testing, respectively.
3. Section 1705.3.3(2) specifies seismic requirements for piping systems and mechanical units containing flammable, combustible or highly toxic materials be included in the statement but Section 1707.7(3) requires special inspection for piping systems and "**their associated**" (emphasis mine) mechanical units.
4. Section 1705.3.4(2) specifies seismic requirements for exterior wall panels and their anchorage be included in the statement but Section 1707.6 requires special inspection for exterior cladding, nonbearing walls and veneer greater than 30 feet above grade or a walking surface except for exterior cladding and veneer weighing 5 psf or less.
5. Section 1705.3 does not specify seismic requirements for interior wall panels be included in the statement but Section 1707.6 requires special inspection for interior nonbearing walls and veneer greater than 30 feet above grade or a walking surface except for interior veneer weighing 5 psf or less and interior nonbearing walls weighing 15 psf or less.
6. Section 1705.3.4(3) specifies seismic requirements for suspended ceiling systems and their anchorage be included in the statement but Sections 1704, 1707 1708 do not require special inspection or testing for them.
7. Section 1705.3.4(5) specifies seismic requirements for steel storage racks and their anchorage be included in the statement where the importance factor, *I*, is required by ASCE 7 Section 15.5.3 to be 1.5 (e.g., open to the public) but Section 1707.5 requires special inspection for storage rack 8 feet or greater in height.
8. Section 1705.3 does not specify seismic requirements for seismic isolation systems be included in the statement but Sections 1707.9 and 1708.5 require special inspection and testing, respectively.

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

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

ICCFILENAME: Brazil-S33-1613.4

S130–09/10

1705.3.1

Proponent: Bonnie Manley, representing American Iron and Steel Institute

Revise as follows:

1705.3.1 Seismic-force-resisting systems. The seismic force- resisting systems in structures assigned to *Seismic Design Category* C, D, E or F, in accordance with Section 1613.

> **Exception:** Requirements for the seismic-force-resisting system are permitted to be excluded from the statement of special inspections for steel systems in structures assigned to *Seismic Design Category* C that are not specifically detailed for seismic resistance, with a response modification coefficient, *R*, of 3~~ or less~~, excluding cantilever column systems.

Reason: This slight modification corrects the statement to reflect only those buildings and structures designed as "Steel systems not detailed specifically detailed for seismic resistance, excluding cantilever column systems," per ASCE 7, Table 12.2-1, Line H. Similar modifications are recommended in Sections 1707.2 and 1708.3.

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

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

ICCFILENAME: Manley-S14-1705.3.1

S131–09/10

1705.3.4

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

Revise as follows:

1705.3.4 Seismic Design Category D. The following additional systems and components in structures assigned to Seismic Design Category D:

1. Systems required for Seismic Design Category C.
2. Exterior wall panels and their anchorage.
~~3.~~ ~~Suspended ceiling systems and their anchorage~~.
<u>3</u>~~4~~. Access floors and their anchorage.
<u>4</u>~~5~~. Steel storage racks and their anchorage, where the importance factor is equal to 1.5 in accordance with Section 15.5.3 of ASCE 7.

Reason: Based upon IBC Section 1613.1 that specifically states that the building code does not adopt ASCE 7-05 Appendix Chapter 11A, where the special inspection requirements for suspended ceilings are located, it appears reasonable to omit suspended ceiling systems from the list of items that should have special inspection. The very simple and straightforward seismic requirements in ASCE 7-05 Section 13.5.6.2.2 and the ASTM and CISCA standards for installation of suspended ceilings, can easily be inspected by the traditional local government building inspectors. Finally, ICC staff written responses to inquires on this matter have indicated that the building code does not require special inspection of suspended ceiling systems because it specifically does not adopt ASCE 7 Appendix Chapter 11A.

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

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

ICCFILENAME: MAIEL-S4-1705.3.4

S132–09/10
1705.3.6, 1705.4.1

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

Revise as follows:

~~1705.3.6~~ 1705.3 Seismic requirements in the statement of special inspections. ~~When Sections 1705.3 through 1705.3.5 specify that~~ Where Section 1707 or 1708 specifies special inspection, testing or qualification for seismic ~~requirements be included~~ resistance, the statement of special inspections shall identify the ~~following:~~

1. ~~The~~ designated seismic systems and seismic-force-resisting systems that are subject to special inspection~~s in accordance with Sections 1705.3 through 1705.3.5~~.
2. ~~The additional special inspections and testing to be provided as required by Sections 1707 and 1708 and other applicable sections of this code, including the applicable standards referenced by this code.~~

~~1705.4.1~~ 1705.4 Wind requirements in the statement of special inspections. ~~When Section 1705.4~~ Where Section 1706 specifies ~~that~~ special inspection for wind requirements ~~be included~~, the statement of special inspections shall identify the main wind-force-resisting systems and wind-resisting components subject to special inspection~~s as specified in Section 1705.4.2~~.

Reason: This proposal is being submitted in conjunction with a proposal to delete superfluous and conflicting text from Chapter 17. That proposal deletes Sections 1705.3 through 1705.4.2, which specify certain seismic and wind requirements to be included in the statement of special inspections. However, these sections overlap or conflict with Section 1705.2, which also specifies what is to be included in the statement of special inspections, and with Sections 1704, 1706, 1707 and 1708, which specify required special inspections and tests.

In spite of this, it appears that retaining the substance of Section 1705.3.6, which requires the seismic-force-resisting systems and designated seismic systems, and Section 1705.4.1, which requires the main wind-force-resisting systems and wind-resisting components, to be identified in the statement of special inspections will contribute to a better understanding of the portions of the structural system where special inspection, testing or qualification is required.

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

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

ICCFILENAME: Brazil-S18-1705.3

S133–09/10
1702.1, 1707.1, 1707.8, 1707.6, 1707.5, 1707.7, 1707.9

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.

DESIGNATED SEISMIC SYSTEM. Those ~~architectural, electrical and mechanical systems and their~~ nonstructural components that require design in accordance with Chapter 13 of ASCE 7 and for which the component importance factor, I_p, is greater than 1 in accordance with Section 13.1.3 of ASCE 7.

1707.1 Special inspections for seismic resistance. Special inspections itemized in Sections 1707.2 through 1707.9, unless exempted by the exceptions of Section 1704.1, 1705.3 or 1705.3.1, are required for the following:

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C, D, E or F, as determined in Section 1613, in accordance with Sections 1707.2 through 1707.4, as applicable.
2. Designated seismic systems in structures assigned to Seismic Design Category C, D, E or F in accordance with Section 1707.5.
3. Architectural, mechanical and electrical components ~~in structures assigned to Seismic Design Category C, D, E or F that are required~~ in accordance with Sections 1707.6 and 1707.7.
4. Storage racks in structures assigned to Seismic Design Category D, E or F in accordance with Section 1707.8.
5. Seismic isolation systems in accordance with Section 1707.9.

~~1707.8~~ 1707.5 Designated seismic systems ~~verifications~~. The special inspector shall examine designated seismic systems requiring seismic qualification in accordance with Section 1708.4 and verify that the label, anchorage or mounting conforms to the certificate of compliance.

~~1707.5~~ 1707.6.1 ~~Storage racks and~~ Access floors. Periodic special inspection is required ~~during~~ for the anchorage of access floors ~~and storage racks 8 feet (2438 mm) or greater in height~~ in structures assigned to Seismic Design Category D, E or F.

1707.7 Mechanical and electrical components. Special inspection for mechanical and electrical ~~equipment~~ components shall be as follows:

1. Periodic special inspection is required during the anchorage of electrical equipment for emergency ~~or~~ and standby power systems in structures assigned to Seismic Design Category C, D, E or F;
2. Periodic special inspection is required during the installation of anchorage of other electrical equipment in structures assigned to Seismic Design Category E or F;
3. Periodic special inspection is required during installation of piping systems intended to carry flammable, combustible or highly toxic contents and their associated mechanical units in structures assigned to Seismic Design Category C, D, E or F;
4. Periodic special inspection is required during the installation of HVAC ductwork that will contain hazardous materials in structures assigned to Seismic Design Category C, D, E or F; and
5. Periodic special inspection is required during the installation of vibration isolation systems in structures assigned to Seismic Design Category C, D, E or F where the construction documents require a nominal clearance of 1/4 inch (6.4 mm) or less between the equipment support frame and restraint.

1707.8 Storage racks. Periodic special inspection is required for the anchorage of storage racks 8 feet (2438 mm) or greater in height in structures assigned to Seismic Design Category D, E or F.

1707.9 Seismic isolation system. Periodic special inspection ~~is required~~ shall be provided for seismic isolation systems during the fabrication and installation of isolator units and energy dissipation devices ~~that are part of the seismic isolation system~~.

Reason: The purpose for this proposal is to correlate IBC Section 1707 with ASCE 7-10. In Section 1702.1, the definition of “designated seismic system” is changed for consistency with the revised definition in Section 11.2 of ASCE 7-10.

In Item #2 of Section 1707.1 on special inspection of designated seismic systems, Seismic Design Category C is added for consistency with Item #2 of Section 1708.1 on testing of designated seismic systems, which specifies Seismic Design Category C, and references to the requirements for certification in Section 13.2.2 of ASCE 7, which also specifies Seismic Design Category C.

In Section 1707.1, Item #3 is revised to eliminate superfluous text and avoid potential conflicts. The reference to Seismic Design Category D, E or F is deleted in favor of the references to seismic design categories in Sections 1707.6 and 1707.7 and because these sections do not “require” the components as Item #3 currently states.

Section 1707.5 on storage racks and access floors is changed to Section 1707.6.1 on access floors and Section 1707.8 on storage racks because ASCE 7 identifies access floors as architectural components (Section 13.5.7) and steel storage racks as nonbuilding structures (Section 15.5.3). In Section 1707.1, Item #4 is added for storage racks, which are currently not accounted for in the items of Section 1707.1 except indirectly by Item #1 of Section 1707.1.

In Section 1707.1, Item #5 on seismic isolation systems is added because the other items do not clearly account for them. Section 1707.9 is revised because the current text is nonmandatory. It is also not clear with respect to the seismic design category and the changes will make it clear that periodic special inspection is required for seismic isolation systems in structures regardless of which seismic design category the structure is assigned.

In Section 1707.7, mechanical and electrical “equipment” is changed to “components” for consistency with Section 13.6 of ASCE 7 on mechanical and electrical components and because the items in the section are not limited to equipment but also include piping systems, HVAC ductwork and vibration isolation systems.

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

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

ICCFILENAME: Brazil-S27-1702.1

S134–09/10

1705.3, 1707.1

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

Revise as follows:

1707.1 Special inspections for seismic resistance. *Special inspections* itemized in Sections 1707.2 through 1707.9, unless exempted by the exceptions of Section 1704.1, 1705.3, or 1705.3.1, are required for the following:

1. The seismic-force-resisting systems in structures assigned to *Seismic Design Category* C, D, E or F, as determined in Section 1613.
2. Designated seismic systems in structures assigned to *Seismic Design Category* D, E or F.
3. Architectural, mechanical and electrical components in structures assigned to *Seismic Design Category* C, D, E or F that are required in Sections 1707.6 and 1707.7.

~~**1705.3 Seismic resistance.** The statement of special inspections shall include seismic requirements for cases covered in Sections 1705.3.1 through 1705.3.5.~~

Exception: ~~Seismic requirements are permitted to be excluded from the statement of special inspections~~ Special inspections itemized in Sections 1707.2 through 1707.9 are not required for structures designed and constructed in accordance with one of the following:

1. The structure consists of light-frame construction; the design spectral response acceleration at short periods, S_{DS}, as determined in Section 1613.5.4, does not exceed 0.5 ~~g~~; and the building height of the structure does not exceed 35 feet (10 668 mm) ~~above grade plane; or~~.
2. The seismic-force-resisting system of the structure ~~is constructed using a~~ consists of reinforced masonry ~~structural system~~ or reinforced concrete ~~structural system~~; the design spectral response acceleration at short periods, S_{DS}, as determined in Section 1613.5.4, does not exceed 0.5 ~~g~~; and the building height of the structure does not exceed 25 feet (7620 mm) ~~above grade plane; or~~.
3. The structure is a detached one- or two-family dwelling~~s~~ not exceeding two stories above grade plane~~,~~ ~~provided the structure~~ and does not have any of the following ~~plan~~ horizontal or vertical irregularities in accordance with Section ~~12.3.2~~ 12.3 of ASCE 7:
   - 3.1. Torsional or extreme torsional irregularity.
   - 3.2. Nonparallel systems irregularity.
   - 3.3. ~~Stiffness irregularity-extreme~~ Stiffness-soft story ~~and~~ or stiffness-extreme soft story irregularity.
   - 3.4. Discontinuity in ~~capacity~~ lateral strength-weak story irregularity.

Reason: This proposal is being submitted in conjunction with a proposal to delete superfluous and conflicting text from Chapter 17. That proposal deletes Sections 1705.3 through 1705.4.2, which specify certain seismic and wind requirements to be included in the statement of special inspections. However, these sections overlap or conflict with Section 1705.2, which also specifies what is to be included in the statement of special inspections, and with Sections 1704, 1706, 1707 and 1708, which specify required special inspections and tests.

In spite of this, it appears the exceptions to Section 1705.3, which exempt the seismic requirements of certain structures from being included in the statement of special inspections, are intended to be exemptions from the special inspections for seismic resistance in Section 1707. The purpose for this proposal is to relocate these exceptions to Section 1707 so that they will serve their apparent purpose.

The proposal includes several editorial revisions. In Exceptions #1 and #2, “height” is changed to “building height” for consistency with the definition of “building height” in Section 502.1 the charging text of which makes the defined term applicable throughout the IBC, including the structural chapters. With this change, the reference to “above grade plane” becomes superfluous because it is included in the definition of “building height” and is deleted.

In Exception #2, “structural system,” which has no technical meaning, is replaced with “seismic-force-resisting system,” which is defined in Section 1613.2. In Exception #3, several changes are made for consistency with Tables 12.3-1 and 12.3.2 of ASCE 7-10 on horizontal and vertical structural irregularities, respectively.

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

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

ICCFILENAME: Brazil-S35-1705.3

S135–09/10
1707.2

Proponent: Bonnie Manley, representing American Iron and Steel Institute

Revise as follows:

1707.2 Structural steel. *Special inspection* for structural steel shall be in accordance with the quality assurance ~~plan~~ requirements of AISC 341.

Exceptions:

1. *Special inspections* of structural steel in structures assigned to *Seismic Design Category* C that are not specifically detailed for seismic resistance, with a response modification coefficient, *R*, of 3~~ or less~~, excluding cantilever column systems.
2. For ordinary moment frames, ultrasonic and magnetic particle testing of complete joint penetration groove welds are only required for demand critical welds.

Reason: The first editorial modification tightens up the reference to AISC 341 and the second modification in the exception corrects the requirement to reflect only those buildings and structures designed as "Steel systems not detailed specifically detailed for seismic resistance, excluding cantilever column systems," per ASCE 7, Table 12.2-1, Line H. Similar modifications are recommended in Sections 1705.3.1 and 1708.3.

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

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

ICCFILENAME: Manley-S6-1707.2

S136–09/10
1707.2

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

Revise as follows:

1707.2 Structural steel. *Special inspection* for structural steel shall be in accordance with the quality assurance plan requirements of AISC 341.

Exception~~s~~:

~~1.~~ *Special inspections* of structural steel in structures assigned to *Seismic Design Category* C that are not specifically detailed for seismic resistance, with a response modification coefficient, *R*, of 3 or less, excluding cantilever column systems.

~~2. For ordinary moment frames, ultrasonic and magnetic particle testing of complete joint penetration groove welds are only required for demand critical welds.~~

Reason: This exception was new to the 2009 IBC and was added in the public comment on S128-07/08. Intended as a temporary solution, its purpose was to make 2009 IBC consistent with what had been proposed for the 2010 edition of AISC 341. A separate proposal recommends the adoption of the 2010 edition of AISC 341. Since AISC 341-10 now addresses this issue, the exception is no longer needed. A similar modification is recommended in Section 1708.3.

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.

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

ICCFILENAME: Manley-S7-1707.2

S137–09/10

1708, 1708.1, 1708.4, 1708.5

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

Revise as follows:

SECTION 1708
~~STRUCTURAL~~ TESTING AND QUALIFICATION FOR SEISMIC RESISTANCE

1708.1 Testing and qualification for seismic resistance. The testing and qualification specified in Sections 1708.2 through 1708.5, unless exempted from special inspection by the exceptions of Section 1704.1, 1705.3 or 1705.3.1, are required as follows:

1. The seismic-force-resisting systems in structures assigned to Seismic Design Category C, D, E or F, as determined in Section 1613 shall meet the requirements of Sections 1708.2 and 1708.3, as applicable.
2. Designated seismic systems in structures assigned to Seismic Design Category C, D, E or F and subject to the ~~special~~ certification requirements of ASCE 7 Section 13.2.2 ~~are required to be tested in accordance~~ shall comply with Section 1708.4.
3. Architectural, mechanical and electrical components in structures assigned to Seismic Design Category C, D, E or F ~~with an I_p = 1.0 are required to be tested in accordance with Section 1708.4~~ and where the ~~general design~~ requirements of ASCE 7 Section 13.2.1~~, Item 2 for~~ are met by submittal of manufacturer's certification ~~are satisfied by testing~~ in accordance with Item 2 therein shall comply with Section 1708.4.
4. The seismic isolation system in seismically isolated structures shall meet the testing requirements of Section 1708.5.

1708.4 Seismic certification of nonstructural components. The registered design professional shall ~~state~~ specify on the construction documents the ~~applicable seismic certification~~ requirements for certification by analysis, testing or experience data for nonstructural components and designated seismic systems in accordance with Section 13.2 of ASCE 7, where such certification is required by Section 1708.1 ~~on the construction documents~~.

~~1. The manufacturer of each designated seismic system component subject to the provisions of ASCE 7 Section 13.2.2 shall test or analyze the component and its mounting system or anchorage and submit a certificate of compliance for review and acceptance by the registered design professional responsible for the design of the designated seismic system and for approval by the building official. Certification shall be based on an actual test on a shake table, by three-dimensional shock tests, by an analytical method using dynamic characteristics and forces, by the use of experience data (i.e., historical data demonstrating acceptable seismic performance) or by more rigorous analysis providing for equivalent safety.~~

~~2. Manufacturers certification of compliance for general design requirements of ASCE 7 Section 13.2.1 shall be based on analysis, testing or experience data.~~

1708.5 ~~Seismically isolated structures~~ Seismic isolation systems. ~~For required system tests, see~~ Seismic isolation systems shall be tested in accordance with Section 17.8 of ASCE 7.

Reason The purpose for this proposal is to correlate IBC Section 1708 with ASCE 7-10. The title of Section 1708 is changed for consistency with the title and charging text of Section 1708.1. Section 1708.5 is revised for consistency with Section 1707.9 on seismic isolation systems and because the current text is nonmandatory.

Section 1708.4 currently contains technical requirements for qualification (1) of certain architectural, mechanical and electrical components for which manufacturer's certification is utilized to comply with Section 13.2.1 of ASCE 7, and (2) of certain designated seismic systems for which manufacturer's certification is required by Section 13.2.2 of ASCE 7. Much of the IBC text, however, is duplicated in ASCE 7 and may also conflict with the corresponding text in ASCE 7. The technical requirements in ASCE 7 typically consist of analysis, shake table testing or experience data. It serves little purpose to duplicate these requirements in the IBC. The requirements for certification, however, should remain in the IBC and this proposal revises the text accordingly.

Items 2 and 3 of IBC Section 1708.1 are revised for consistency with Sections 13.2.2 and 13.2.1, respectively, of ASCE 7-10. Note that these sections have been revised in ASCE 7-10 by Proposals GPSC-5R2, SSC TC8-CH13-12-R3 and SSC TC9-CH13-01-R4.
Section 13.2.1 of ASCE 7 requires architectural, mechanical and electrical components, supports and attachments to comply with the provisions of the sections listed in Table 13.2-1 and to meet the requirements in Item 1 or Item 2 of Section 13.2.1. Item 2 requires the submittal of the manufacturer's certification that the component is seismically qualified.

Section 13.2.2 of ASCE 7 requires for certain designated seismic systems the submittal of the manufacturer's certification that the designated seismic system meets the specified requirements therein. According to its definition in IBC Section 1702.1 and Section 11.2 of ASCE 7-10, "designated seismic system" applies to architectural, mechanical and electrical components for which the component importance factor, I_p, is greater than 1.0. Designated seismic systems not required to comply with Section 13.2.2 will still be required to comply with Section 13.2.1.

The proposal was prepared in conjunction with a related proposal to correlate IBC Section 1707 with ASCE 7-10.

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

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

ICCFILENAME: Brazil-S15-1708.1

S138–09/10

1708.3

Proponent: Bonnie Manley, representing American Iron and Steel Institute

Revise as follows:

1708.3 Structural steel. Testing for structural steel shall be in accordance with the quality assurance plan requirements of AISC 341.

Exception~~s~~:

1. Testing for structural steel in structures assigned to *Seismic Design Category* C that are not specifically detailed for seismic resistance, with a response modification coefficient, *R*, of 3 or less, excluding cantilever column systems.
2. ~~For ordinary moment frames, ultrasonic and magnetic particle testing of complete joint penetration groove welds are only required for demand critical welds.~~

Reason: This exception was new to the 2009 IBC and was added in the public comment on S137-07/08. Intended as a temporary solution, its purpose was to make 2009 IBC consistent with what had been proposed for the 2010 edition of AISC 341. A separate proposal recommends the adoption of the 2010 edition of AISC 341. Since AISC 341-10 now addresses this issue, the exception is no longer needed. A similar modification is recommended in Section 1707.2.

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.

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

ICCFILENAME: Manley-S9-1708.3

S139–09/10

1710.2, 1710.3

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

Revise as follows:

1710.2 Structural observations for seismic resistance. Structural observations shall be provided for those structures assigned to Seismic Design Category D, E or F, as determined in Section 1613, where one or more of the following conditions exist:

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

1710.3 Structural observations for wind requirements. Structural observations shall be provided for those structures sited where the basic wind speed exceeds 110 mph (49 m/sec), determined from Figure 1609, where one or more of the following conditions exist:

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

Reason: The IBC is a model code written with mandatory text that has the force of law when adopted by a jurisdiction. Violations are subject to penalties as prescribed by the laws of the jurisdiction and as specified in Section 114. In this regard, a requirement that the owner employ a registered design professional to perform structural observations should not be imposed based on a designation by the registered design professional responsible for the structural design. If the registered design professional responsible for the structural design believes that structural observation is warranted, they should convey their belief to their clients. The building code should not contain a requirement for what is primarily a private matter between registered design professionals and their clients.

A requirement that the owner employ a registered design professional to perform structural observations should also not be imposed where it is required by the building official. Sections 1710.2 and 1710.3 specify limits based on occupancy category or building height for which structural observation is required where the limits are exceeded. These requirements provide sufficient means for the building official to determine when structural observation is required. Additional authority on the part of the building official is not warranted. If it was warranted, the limits based on occupancy category or building height could be deleted and the determination could be solely based on the requirement of the building official without regard to occupancy category or building height.

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

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

ICCFILENAME: Brazil-S11-1710.2

S140–09/10
1715.5.1

Proponent: Thomas D. Culp, Ph.D. Birch Point Consulting LLC, representing Aluminum Extruders Council

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/CSA101/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 <u>in buildings of Group R not more than three stories above grade plane</u> 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: Chapter 24 and ASTM E1300 require that glazing be firmly supported to prevent breakage under the design load by establishing maximum framing deflection limits. However, certain products are currently and inappropriately exempted from this requirement if they are labeled to the AAMA/WDMA/CSA 101/I.S.2/A440 standard. This proposal would remove that exemption to restore an appropriate safety margin of less than 8 in 1000 probability of glass breakage, consistent with ASTM E1300.

This proposal only applies to the IBC, and does not affect lighter products used when building to the IRC. However, in the last code cycle, the committee correctly pointed out that the IBC is also used for lowrise residential buildings, including both detached homes, townhomes, and apartments. Therefore, this proposal has been modified from last cycle to address the committee's concern by reinstating the exemption for low-rise residential buildings, but maintaining the structural deflection limit requirements for products in all other applications, as the top priority should be restoring a safety margin consistent with what is already in Chapter 24 and ASTM E1300.

Finally, the committee reason for disapproving the previous proposal stated that there is too much uncertainty to remove the exemption at this time. As the exemption increases the probability of glass breakage, we believe safety concerns would dictate that any exemption SHOULD be removed until any uncertainty is resolved.

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

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

ICCFILENAME: CULP-S1-1714.5.1

S141–09/10
1715.5.1

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/CSA101/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 <u>installed in buildings of Group R not more than three stories above grade plane that are</u> 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 ¾ inch, whichever is less) when subjected to the design load. Chapter 24 of the IBC relies on glass design curves that are contained in ASTM E 1300. This ASTM standard recognizes the importance of limiting edge deflection of the glass and also recommends a limitation of 1/175 of the glass edge length. Prior to the IBC, the legacy codes required deflection limitations of 1/175 of the span for glass holding members. It was not until the IBC was published that this exemption was allowed.

AAMA/WDMA/CSA 101/I.S.2/A440 does require testing in accordance with ASTM E330 and measurement of deflection. However, AAMA/WDMA/CSA 101/I.S.2/A440 only places a limit on the frame and sash deflection for heavy commercial (HC) and architectural products (AW), and has no requirement on deflection for residential (R), light commercial (LC), and commercial (C) products. Excessive deflection of the frame or sash can have an adverse effect on stress in the glass and could result in glass breakage at or below design loads creating a safety concern. The single ASTM E330 load test required in AAMA/WDMA/CSA 101/I.S.2/A440 is not statistically significant in ensuring that the stress does not increase the probability of breakage beyond the industry standard of eight lites per thousand when the deflection limitation of 1/175 is exceeded. 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.

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

ICCFILENAME: KOFFELL-S1-1714.5.1

S142-09/10
1715.5.2

Proponent: Joseph R. Hetzel, Thomas Associates Inc. representing the Door & Access Systems Manufacturers Association

Revise as follows:

1715.5.2 Exterior windows and door assemblies not provide for in Section 1715.5.1. Exterior window and door assemblies shall be tested in accordance with ASTM E 330. Structural performance of garage doors <u>and rolling doors</u> 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 10 seconds at a load equal to 1.5 times the design pressure.

Reason: The purpose of the proposed code change is to expand the sentence on “garage doors” to include reference to “rolling doors”. The scope of ANSI/DASMA 108-2005, referenced in the 2009 IBC, encompasses both garage doors and rolling doors.

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

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

ICCFILENAME: Hetzel-G1-1714.5.2

S143–09/10
1715.5.2, Chapter 35

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. <u>Structural performance of exterior side-hinged door assemblies shall be determined in accordance with either ASTM E330 or ANSI A250.13.</u> Structural performance of garage doors shall be determined in accordance with either ASTM E 330 or ANSI/DASMA 108, and shall meet the acceptance criteria of ANSI/DASMA 108. Exterior window and door assemblies containing glass shall comply with Section 2403. The design pressure for testing shall be calculated in accordance with Chapter 16. Each assembly shall be tested for <u>a minimum of</u> 10 seconds at a load equal to 1.5 times the design pressure.

2. Add new standard to Chapter 35 as follows:

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

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 <u>30</u> seconds at a load equal to 1.5 times the design pressure. Currently 1714.5.2 requires testing for <u>10</u> 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.

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

ICCFILENAME: WOESTMAN-S2-1714.5.2

S144–09/10
1715.6 (New), 202; IRC R308.6.1

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

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

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. <u>Unit skylights include, but are not limited to, tubular daylighting devices (TDDs).</u>

2. Add new text as follows:

<u>**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.</u>

(Renumber subsequent sections)

PART II – IRC BUILDING/ENERGY

Revise as follows:

R308.6.1 Definitions.

~~**UNIT SKYLIGHT**~~ <u>**SKYLIGHT, UNIT**</u>**.** 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. <u>Unit skylights include, but are not limited to, tubular daylighting devices (TDDs).</u>

Reason:
PART I- 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.
PART II- 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.

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: RUTH-G1-202

S145–09/10
1716.1, 2303.1, 2303.5, 2304.9.3

Proponent: Randall Shackelford, PE, representing Simpson Strong-Tie Co.

1. Revise as follows:

2303.1 General. Structural sawn lumber; end-jointed lumber; prefabricated wood I-joists; structural glued-laminated timber; wood structural panels, fiberboard sheathing (when used structurally); hardboard siding (when used structurally); particleboard; preservative-treated wood; structural log members; structural composite lumber; round timber poles and piles; fire-retardant-treated wood; hardwood plywood; wood trusses; ~~joist hangers;~~ nails; and staples shall conform to the applicable provisions of this section.

2304.9.3 Joist hangers and framing anchors. Connections depending on joist hangers or framing anchors, ties and other mechanical fastenings not otherwise covered are permitted where approved. ~~The vertical load-bearing capacity, torsional moment capacity and deflection characteristics of joist hangers shall be determined in accordance with Section 1716.1.~~

2. Delete without substitution:

~~**1716.1 Test standards for joist hangers and connectors.**~~

~~**1716.1.1 Test standards for joist hangers.** The vertical load-bearing capacity, torsional moment capacity and deflection characteristics of joist hangers shall be determined in accordance with ASTM D1761 using lumber having a specific gravity of 0.49 or greater, but not greater than 0.55, as determined in accordance with AF&PA NDS for the joist and headers.~~

~~**Exception:** The joist length shall not be required to exceed 24 inches (610 mm).~~

~~**1716.1.2 Vertical load capacity for joist hangers.** The vertical load capacity for the joist hanger shall be determined by testing a minimum of three joist hanger assemblies as specified in ASTM D 1761. If the ultimate vertical load for any one of the tests varies more than 20 percent from the average ultimate vertical load, at least three additional tests shall be conducted. The allowable vertical load of the joist hanger shall be the lowest value determined from the following:~~

1. ~~The lowest ultimate vertical load for a single hanger from any test divided by three (where three tests are conducted and each ultimate vertical load does not vary more than 20 percent from the average ultimate vertical load).~~
2. ~~The average ultimate vertical load for a single hanger from all tests divided by three (where six or more tests are conducted).~~
3. ~~The average from all tests of the vertical loads that produce a vertical movement of the joist with respect to the header of 0.125 inch (3.2 mm).~~
4. ~~The sum of the allowable design loads for nails or other fasteners utilized to secure the joist hanger to the wood members and allowable bearing loads that contribute to the capacity of the hanger.~~
5. ~~The allowable design load for the wood members forming the connection.~~

~~**1716.1.3 Torsional moment capacity for joist hangers.** The torsional moment capacity for the joist hanger shall be determined by testing at least three joist hanger assemblies as specified in ASTM D 1761. The allowable torsional moment of the joist hanger shall be the average torsional moment at which the lateral movement of the top or bottom of the joist with respect to the original position of the joist is 0.125 inch (3.2 mm).~~

~~**1716.1.4 Design value modifications for joist hangers.** Allowable design values for joist hangers that are determined by Item 4 or 5 in Section 1716.1.2 shall be permitted to be modified by the appropriate duration of loading factors as specified in AF&PA NDS but shall not exceed the direct loads as determined by Item 1, 2 or 3 in Section 1716.1.2. Allowable design values determined by Item 1, 2 or 3 in Section 1716.1.2 shall not be modified by duration of loading factors.~~

(Renumber subsequent sections)

~~**2303.5 Test standard for joist hangers and connectors.** For the required test standards for joist hangers and connectors, see Section 1716.1.~~

(Renumber subsequent sections)

Reason: The purpose of this code change is to delete current material that references outdated test standards.

Section 1716.1 was originally added into the code before there were ICC-ES acceptance criteria for joist hangers or other types of connectors. It was needed to give connector manufacturers guidance on how to load rate their products.

Now, however, there are several ICC-ES acceptance criteria for joist hangers and other types of connectors, so this language is no longer needed. For example, AC13 covers "joist hangers, framing anchors and similar devices".

In addition, this section references ASTM D 1761 as the test standard for joist hangers. This was the correct standard in the past. But in the 2009 IBC, ASTM D 1761 has been updated to the 2006 edition, which does not cover testing of joist hangers. Since this test standard no longer describes how to test a joist hanger, this section is not correct as written.

Joist hangers and framing anchors will still have to be approved by the Building Official per Section 2304.9.3.

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

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

ICCFILENAME: SHACKELFORD-S1-1716.1

S146–09/10
1801.1; IEBC 1202.2

Proponent: Patrick Vandergriff, Vandergriff Code Consulting Services representing Modular Building Institute

THIS IS A 2 PART CODE CHANGE. BOTH PARTS WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE AS 2 SEPARATE CODE CHANGES. SEE THE TENTATIVE HEARING ORDER FOR THE STRUCTURAL COMMITTEE.

PART I- IBC STRUCTURAL

Revise as follows:

1801.1 Scope. The provisions of this chapter shall apply to building and foundation systems.

> <u>**Exception:** Foundations and foundation connections for temporary structures shall comply with manufacturers design details or an engineered design.</u>

PART II- IEBC

Revise as follows:

1202.2 Foundation. The foundation system of relocated buildings shall comply with the *International Building Code* or the *International Residential Code* as applicable.

> <u>**Exception**: Foundations for temporary structures shall be permitted to be of any material allowed by this code in accordance with manufacturers design details or an engineered design.</u>

1202.2.1 Connection to the foundation. The connection of the relocated building to the foundation shall comply with the *International Building Code* or the *International Residential Code* as applicable.

> <u>**Exceptions:**</u>
>
> 1. <u>Helical pier tie downs rated for sufficient resistance to uplift shall be permitted to be used on temporary structures placed on temporary foundations as permitted in the exception to Section 1202.2.</u>
> 2. <u>Other methods as provided by engineered design.</u>

Reason:
PART I- The code addresses temporary structures, but fails to determine the foundations for temporary structures.
PART II- The change in the language provides for an exception on methods of foundation construction for temporary buildings. The temporary foundation would be allowed in accordance with the provisions, and provide sufficient foundation support for the structure while maintaining the need to insure connections that resist uplift and other forces required by the code.

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

PART I- IBC STRUCTURAL

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

PART II- IEBC

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

ICCFILENAME: VANDERGRIFF-S1-1801.1-EB2-1202.2

S147–09/10
1803.2

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

Revise as follows:

1803.2 Investigation required. Geotechnical investigations shall be conducted in accordance with Sections 1803.3 through 1803.5.

Exceptions:

1. The building official shall be permitted to waive the requirement for a geotechnical investigation where satisfactory data from adjacent areas is available that demonstrates an investigation is not necessary for any of the conditions in Sections 1803.5.1 through 1803.5.6 and Sections 1803.5.10 and 1803.5.11.
2. Unless there are known potential geologic or seismic hazards or as required by any of the conditions in Section 1803.5.2, 1803.5.3, 1803.5.8, 1803.5.9 or 1803.5.10, the building official shall be permitted to waive the requirement for a geotechnical investigation for additions to light-frame R-3 or U occupancies, that are no more than two stories in height and have no basement, when they use the same foundation system as the structure to which they are attached,

Reason: It is an unreasonable hardship and cost to require a geotechnical report for an addition where the existing foundation system has proved to be adequate. The building official can still require a report if he/she deems that one is necessary or if known hazards exist.

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

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

ICCFILENAME: MAIEL-S3-1803.2

S148–09/10
1803.5.11, 1803.5.12

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

Revise as follows:

1803.5.11 Seismic Design Categories C through F. For structures assigned to Seismic Design Category C, D, E, or F in accordance with Section 1613, a geotechnical investigation shall be conducted, and shall include an evaluation of all of the following potential geologic and seismic hazards:

1. Slope instability.
2. Liquefaction.
3. Total and differential settlement.
4. Surface displacement due to faulting or seismically induced lateral spreading or lateral flow.

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 all of the following, as applicable:

1. The determination of dynamic seismic lateral earth pressures on foundation walls and retaining walls due to design earthquake ground motions.
2. The potential for liquefaction and soil strength loss evaluated for site peak ground accelerations~~s~~, earthquake magnitude~~s~~, and source characteristics consistent with the ~~design~~ maximum considered earthquake ground motions. Peak ground acceleration shall be ~~permitted to be~~ determined based on:
   2.1 A site-specific study ~~taking into account soil amplification effects, as specified in Chapter 21~~ in accordance with Section 11.4.7 of ASCE 7; or

2.2 ~~in the absence of such a study, peak ground accelerations shall be assumed equal to S_{DS} / 2.5, where S_{DS} is determined~~ The maximum considered earthquake geometric mean peak ground acceleration adjusted for site class in accordance with Section ~~1613.5.4~~ 11.8.3 of ASCE 7.

3. An assessment of potential consequences of liquefaction and soil strength loss, including, but not limited to:
   3.1 Estimation of total and differential settlement;
   3.2 Lateral soil movement;
   3.3 Lateral soil loads on foundations;
   3.4 Reduction in foundation soil-bearing capacity and lateral soil reaction;
   3.5 Soil downdrag and reduction in axial and lateral soil reaction for pile foundations;
   3.6 Increases in soil lateral pressures on retaining walls; and
   3.7 Flotation of buried structures.
4. Discussion of mitigation measures such as, but not limited to~~, ground stabilization~~:
   4.1 Selection of appropriate foundation type and depths;
   4.2 Selection of appropriate structural systems to accommodate anticipated displacements and forces;
   4.3 Ground stabilization; or
   4.4 Any combination of these measures and how they shall be considered in the design of the structure.

Reason: The purpose for this proposal is to correlate the IBC with the 2010 edition of ASCE 7. The need for correlation is due to ASCE 7 Proposal SSC TC-1-CH11-103-R2, which was approved by the Seismic Subcommittee on 5/15/09 and is being balloted by the Main Committee (Item #4 of the Sixth Main Committee Ballot on Seismic Provisions). It is expected that the Main Committee will approve the proposal.

Section 11.8.3 of ASCE 7-10 is being modified to require evaluations of liquefaction potential be made for maximum considered earthquake (MCE) ground motions rather than design earthquake (DE) ground motions to ensure that the potential occurrence and effects of liquefaction during the MCE are considered in geotechnical and structural design. This change is consistent with the risk-based targets for collapse prevention as a performance goal and other evaluations for the MCE that are specified in ASCE 7 for the performance goal of collapse-prevention during MCE loading.

Section 11.8.3 of ASCE 7-10 is also being modified to require liquefaction potential evaluations be conducted using maximum considered earthquake geometric mean peak ground acceleration (PGA) adjusted for site effects rather than the current approximation for peak ground acceleration of dividing short-period spectral acceleration by a factor of 2.5. Maps provided in ASCE 7-10 for the purpose of determining the accelerations are substantially more accurate since they are based on PGA attenuation relationships. PGA is modified for site class effects by Eq. 11.8-1 of ASCE 7-10 where the site coefficient, F_{PGA}, is obtained from Table 11.8-1 of ASCE 7-10. The values of F_{PGA} are identical to the site coefficient, F_a, in Table 11.4-1 of ASCE 7-10 but are a function of PGA rather than the mapped MCE spectral response acceleration at short periods, S_s. Because PGA is a short-period parameter (equal to zero-period spectral acceleration), it is appropriate and consistent with current practice to use the same site coefficients for PGA and S_S. It is also consistent with the original development of F_a as a function of PGA.

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

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

ICCFILENAME: Brazil-S8-1803.5.11

S149–09/10
1803.5.12

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.

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

ICCFILENAME:Fattah-S2-1803.5.1.2

S150–09/10
1803.5.12

Proponent: Jim Rossberg, SEI of ASCE, representing self

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 pressures on foundation and retaining walls 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 21.5 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~~ taken equal to the value determined in accordance with Section 11.8.3 of ASCE 7.
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: This proposed change coordinates the provision of the IBC with those of the 2010 edition of ASCE 7. This provision has been considered and approved by the Seismic Subcommittee of ASCE 7 for inclusion into the 2010 edition of ASCE 7 hence with the adoption of ASCE 7-10 by reference this provision becomes duplicative. As of the submission date of this code change, the ASCE 7 Standards Committee is completing the committee balloting portion of the 2010 edition of ASCE/SEI 7. The document is designated ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* and it is expected that it will be completed and available for purchase prior to the ICC Final Action Hearings in May of 2010 . Any person interested in obtaining a public comment copy of ASCE/SEI 7-10 may do so by contacting the proponent at jrossberg@asce.org.

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

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

ICCFILENAME: ROSSBERG-S2-1803.5.12

S151–09/10
1807.3.2.1

Proponent: Brian Johnson, PE, representing self

Revise as follows:

1807.3.2.1 Non-constrained. The following formula shall be used in determining the depth of embedment required to resist lateral loads where no lateral constraint is provided at the ground surface, such as by a rigid floor or rigid ground surface pavement, and where no lateral constraint is provided above the ground surface, such as by a structural diaphragm.

~~*d* = 0.5*A*{1 + [1 + (4.36*h*/*A*)]1/2}~~ $\underline{d = \frac{A}{2}\left(1 + \sqrt{1 + \frac{4.36h}{A}}\right)}$ **(Equation 18-1)**

where:

<u>The constant in (Equation 18-1) is dimensionless.</u>

~~*A* = 2.34*P*/S_1 *b*.~~ $\underline{A = \frac{2.34P}{S_1 b}}$, <u>feet (m)</u>

B = Diameter of round post or footing or diagonal dimension of square post or footing, feet (m).

d = Depth of embedment in earth in feet (m) ~~but not over 12 feet (3658 mm) for purpose of computing lateral pressure.~~

h = Distance in feet (m) from ground surface to point of application of ~~"*P*."~~ <u>P.</u>

P = Applied lateral force in pounds (kN).

S_1 = Allowable lateral soil-bearing pressure as set forth in Section 1806.2 based on a depth of one-third the depth of embedment in pounds per square foot (psf) (kPa)<u> but not over 12 feet (3.66 m) for purpose of computing lateral</u> <u>pressure</u>.

Reason: In the IBC 2006 formula for *A* it is unclear if *b* is a divisor or a multiplier. The formula needs parentheses or better formatting to answer this question. If it is a divisor, *A* is dimensionless and so *d* has no dimensions either. Consulting UBC 1997 revealed a properly formatted formula that should replace the one in IBC 2006, etc (*b* is a divisor). I added that *A* is in feet, thus equation 18-1 gives a result in feet. I added a clarification that 0.5 (1/2) carries no dimensions so equation 18-1 now clearly results in an answer in feet or meters based on the units entered.

The restriction on *d* doesn't make sense, the restriction should be on the soil pressure, the embedment depth can exceed 12 feet, it is the soil pressure that is to be limited. Note that 12 feet limit conflicts with IBC 1804.3.1 and I have not changed this.

Cost Impact: Decrease the cost of construction by clarifying formula.

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

ICCFILENAME: JOHNSON-S1-1807.3.2.1.DOC

S152–09/10
1807.3.2.1, 1807.3.2.2

Proponent: Brian Johnson, PE, representing self

Revise as follows:

1807.3.2.1 Nonconstrained. The following formula shall be used in determining the depth of embedment required to resist lateral loads where no lateral constraint is provided at the ground surface, such as by a rigid floor or rigid ground surface pavement, and where no lateral constraint is provided above the ground surface, such as by a structural diaphragm.

$d = 0.5A\{1 + [1 + (4.36h/A)]^{1/2}\}$ **(Equation 18-1)**

where:

<u>[formula is valid for SI and Imperial units]</u>

A = $2.34P/S_1$ b.

b = Diameter of round post or footing or diagonal dimension of square post or footing, feet (m).

d = Depth of embedment in earth in feet (m) but not over 12 feet (3658 mm) for purpose of computing lateral pressure.

h = Distance in feet (m) from ground surface to point of application of "P."

P = Applied lateral force in pounds (kN).
S_1 = Allowable lateral soil-bearing pressure as set forth in Section 1806.2 based on a depth of one-third the depth of embedment in pounds per square foot (psf) (kPa).

1807.3.2.2 Constrained. The following formula shall be used to determine the depth of embedment required to resist lateral loads where lateral constraint is provided at the ground surface, such as by a rigid floor or pavement.

$$d = \sqrt{\frac{4.25Ph}{S_3 b}}$$ **(Equation 18-2)**

or alternatively

$$d = \sqrt{\frac{4.25 M_g}{S_3 b}}$$ **(Equation 18-3)**

where:

<u>[formula is valid for SI and Imperial units]</u>

M_g = Moment in the post at grade, in foot-pounds (kN-m).
S_3 = Allowable lateral soil-bearing pressure as set forth in Section 1806.2 based on a depth equal to the depth of embedment in pounds per square foot (kPa).

Reason: If formulas are valid for both Imperial and SI, I'd like it stated explicitly in the code. As near as I can tell, the units cancel in the formulas, so they're valid in both Imperial and SI.

Alternatively, it the above is not correct, all the metric equivalents should be deleted from the definitions following the formulas as follows:
b = Diameter of round post or footing or diagonal dimension of square post or footing, feet ~~(m)~~.
d = Depth of embedment in earth in feet ~~(m)~~ but not over 12 ~~(3658 mm)~~ feet for purpose of computing lateral pressure.
P = Applied lateral force in pounds ~~(kN)~~.
h = Distance in feet ~~(m)~~ from ground surface to point of application of ~~"~~P.~~"~~
S_1 = Allowable lateral soil bearing pressure as set forth in Section 1804.3 based on a depth of one-third the depth of embedment in pounds per square foot (psf) ~~(kPa)~~.
Mg = Moment in the post at grade, in foot-pounds ~~(kN-m).~~
S3 = Allowable lateral soil-bearing pressure as set forth in Section 1806.2 based on a depth equal to the depth of embedment in pounds per square foot ~~(kPa).~~

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

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

ICCFILENAME: JOHNSON-S4-1807.3.2.1-3.DOC

S153–09/10
1807.3.2.1, 1807.3.2.2

Proponent: Brian Johnson, PE, representing self

Revise as follows:

1807.3.2.1 Nonconstrained. The following formula shall be used in determining the depth of embedment required to resist lateral loads where no lateral constraint is provided at the ground surface, such as by a rigid floor or rigid ground surface pavement, and where no lateral constraint is provided above the ground surface, such as by a structural diaphragm.

$D = 0.5A\{1 + [1 + (4.36h/A)]^{1/2}\}$ **(Equation 18-1)**

where:

A = $2.34P/S_1b$.
B = Diameter of round post or footing or diagonal dimension of square post or footing, feet (m).

d = Depth of embedment in earth in feet (m) but not over ~~12 (3658 mm)~~ 15 feet (4.57 m) for purpose of computing lateral pressure.
H = Distance in feet (m) from ground surface to point of application of "*P*."
P = Applied lateral force in pounds (kN).
S_1 = Allowable lateral soil-bearing pressure as set forth in Section 1806.2 based on a depth of one-third the depth of embedment in pounds per square foot (psf) (kPa).

1807.3.2.2 Constrained. The following formula shall be used to determine the depth of embedment required to resist lateral loads where lateral constraint is provided at the ground surface, such as by a rigid floor or pavement.

$$d = \sqrt{\frac{4.25Ph}{S_3 b}}$$ **(Equation 18-2)**

or alternatively

$$d = \sqrt{\frac{4.25M}{S_3 b}}$$ **(Equation 18-3)**

where:

d = depth of embedment in earth in feet (m) but not over 15 feet (4.57 m) for purpose of computing lateral pressure.
M_g = Moment in the post at grade, in foot-pounds (kN-m).
S_3 = Allowable lateral soil-bearing pressure as set forth in Section 1806.2 based on a depth equal to the depth of embedment in pounds per square foot (kPa).

Reason: To bring limitation to match section 1804.3.1. While we're at it, why not fix the metric dimension to something sensible. Who designs pole embedment to the millimeter?

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

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

ICCFILENAME: JOHNSON-S5-1807.3.2.1-4.DOC

S154–09/10
1807.3.2.2

Proponent: Brian Johnson, PE, representing self

Revise as follows:

1807.3.2.2 Constrained. The following formula shall be used to determine the depth of embedment required to resist lateral loads where lateral constraint is provided at the ground surface, such as by a rigid floor or pavement.

$$d = \sqrt{\frac{4.25Ph}{S_3 b}}$$ **(Equation 18-2)**

or alternatively

$$d = \sqrt{\frac{4.25M_g}{S_3 b}}$$ **(Equation 18-3)**

where:

b = Diameter of round post or footing or diagonal dimension of square post or footing, feet (m).
d = Depth of embedment in earth in feet (m)
h = Distance in feet (m) from ground surface to point of application of P.

M_g = Moment in the post at grade, in foot-pounds (kN-m).
<u>P = Applied lateral force in pounds (kN).</u>
S_3 = Allowable lateral soil-bearing pressure as set forth in Section 1806.2 based on a depth equal to the depth of embedment in pounds per square foot (kPa) <u>but not over 12 feet (3.66 m) for purpose of computing lateral pressure</u>.

Reason: The restriction on *d* doesn't make sense [see section 1807.3.2.1 definition), the restriction should be on the soil pressure. Definitions of *b*, *d*, *h*, and *P* added to eliminate cross-reference and page flipping.

Cost Impact: Decrease the cost of construction by clarifying formula, simplifying formulas, and preventing cross-referencing.

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

ICCFILENAME: JOHNSON-S3-1807.3.2.2.DOC

S155–09/10
1810.3.3.1.5

Proponent: Lori A. Simpson, Treadwell & Rollo, Inc., representing self

Revise as follows:

1810.3.3.1.5 Uplift capacity of a single deep foundation element. Where required by the design, the uplift capacity of a single deep foundation element shall be determined by an *approved* method of analysis based on a minimum factor of safety of three or by load tests conducted in accordance with ASTM D 3689. The maximum allowable uplift load shall not exceed the ultimate load capacity as determined in Section 1810.3.3.1.2, using the results of load tests conducted in accordance withASTMD3689, divided by a factor of safety of two.

> **Exception:** Where uplift is due to wind or seismic loading, the minimum factor of safety shall be two where capacity is determined by an analysis and one and one-half where capacity is determined by load tests. <u>A factor of safety of one and one-half is allowed where uplift capacity is determined by an analysis and the local soil conditions are well understood, as substantiated in the geotechnical investigation.</u>

Reason: It is the standard of practice in the San Francisco Bay Area to use a factor of safety of 1.5 for temporary uplift loads, such as wind or seismic. Although there can be cyclic degradation in the strength of the soil, it is offset by an increase in strength during rapid loading. With well known soil conditions, the uplift capacity is well-defined and should not be reduced by a factor of safety of 2 for temporary loading conditions.

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

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

ICCFILENAME: SIMPSON-S2-1810.3.3.1.5

S156–09/10
1810.3.3.1.6

Proponent: Lori A. Simpson, Treadwell & Rollo, Inc., representing self

Revise as follows:

1810.3.3.1.6 Uplift capacity of grouped deep foundation elements. For grouped deep foundation elements subjected to uplift, the allowable working uplift load for the group shall be calculated by an *approved* method of analysis. Where the deep foundation elements in the group are placed at a center-to-center spacing of at least 2.5 times the least horizontal dimension of the largest single element, the allowable working uplift load for the group is permitted to be calculated as the lesser of:

1. The proposed individual uplift working load times the number of elements in the group.
2. Two-thirds of the effective weight of the group and the soil contained within a block defined by the perimeter of the group and the length of the element<u>, plus two-thirds of the ultimate shear resistance along the soil block</u>.

Reason: Allowing only the weight of the piles and soil in the block is unreasonably conservative; not only the weight but also the shear resistance will be developed during an uplift loading event.

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

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

ICCFILENAME: SIMPSON-S3-1810.3.3.1.6

S157–09/10
1810.3.9.7 (New)

Proponent: Michael Morgano, GRL Engineers, representing self

Add new text as follows:

<u>1810.3.9.7 Pile integrity testing .</u> <u>The structural integrity of cast-in-place deep foundation elements shall be verified using pulse echo integrity testing.</u>

Reason: Due to the installation methods required for cast-in-place piles, it is not possible to inspect the element after installation. Integrity testing via pulse echo methods is commonly available, the most economical and easily applied test.

The Federal Highway Administration "Geotechnical Engineering Circular (GEC) No. 8" specifically states in section 7.5.2 (page 184) "The most commonly available, economical, and easily applied type of integrity test is the sonic echo test. The advantage of the method is that a test can be performed rapidly, inexpensively, and without any internal instrumentation or tubes in the pile."

The Federal Highway Administration "Geotechnical Engineering Circular (GEC) No. 8" specifically states in section 8.3.12 (page 221) "Post-installation integrity tests shall be performed on a minimum of 20% of the production piles."

A copy of the Federal Highway Administration "Geotechnical Engineering Circular (GEC) No. 8"can be downloaded at no cost from the following site: www.fhwa.dot.gov/engineering/geotech/pubs/gec8.

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

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

ICCFILENAME: MORGANO-S3-1810.3.9.7

S158–09/10
1810.4.8

Proponent: Michael Morgano, GRL Engineers, representing self

Revise as follows:

1810.4.8 Hollow-stem augered, cast-in-place elements. Where concrete or grout is placed by pumping through a hollow-stem auger, the auger shall be permitted to rotate in a clockwise direction during withdrawal. As the auger is withdrawn at a steady rate or in increments not to exceed 1 foot (305 mm), concreting or grouting pumping pressures shall be measured <u>with Automated Monitoring equipment</u> and maintained high enough at all times to offset hydrostatic and lateral earth pressures. Concrete or grout volumes shall be measured to ensure that the volume of concrete or grout placed in each element is equal to or greater than the theoretical volume of the hole created by the auger. Where the installation process of any element is interrupted or a loss of concreting or grouting pressure occurs, the element shall be redrilled to 5 feet (1524 mm) below the elevation of the tip of the auger when the installation was interrupted or concrete or grout pressure was lost and reformed. Augered cast-in-place elements shall not be installed within six diameters center to center of an element filled with concrete or grout less than 12 hours old, unless *approved* by the *building official*. If the concrete or grout level in any completed element drops due to installation of an adjacent element, the element shall be replaced.

Reason: Using current practice, it is not possible to measure and maintain the grouting pressures during the installation process. These pressures are typically measured with an analog pressure gage which is installed at the pump. These analog pressure gages provide very low resolution, poor frequency response, and have no ability to record and therefore verify that the pressure remained high enough at all times. The crane operator is the individual that needs to see these pressure readings as he has direct control over the installation process, yet with current practice the crane operator will never see these values. Furthermore, it is not possible for the crane operator (or any other individuals on site) to determine lift heights to an accuracy of 1 foot, as the standard practice is to mark the leads at approximately every 5 feet (with many rigs being totally unmarked). Without the use of Automated Monitoring Equipment, this lifting height is simply a rough estimate made by the crane operator. Additionally, as augered cast-in-place pile length increase, it is not possible for the crane operator to visually determine the lift heights accurately due to parallax.
The Federal Highway Administration "Geotechnical Engineering Circular (GEC) No. 8" specifically states in section 7.4.2 that in general, the simple manual observation and control system is not considered to provide sufficient control for transportation projects. The system that is recommended for transportation projects includes automated monitoring of the auger position; volume of grout/concrete that is delivered; pressure with which it is delivered; and rotation and lifting of the auger.

A copy of the Federal Highway Administration "Geotechnical Engineering Circular (GEC) No. 8"can be downloaded at no cost from the following site: www.fhwa.dot.gov/engineering/geotech/pubs/gec8.

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

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

ICCFILENAME: MORGANO-S1-1810.4.8

S159–09/10
1810.4.8

Proponent: Michael Morgano, GRL Engineers, representing self.

Revise as follows:

1810.4.8 Hollow-stem augered, cast-in-place elements. Where concrete or grout is placed by pumping through a hollow-stem auger, the auger shall be permitted to rotate in a clockwise direction during withdrawal. As the auger is withdrawn at a steady rate or in increments not to exceed 1 foot (305 mm), concreting or grouting pumping pressures shall be measured and maintained high enough at all times to offset hydrostatic and lateral earth pressures. Concrete or grout volumes shall be measured <u>with a magnetic flow meter as part of an Automated Monitoring system</u> to ensure that the volume of concrete or grout placed in each element is equal to or greater than the theoretical volume of the hole created by the auger. <u>This volume of concrete or grout shall be measured and recorded for every 2 feet (610 mm) of auger withdrawal.</u> Where the installation process of any element is interrupted or a loss of concreting or grouting pressure occurs, the element shall be redrilled to 5 feet (1524 mm) below the elevation of the tip of the auger when the installation was interrupted or concrete or grout pressure was lost and reformed. Augered cast-in-place elements shall not be installed within six diameters center to center of an element filled with concrete or grout less than 12 hours old, unless *approved* by the *building official*. If the concrete or grout level in any completed element drops due to installation of an adjacent element, the element shall be replaced.

Reason: Manual control of the grout/concrete placement process involves counting pump strokes and assigning an approximate volume to each pump stroke. It is widely known that grout/concrete pumps do not maintain a constant volume for every pump stroke. These mechanical pumps a highly variable and it is not possible to determine with any degree of accuracy the volume associated with any particular pump stroke. Further, it is common for a mechanical pump to miss several pump strokes in succession, leaving the possibility for serious voids in the element. It is generally not possible for an individual counting pump strokes or for a mechanical stroke counter to discern when the mechanical pump operates normally and when it fails to deliver full theoretical grout/ concrete volume for any stroke. The only reliable method for determining grout/concrete volume is to measure it directly. According to the Federal Highway Administration "General Engineering Circular (GEC) No. 8" section 7.4.2 "In general, the simple manual observation and control system is not considered to provide sufficient control for transportation projects. The system recommended for transportation projects includes automated monitoring of the auger position; volume of concrete delivered (measured by an in-line flow meter that provides a reliable and accurate measure of the grout/concrete that is delivered in real time); pressure with which it is delivered; and rotation and lifting speed of the auger, with the entire process recorded as part of the documentation process.

Measuring the total volume for an element is not sufficient. There is a significant risk in only obtaining the overall volume placed as there can be voids throughout the element that can only be detected by measuring the incremental volume placed in the element. According to the Federal Highway Administration "General Engineering Circular (GEC) No. 8" section 8.3.6 " The volume of grout or concrete placed as a function of depth shall be measured at intervals not exceeding 2 ft (0.6m) using automated monitoring equipment."

A copy of the Federal Highway Administration "Geotechnical Engineering Circular (GEC) No. 8"can be downloaded at no cost from the following site: www.fhwa.dot.gov/engineering/geotech/pubs/gec8.

According to the Deep Foundation Institute's "AUGERED CAST-IN-PLACE PILE MANUAL" section 3.4.7 "The most desirable precautions that are available in the construction of ACIP piles are to: pump the specified initial grout head prior to the withdrawal of the auger, maintain grout pumping while installing the required incremental grout volume as the auger is being withdrawn, and observing grout return when the auger's depth below the ground surface is equal or greater than the specified initial grout head. The observation of these three conditions are the most useful quality control tools that can be utilized in ACIP pile construction."

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

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

ICCFILENAME: MORGANO-S2-1810.4.8

S160–09/10
1901.3 through 1901.4, 1904, 1905, 1906, 1907, 1909

Proponent: Matthew Senecal, PE, American Concrete Institute, representing American Concrete Institute

1. Delete without substitution:

~~**1901.3 Source and applicability.** The format and subject matter of Sections 1902 through 1907 of this chapter are patterned after, and in general conformity with, the provisions for structural concrete in ACI 318.~~

2. Revise as follows:

~~**1901.4**~~ **<u>1901.3</u> Construction documents.** The *construction documents* for structural concrete construction shall include:

1. The specified compressive strength of concrete at the stated ages or stages of construction for which each concrete element is designed.
2. The specified strength or grade of reinforcement.
3. The size and location of structural elements, reinforcement and anchors.
4. Provision for dimensional changes resulting from creep, shrinkage and temperature.
5. The magnitude and location of prestressing forces.
6. Anchorage length of reinforcement and location and length of lap splices.
7. Type and location of mechanical and welded splices of reinforcement.
8. Details and location of contraction or isolation joints specified for plain concrete.
9. Minimum concrete compressive strength at time of posttensioning.
10. Stressing sequence for posttensioning tendons.
11. For structures assigned to *Seismic Design Category* D, E or F, a statement if slab on grade is designed as a structural diaphragm ~~(see Section 21.12.3.4 of ACI 318)~~.

(Renumber remaining sections)

3. Delete without substitution:

~~**1904.1 Water-cementitious materials ratio.** Where maximum water-cementitious materials ratios are specified in ACI 318, they shall be calculated in accordance with ACI 318, Section 4.1.~~

4. Revise as follows:

~~**1904.2**~~ **<u>1904.1</u> Exposure categories and classes.** Concrete shall be assigned to exposure classes in accordance with <u>the durability requirements of</u> ACI 318~~, Section 4.2,~~ based on:

1. Exposure to freezing and thawing in a moist condition or deicer chemicals;
2. Exposure to sulfates in water or soil;
3. Exposure to water where the concrete is intended to have low permeability; and
4. Exposure to chlorides from deicing chemicals, salt, saltwater, brackish water, seawater or spray from these sources, where the concrete has steel reinforcement.

~~**1904.3**~~ **<u>1904.2</u> Concrete properties.** Concrete mixtures shall conform to the most restrictive maximum water-cementitious materials ratios, <u>maximum cementitious admixtures,</u> <u>minimum air-entrainment</u> and minimum specified concrete compressive strength requirements of ACI 318~~, Section 4.3,~~ based on the exposure classes assigned in Section ~~1904.2~~ <u>1904.1</u>.

Exception: For occupancies and appurtenances thereto in Group R occupancies that are in buildings less than four stories above grade plane, normal-weight aggregate concrete is permitted to comply with the requirements of Table ~~1904.3~~ <u>1904.2</u> based on the weathering classification (freezing and thawing) determined from Figure ~~1904.3~~ <u>1904.2</u> in lieu of the <u>durability</u> requirements of ACI 318~~, Table 4.3.1~~.

TABLE ~~1904.3~~ <u>1904.2</u>
MINIMUM SPECIFIED COMPRESSIVE STRENGTH
No change to table

a. Concrete in these locations that can be subjected to freezing and thawing during construction shall be of air-entrained concrete in accordance with ~~1904.3~~ <u>1904.2</u> ~~Section 1904.4.1~~.
b. Concrete shall be air entrained in accordance with <u>ACI 318</u> ~~Section 1904.4.1~~.
c. Structural plain concrete basement walls are exempt from the requirements for exposure conditions of Section ~~1904.3~~ <u>1904.2</u> (see Section 1909.6.1).
d. For garage floor slabs where a steel trowel finish is used, the total air content required by <u>ACI 318</u> ~~Section 1904.4.1~~ is permitted to be reduced to not less than 3 percent, provided the minimum specified compressive strength of the concrete is increase to 4,000 psi.

FIGURE ~~1904.3~~ <u>1904.2</u>
WEATHERING PROBABILITY MAP FOR CONCRETE [a, b, c]
(No changes to map and footnotes)

5. Delete without substitution:

~~**1904.4 Freezing and thawing exposures.** Concrete that will be exposed to freezing and thawing, in the presence of moisture, with or without deicing chemicals being present, shall comply with Sections 1904.4.1 and 1904.4.2.~~

~~**1904.4.1 Air entrainment.** Concrete exposed to freezing and thawing while moist shall be air entrained in accordance with ACI 318, Section 4.4.1.~~

~~**1904.4.2 Deicing chemicals.** For concrete exposed to freezing and thawing in the presence of moisture and deicing chemicals, the maximum weight of fly ash, other pozzolans, silica fume or slag that is included in the concrete shall not exceed the percentages of the total weight of cementitious materials permitted by ACI 318, Section 4.4.2.~~

~~**1904.5 Alternative cementitious materials for sulfate exposure.** Alternative combinations of cementitious materials for use in sulfate-resistant concrete to those listed in ACI 318, Table 4.3.1 shall be permitted in accordance with ACI 318, Section 4.5.1.~~

~~SECTION 1905~~
~~CONCRETE QUALITY, MIXING AND PLACING~~

~~**1905.1 General.** The required strength and durability of concrete shall be determined by compliance with the proportioning, testing, mixing and placing provisions of Sections 1905.1.1 through 1905.13.~~

~~**1905.1.1 Strength.** Concrete shall be proportioned to provide an average compressive strength as prescribed in Section 1905.3 and shall satisfy the durability criteria of Section 1904. Concrete shall be produced to minimize the frequency of strengths below f'_c as prescribed in Section 1905.6.3. For concrete designed and constructed in accordance with this chapter, f'_c shall not be less than 2,500 psi (17.22 MPa). No maximum specified compressive strength shall apply unless restricted by a specific provision of this code or ACI 318.~~

~~**1905.2 Selection of concrete proportions.** Concrete proportions shall be determined in accordance with the provisions of ACI 318, Section 5.2.~~

~~**1905.3 Proportioning on the basis of field experience and/or trial mixtures.** Concrete proportioning determined on the basis of field experience and/or trial mixtures shall be done in accordance with ACI 318, Section 5.3.~~

~~**1905.4 Proportioning without field experience or trial mixtures.** Concrete proportioning determined without field experience or trial mixtures shall be done in accordance with ACI 318, Section 5.4.~~

~~**1905.5 Average strength reduction.** As data become available during construction, it is permissible to reduce the amount by which the average compressive strength (f'_c) is required to exceed the specified value of f'_c in accordance with ACI 318, Section 5.5.~~

~~**1905.6 Evaluation and acceptance of concrete.** The criteria for evaluation and acceptance of concrete shall be as specified in Sections 1905.6.2 through 1905.6.5.~~

~~**1905.6.1 Qualified technicians.** Concrete shall be tested in accordance with the requirements in Sections 1905.6.2 through 1905.6.5. Qualified field testing technicians shall perform tests on fresh concrete at the job site, prepare specimens required for curing under field conditions, prepare specimens required for testing in the laboratory and record the temperature of the fresh concrete when preparing specimens for strength tests. Qualified laboratory technicians shall perform all required laboratory tests.~~

~~**1905.6.2 Frequency of testing.** The frequency of conducting strength tests of concrete and the minimum number of tests shall be as specified in ACI 318, Section 5.6.2.~~

> ~~**Exception:** When the total volume of a given class of concrete is less than 50 cubic yards (38m3), strength tests are not required when evidence of satisfactory strength is submitted to and approved by the building official.~~

~~**1905.6.3 Strength test specimens.** Specimens prepared for acceptance testing of concrete in accordance with Section 1905.6.2 and strength test acceptance criteria shall comply with the provisions of ACI 318, Section 5.6.3.~~

~~**1905.6.4 Field-cured specimens.** Where required by the building official to determine adequacy of curing and protection of concrete in the structure, specimens shall be prepared, cured, tested and test results evaluated for acceptance in accordance with ACI 318, Section 5.6.4.~~

~~**1905.6.5 Low-strength test results.** Where any strength test (see ACI 318, Section 5.6.2.4) falls below the specified value of f'_c, the provisions of ACI 318, Section 5.6.5, shall apply.~~

~~**1905.7 Preparation of equipment and place of deposit.** Prior to concrete being placed, the space to receive the concrete and the equipment used to deposit it shall comply with ACI 318, Section 5.7.~~

~~**1905.8 Mixing**. Mixing of concrete shall be performed in accordance with ACI 318, Section 5.8.~~

~~**1905.9 Conveying.** The method and equipment for conveying concrete to the place of deposit shall comply with ACI 318, Section 5.9.~~

~~**1905.10 Depositing.** The depositing of concrete shall comply with the provisions of ACI 318, Section 5.10.~~

~~**1905.11 Curing.** The length of time, temperature and moisture conditions for curing of concrete shall be in accordance with ACI 318, Section 5.11.~~

~~**1905.12 Cold weather requirements.** Concrete to be placed during freezing or near-freezing weather shall comply with the requirements of ACI 318, Section 5.12.~~

~~**1905.13 Hot weather requirements.** Concrete to be placed during hot weather shall comply with the requirements of ACI 318, Section 5.13.~~

~~SECTION 1906~~
~~FORMWORK, EMBEDDED PIPES AND CONSTRUCTION JOINTS~~

~~**1906.1 Formwork.** The design, fabrication and erection of forms shall comply with ACI 318, Section 6.1.~~

~~**1906.2 Removal of forms, shores and reshores.** The removal of forms and shores, including from slabs and beams (except where cast on the ground), and the installation of reshores shall comply with ACI 318, Section 6.2.~~

~~**1906.3 Conduits and pipes embedded in concrete.** Conduits, pipes and sleeves of any material not harmful to concrete and within the limitations of ACI 318, Section 6.3, are permitted to be embedded in concrete with approval of the registered design professional.~~

~~**1906.4 Construction joints.** Construction joints, including their location, shall comply with the provisions of ACI 318, Section 6.4.~~

~~SECTION 1907~~
~~DETAILS OF REINFORCEMENT~~

~~**1907.1 Hooks.** Standard hooks on reinforcing bars used in concrete construction shall comply withACI 318, Section 7.1.~~

~~**1907.2 Minimum bend diameters.** Minimum reinforcement bend diameters utilized in concrete construction shall comply with ACI 318, Section 7.2.~~

~~**1907.3 Bending.** The bending of reinforcement shall comply with ACI 318, Section 7.3.~~

~~**1907.4 Surface conditions of reinforcement.** The surface conditions of reinforcement shall comply with the provisions of ACI 318, Section 7.4.~~

~~**1907.5 Placing reinforcement.** The placement of reinforcement, including tolerances on depth and cover, shall comply with the provisions of ACI 318, Section 7.5. Reinforcement shall be accurately placed and adequately supported before concrete is placed.~~

~~**1907.6 Spacing limits for reinforcement.** The clear distance between reinforcing bars, bundled bars, tendons and ducts shall comply with ACI 318, Section 7.6.~~

~~**1907.7 Concrete protection for reinforcement.** The minimum specified concrete cover for reinforcement shall comply with Sections 1907.7.1 through 1907.7.8.~~

~~**1907.7.1 Cast-in-place concrete (nonprestressed).** Minimum specified concrete cover shall be provided for reinforcement in nonprestressed, cast-in-place concrete construction in accordance with ACI 318, Section 7.7.1.~~

~~**1907.7.2 Cast-in-place concrete (prestressed).** The minimum specified concrete cover for prestressed and nonprestressed reinforcement, ducts and end fittings in cast-in-place prestressed concrete shall comply with ACI 318, Section 7.7.2.~~

~~**1907.7.3 Precast concrete (manufactured under plant control conditions).** The minimum specified concrete cover for prestressed and nonprestressed reinforcement, ducts and end fittings in precast concrete manufactured under plant control conditions shall comply with ACI 318, Section 7.7.3.~~

~~**1907.7.4 Bundled bars.** The minimum specified concrete cover for bundled bars shall comply with ACI 318, Section 7.7.4.~~

~~**1907.7.5 Headed shear stud reinforcement.** For headed shear stud reinforcement, the minimum specified concrete cover shall comply with ACI 318, Section 7.7.5.~~

~~**1907.7.6 Corrosive environments.** In corrosive environments or other severe exposure conditions, prestressed and nonprestressed reinforcement shall be provided with additional protection in accordance with ACI 318, Section 7.7.6.~~

~~**1907.7.7 Future extensions.** Exposed reinforcement, inserts and plates intended for bonding with future extensions shall be protected from corrosion.~~

~~**1907.7.8 Fire protection.** When this code requires a thickness of cover for fire protection greater than the minimum concrete cover in Section 1907.7, such greater thickness shall be specified.~~

~~**1907.8 Special reinforcement details for columns.** Offset bent longitudinal bars in columns and load transfer in structural steel cores of composite compression members shall comply with the provisions of ACI 318, Section 7.8.~~

~~**1907.9 Connections.** Connections between concrete framing members shall comply with the provisions of ACI 318, Section 7.9.~~

~~**1907.10 Lateral reinforcement for compression members.** Lateral reinforcement for concrete compression members shall comply with the provisions of ACI 318, Section 7.10.~~

~~**1907.11 Lateral reinforcement for flexural members.** Lateral reinforcement for compression reinforcement in concrete flexural members shall comply with the provisions of ACI 318, Section 7.11.~~

~~**1907.12 Shrinkage and temperature reinforcement.** Reinforcement for shrinkage and temperature stresses in concrete members shall comply with the provisions of ACI 318, Section 7.12.~~

~~**1907.13 Requirements for structural integrity.** The detailing of reinforcement and connections between concrete members shall comply with the provisions of ACI 318, Section 7.13, to improve structural integrity.~~

(Renumber remaining sections)

6. Revise as follows:

~~**1909.1**~~ **1906.1 Scope**. The design and construction of structural plain concrete, both cast-in-place and precast, shall comply with the minimum requirements of ~~Section 1909 and~~ ACI 318, ~~Chapter 22,~~ as modified in Section ~~1908~~1905 .

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

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

7. Delete without substitution:

~~**1909.1.1 Special structures**. For special structures, such as arches, underground utility structures, gravity walls and shielding walls, the provisions of this section shall govern where applicable.~~

~~**1909.2 Limitations.** The use of structural plain concrete shall be limited to:~~

1. ~~Members that are continuously supported by soil, such as walls and footings, or by other structural members capable of providing continuous vertical support.~~
2. ~~Members for which arch action provides compression under all conditions of loading.~~
3. ~~Walls and pedestals.~~
   ~~The use of structural plain concrete columns and structural plain concrete footings on piles is not permitted. See Section 1908.1.8 for additional limitations on the use of structural plain concrete.~~

~~**1909.3 Joints.** Contraction or isolation joints shall be provided to divide structural plain concrete members into flexurally discontinuous elements in accordance with ACI 318, Section 22.3.~~

~~**1909.5 Precast members.** The design, fabrication, transportation and erection of precast, structural plain concrete elements shall be in accordance with ACI 318, Section 22.9.~~

~~**1909.6 Walls.** In addition to the requirements of this section, structural plain concrete walls shall comply with the applicable requirements of ACI 318, Chapter 22~~.

~~**1909.6.1 Basement walls.** The thickness of exterior basement walls and foundation walls shall be not less than 7-1/2 inches (191 mm).~~

~~**1909.6.2 Other walls**. Except as provided for in Section 1909.6.1, the thickness of bearing walls shall be not less than 1/24 the unsupported height or length, whichever is shorter, but not less than 5-1/2 inches (140 mm).~~

~~**1909.6.3 Openings in walls**. Not less than one No. 5 bar shall be provided around window, door and similar sized openings. The bar shall be anchored to develop f_y in tension at the corners of openings.~~

(Renumber remaining sections)

Reason: This code change proposal removes provisions in the IBC that do not add new code requirements but either inform the user where information is located in ACI 318 or repeats ACI 318 information. The change proposal is strictly editorial in nature. The current format requires users to thoroughly read these provisions to determine if the IBC modifies ACI 318. By removing these provisions, the IBC will be easier for users to understand. Additionally, by removing these provisions the job of coordination information between the IBC and ACI 318 is made much easier.

The following explanations are given to for the revised Sections stated above:

1901.3	Statement is no longer necessary due to the removal of most of this information.
1901.4	Section 21.12.3.4 states the same information that is given in Item 11. No need for additional referencing.
Section 1904	This section repeats or points to the information in ACI 318, except for the alternate durability map. This section was reduced to the minimum language that would still allow for the alternate.
Section 1905	This section repeats or points to the information in ACI 318 without exception. This section was removed.
Section 1906	This section repeats or points to the information in ACI 318 without exception. This section was removed.
Section 1907	This section repeats or points to the information in ACI 318 without exception. This section was removed.
Section 1909	This section repeats or points to the information in ACI 318. There is only one exception to the code. The section was reworked to state only the exception.

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

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

ICCFILENAME: SENECAL-S6-TABLE 1704.4

S161-09/10
1903.3, Chapter 35

Proponent: Steve Heller, representing the Insulating Concrete Form Association

1. Revise as follows:

1903.3 Flat wall insulating concrete form (ICF) systems. Insulating concrete form material used for forming flat concrete walls shall conform to ASTM E 2634.

2. Add New Standard to Chapter 35 as follows:

ASTM International
E2634—08 Standard Specification for Flat Wall Insulating Concrete Form (ICF) Systems……………………………………………………………..1903.3

Reason: This proposal adds ASTM E 2634 to the code to help users determine acceptance of Flat Wall ICF Form Systems.

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

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM E2634-08, 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.

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

FILENAME: HELLER-S1-1903.3 NEW

S162–09/10
Table 1704.3, Table 1704.4, 1704.4.1, 1708.2, 1808.8.2, Table 1808.8.2, 1808.8.5, 1808.8.6, 1810.2.4.1, 1810.3.2.1.2, 1810.3.8.3.3, 1810.3.9.4.2.1, 1810.3.9.4.2.2, 1901.4, 1904, 1905, 1906, 1907, 1909; IRC R402.2

Proponent: Matthew Senecal, PE, American Concrete Institute, representing American Concrete Institute

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC-STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

TABLE 1704.3
REQUIRED VERIFICATION AND INSPECTION OF STEEL CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERNCED STANDARD	IBC REFERENCE
5. Inspection of welding				
b. Reinforcing steel:				
1) Verification of weldability of reinforcing steel other than ASTM A 706.	—	X		
2) Reinforcing steel-resisting flexural and axial forces in intermediate and special moment frames, and boundary elements of special structural walls of concrete and shear reinforcement.	X	—	AWS D1.4 ACI 318~~: Section 3.5.2~~	—
3) Shear reinforcement.	X	—		
4) Other reinforcing steel.	—	X		

(Remainder of table is unchanged; No changes to footnotes)

TABLE 1704.4
REQUIRED VERIFICATION AND INSPECTION OF CONCRETE CONSTRUCTION

VERIFICATION AND INSPECTION	CONTINUOUS	PERIODIC	REFERNCED STANDARD	IBC REFERENCE
1. Inspection of reinforcing steel, including prestressing tendons, and placement.	—	X	ACI 318~~: 3.5, 7.1-7.7~~	1913.4
2. Inspection of reinforcing steel welding in accordance with Table 1704.3, Item 5b.	—	—	AWS D1.4 ACI 318~~: 3.5.2~~	—
3. Inspection of bolts to be installed in concrete prior to and during placement of concrete where allowable loads have been increased or where strength design is used.	X	—	ACI 318~~:~~ ~~8.1.3, 21.2.8~~	1911.5 1912.1
4. Inspection of anchors installed in hardened concrete.	—	X	ACI 318~~:~~ ~~3.8.6, 8.1.3, 21.2.8~~	1912.1
5. Verifying use of required design mix.	—	X	ACI 318~~:~~ ~~Ch. 4, 5.2-5.4~~	1904.2.2 1913.2 1913.3
6. At the time fresh concrete is sampled to fabricate specimens for strength tests, perform slump and air content tests, and determine the temperature of the concrete.	X	—	ASTM C 172 ASTM C 31 ACI 318~~: 5.6, 5.8~~	1913.10
7. Inspection of concrete and shotcrete placement for proper application techniques.	X	—	ACI 318~~: 5.9, 5.10~~	1913.6 1913.7 1913.8
8. Inspection for maintenance of specified curing temperature and techniques.	—	X	ACI 318~~: 5.11-5.13~~	1913.9
9. Inspection of prestressed concrete: a. Application of prestressing forces. b. Grouting of bonded prestressing tendons in the seismic-force-resisting system.	 X X	—	ACI 318~~: 18.20~~ ~~ACI 318: 18.18.4~~	—
10. Erection of precast concrete members.	—	X	ACI 318~~: Ch. 16~~	—
11. Verification of in-situ concrete strength, prior to stressing of tendons in posttensioned concrete and prior to removal of shores and forms from beams and structural slabs.	—	X	ACI 318~~: 6.2~~	—
12. Inspect formwork for shape, location and dimensions of the concrete member being formed.	—	X	ACI 318~~: 6.1.1~~	—

(No changes to footnotes)

1704.4.1 Materials. In the absence of sufficient data or documentation providing evidence of conformance to quality standards for materials in ~~Chapter 3 of~~ ACI 318, the building official shall require testing of materials in accordance with the appropriate standards and criteria for the material in ~~Chapter 3 of~~ ACI 318. Weldability of reinforcement, except that which conforms to ASTM A 706, shall be determined in accordance with the steel reinforcement material requirements of ~~Section 3.5.2 of~~ ACI 318.

1708.2 Concrete reinforcement. Where reinforcement complying with ASTM A 615 is used to resist earthquake-induced flexural and axial forces in special moment frames, special structural walls and coupling beams connecting special structural walls, in structures assigned to *Seismic Design Category* B, C, D, E or F as determined in Section 1613, the reinforcement shall comply with deformed reinforcement material requirements ~~Section 21.1.5.2~~ of ACI 318 for special moment frames and special structural walls. Certified mill test reports shall be provided for each shipment of such reinforcement. ~~Where reinforcement complying with ASTM A 615 is to be welded, chemical tests shall be performed to determine weldability in accordance with Section 3.5.2 of ACI 318.~~

1808.8.2 Concrete cover. The concrete cover provided for prestressed and nonprestressed reinforcement in foundations shall be no less than the largest applicable value specified in Table 1808.8.2. Longitudinal bars spaced less than 1½ inches (38 mm) clear distance apart shall be considered as bundled bars according to ACI 318 for ~~which~~ the determination of concrete cover ~~provided shall also be no less than that required by Section 7.7.4 of ACI 318~~. Concrete cover shall be measured from the concrete surface to the outermost surface of the steel to which the cover requirements applies. Where concrete is placed in a temporary or permanent casing or a mandrel, the inside face of the casing or mandrel shall be considered the concrete surface.

TABLE 1808.8.2
MINIMUM SPECIFIED CONCRETE COVER

FOUNDATION ELEMENT OR CONDTITION	MINIMUM COVER
1. Shallow foundations	~~In accordance with Section 7.7 of~~ ACI 318
2. Precast nonprestressed deep foundation elements Exposed to seawater Not manufactured under plant conditions Manufactured under plant control conditions	 3 inches 2 inches ~~In accordance with Section 7.7.3 of~~ Cover requirements for precast concrete manufactured under plant controlled conditions per ACI 318
3. Precast prestressed deep foundation elements Exposed to seawater Other	 2.5 inches ~~In accordance with Section 7.7.3 of~~ Cover requirements for precast concrete manufactured under plant controlled conditions per ACI 318
4. Cast-in-place deep foundation elements not enclosed by a steel pipe, tube or permanent casing	2.5 inches
5. Cast-in-place deep foundation elements enclosed by a steel pipe, tube or permanent casing	1 inch
6. Structural steel core within a steel pipe, tube or permanent casing	2 inches
7. Cast-in-place drilled shafts enclosed by a stable rock socket	1.5 inches

For SI: 1 inch = 25.4 mm.

1808.8.5 Forming of concrete. Concrete foundations are permitted to be cast against the earth where, in the opinion of the *building official*, soil conditions do not require formwork. Where formwork is required, it shall be in accordance with ~~Chapter 6 of~~ ACI 318.

1808.8.6 Seismic requirements. See Section 1908 for additional requirements for foundations of structures assigned to *Seismic Design Category* C, D, E or F.

For structures assigned to *Seismic Design Category* D, E or F, provisions of ACI 318 for foundations resisting earthquake induced forces~~, Sections 21.12.1 through 21.12.4~~ shall apply where not in conflict with the provisions of Section 1808 through 1810.

Exceptions:

1. Detached one- and two-family dwellings of light-frame construction and two stories or less above *grade plane* are not required to comply with the provisions of ACI 318 for foundations resisting earthquake induced forces~~, Sections 21.12.1 through 21.12.4~~.
2. ~~Section 21.12.4.4(a) of~~ ACI 318 requirement for transverse reinforcement for foundations resisting earthquake induced forces at the top of piles, piers or caissons for at least five times the member cross-section dimension but not less than six feet below the bottom of the pile cap shall not apply.

1810.2.4.1 Seismic Design Categories D through F. For structures assigned to *Seismic Design Category* D, E or F, deep foundation elements on *Site Class* E or F sites, as determined in Section 1613.5.2, shall be designed and constructed to withstand maximum imposed curvatures from earthquake ground motions and structure response. Curvatures shall include free-field soil strains modified for soil-foundations-structure interaction coupled with foundation element deformations associated with earthquake loads imparted to the foundation by the structure.

Exception: Deep foundation elements that satisfy the following additional detailing requirements shall be deemed to comply with the curvature capacity requirements of this section.

1. Precast prestressed concrete piles detailed in accordance with Section 1810.3.8.3.3.
2. Cast-in-place deep foundation elements with a minimum longitudinal reinforcement ratio of 0.005 extending the full length of the element and with transverse reinforcement detailed in accordance with the requirements for special moment frame members subjected to bending and axial load per ~~Sections 21.6.4.2, 21.6.4.3 and 21.6.4.4 of~~ ACI 318 as required by Section 1810.3.9.4.2.2.

1810.3.2.1.2 ~~ACI 318 Equation (10-5).~~Minimum volumetric spiral ratio. Where this chapter requires ~~detailing of~~ transverse reinforcement in accordance with the requirements for special moment frame members subjected to bending and axial load per ACI 318 in concrete deep foundation elements ~~in accordance with Section 21.6.4.4 of ACI~~

~~318~~, ~~compliance with~~, the minimum volumetric spiral ratio shall be 0.12f'_c / f_{yt}. ~~Equation (10-5) of ACI 318 shall not be required.~~

1810.3.8.3.3 Seismic reinforcement in Seismic Design Categories D through F. For structures assigned to *Seismic Design Category* D, E or F in accordance with Section 1613, precast prestressed piles shall have transverse reinforcement in accordance with the following

1. The earthquake-resistant structures ~~R~~requirements in ACI 318~~, Chapter 21,~~ need not apply, unless specifically referenced.
2. Where the total pile length in the soil is 35 feet (10 668 mm) or less, the lateral transverse reinforcement in the ductile region shall occur through the length of the pile. Where the pile length exceeds 35 feet (10 668 mm), the ductile pile region shall be taken as the greater of 35 feet (10 668 mm) or the distance from the underside of the pile cap to the point of zero curvature plus three times the least pile dimension.
3. In the ductile region, the center-to-center spacing of the spirals or hoop reinforcement shall not exceed one-fifth of the least pile dimension, six times the diameter of the longitudinal strand or 8 inches (203 mm), whichever is smallest.
4. Circular spiral reinforcement shall be spliced by lapping one full turn and bending the end of each spiral to a 90-degree hook or by use of a mechanical or welded splice complying with the general (non-seismic) splicing requirements of ~~Section 12.14.3 of~~ ACI 318.
5. Where the transverse reinforcement consists of circular spirals, the volumetric ratio of spiral transverse reinforcement in the ductile region shall comply with the following:

$$P_s = 0.25(f'_c / f_{yh})(A_g / A_{ch} - 1.0)[0.5 + 1.4P/(f'_c A_g)] \quad \textbf{(Equation 18-6)}$$

but not less than:

$$P_s = 0.12(f'_c / f_{yh})[0.5 + 1.4P/(f'_c A_g)] \geq 0.12 f'_c / f_{yh} \quad \textbf{\textit{(Equation 18-7)}}$$

and need not exceed:

$$P_s = 0.021 \quad \textbf{(Equation 18-8)}$$

where:

A_g = Pile cross-sectional area, square inches (mm^2).
A_{ch} = Core area defined by spiral outside diameter, square inches (mm^2).
f'_c = Specified compressive strength of concrete, psi (MPa)
f_{yh} = Yield strength of spiral reinforcement ≤ 85,000 psi (586 MPa).
P = Axial load on pile, pounds (kN), as determined from Equations 16-5 and 16-7.
P_s = Volumetric ratio (vol. spiral/ vol. core).

This required amount of spiral reinforcement is permitted to be obtained by providing an inner and outer spiral.

6. Where transverse reinforcement consists of rectangular hoops and cross ties, the total cross-sectional area of lateral transverse reinforcement in the ductile region with spacing, s, and perpendicular dimension, h_c, shall conform to:

$$A_{sh} = 0.3s\, h_c (f'_c / f_{yh})(A_g / A_{ch} - 1.0)[0.5 + 1.4P/(f'c A_g)] \quad \textbf{(Equation 18-9)}$$

but not less than:

$$A_{sh} = 0.12s\, h_c (f'_c / f_{yh})[0.5 + 1.4P/(f'_c A_g)] \quad \textbf{(Equation 18-10)}$$

where:

f_{yh} = ≤70,000 psi (483 MPa).
h_c = Cross-sectional dimension of pile core measured center to center of hoop reinforcement, inch (mm).
s = Spacing of transverse reinforcement measured along length of pile, inch (mm).

A_{sh} = Cross-sectional area of tranverse reinforcement, square inches (mm2).
f'_c = Specified compressive strength of concrete, psi (MPa).

The hoops and cross ties shall be equivalent to deformed bars not less than No. 3 in size. Rectangular hoop ends shall terminate at a corner with seismic hooks. Outside of the length of the pile requiring transverse confinement reinforcing, the spiral or hoop reinforcing with a volumetric ratio not less than one-half of that required for transverse confinement reinforcing shall be provided.

1810.3.9.4.2.1 Site Classes A through D. For *Site Class* A, B, C or D sites, transverse ~~confinement~~ reinforcement shall be provided in the element in accordance with the requirements for special moment frame members subjected to bending and axial load per ~~Sections 21.6.4.2, 21.6.4.3 and 21.6.4.4 of~~ ACI 318 within three times the least element dimension of the bottom of the pile cap. ~~A~~ The minimum transverse spiral reinforcement ratio shall be 0.06f'_c / f_{yt}. ~~of not less than one-half of that required in Section 21.6.4.4(a) of ACI 318 shall be permitted.~~

1810.3.9.4.2.2 Site Classes E and F. For *Site Class* E or F sites, transverse ~~confinement~~ reinforcement shall be provided in the element in accordance with the requirements for special moment frame members subjected to bending and axial load per ~~Sections 21.6.4.2, 21.6.4.3 and 21.6.4.4 of~~ ACI 318 within seven times the least element dimension of the pile cap and within seven times the least element dimension of the interfaces of strata that are hard or stiff and strata that are liquefiable or are composed of soft- to medium-stiff clay.

1810.3.11.1 Seismic Design Categories C through F. For structures assigned to *Seismic Design Category* C, D, E or F in accordance with Section 1613, concrete deep foundation elements shall be connected to the pile cap by embedding the element reinforcement or field-placed dowels anchored in the element into the pile cap for a distance equal to their development length in accordance with ACI 318. It shall be permitted to connect precast prestressed piles to the pile cap by developing the element prestressing strands into the pile cap provided the connection is ductile. For deformed bars, the development length is the full development length for compression, or tension in the case of uplift, without reduction in development length for excess reinforcement in flexural members permitted by ~~in accordance with Section 12.2.5 of~~ ACI 318. Alternative measures for laterally confining concrete and maintaining toughness and ductile-like behavior at the top of the element shall be permitted provided the design is such that any hinging occurs in the confined regions.

The minimum transverse steel ratio for confinement shall not be less than one-half of that required for columns.

For resistance to uplift forces, anchorage of steel pipes, tubes or H-piles to the pile cap shall be made by means other than concrete bond to the bare steel section. Concrete-filled steel pipes or tubes shall have reinforcement of not less than 0.01 times the cross-sectional area of the concrete fill developed into the cap and extending into the fill a length equal to two times the required cap embedment, but not less than the development length in tension of the reinforcement.

1901.4 Construction documents. The *construction documents* for structural concrete construction shall include:

1. The specified compressive strength of concrete at the stated ages or stages of construction for which each concrete element is designed.
2. The specified strength or grade of reinforcement.
3. The size and location of structural elements, reinforcement and anchors.
4. Provision for dimensional changes resulting from creep, shrinkage and temperature.
5. The magnitude and location of prestressing forces.
6. Anchorage length of reinforcement and location and length of lap splices.
7. Type and location of mechanical and welded splices of reinforcement.
8. Details and location of contraction or isolation joints specified for plain concrete.
9. Minimum concrete compressive strength at time of posttensioning.
10. Stressing sequence for posttensioning tendons.
11. For structures assigned to *Seismic Design Category* D, E or F, a statement if slab on grade is designed as a structural diaphragm ~~(see Section 21.12.3.4 of ACI 318)~~.

1904.1 Water-cementitious materials ratio. Where maximum water-cementitious materials ratios are specified in ACI 318, they shall be calculated in accordance with ACI 318~~, Section 4.1~~.

1904.2 Exposure categories and classes. Concrete shall be assigned to exposure classes in accordance with ACI 318~~, Section 4.2,~~ based on:

1. Exposure to freezing and thawing in a moist condition or deicer chemicals;
2. Exposure to sulfates in water or soil;
3. Exposure to water where the concrete is intended to have low permeability; and
4. Exposure to chlorides from deicing chemicals, salt, saltwater, brackish water, seawater or spray from these sources, where the concrete has steel reinforcement.

1904.3 Concrete properties. Concrete mixtures shall conform to the most restrictive maximum water-cementitious materials ratios, and minimum specified concrete compressive strength requirements of ACI 318~~, Section 4.3,~~ based on the exposure classes assigned in Section 1904.2.

> **Exception:** For occupancies and appurtenances thereto in Group R occupancies that are in buildings less than four stories above grade plane, normal-weight aggregate concrete is permitted to comply with the requirements of Table 1904.3 based on the weathering classification (freezing and thawing) determined from Figure 1904.3 in lieu of the requirements of ACI 318~~, Table 4.3.1~~.

…

1904.4.1 Air entrainment. Concrete exposed to freezing and thawing while moist shall be air entrained in accordance with ACI 318~~, Section 4.4.1~~.

1904.4.2 Deicing chemicals. For concrete exposed to freezing and thawing in the presence of moisture and deicing chemicals, the maximum weight of fly ash, other pozzolans, silica fume or slag that is included in the concrete shall not exceed the percentages of the total weight of cementitious materials permitted by ACI 318~~, Section 4.4.2~~.

1904.5 Alternative cementitious materials for sulfate exposure. Alternative combinations of cementitious materials for use in sulfate-resistant concrete to those listed in ACI 318~~, Table 4.3.1~~ shall be permitted in accordance with ACI 318~~, Section 4.5.1~~.

1905.2 Selection of concrete proportions. Concrete proportions shall be determined in accordance with the provisions of ACI 318~~, Section 5.2~~.

1905.3 Proportioning on the basis of field experience and/or trial mixtures. Concrete proportioning determined on the basis of field experience and/or trial mixtures shall be done in accordance with ACI 318~~, Section 5.3~~.

1905.4 Proportioning without field experience or trial mixtures. Concrete proportioning determined without field experience or trial mixtures shall be done in accordance with ACI 318~~, Section 5.4~~.

1905.5 Average strength reduction. As data become available during construction, it is permissible to reduce the amount by which the average compressive strength (f'_c) is required to exceed the specified value of f'_c in accordance with ACI 318~~, Section 5.5~~.

1905.6.2 Frequency of testing. The frequency of conducting strength tests of concrete and the minimum number of tests shall be as specified in ACI 318~~, Section 5.6.2~~.

> **Exception:** When the total volume of a given class of concrete is less than 50 cubic yards ($38m^3$), strength tests are not required when evidence of satisfactory strength is submitted to and approved by the building official.

1905.6.3 Strength test specimens. Specimens prepared for acceptance testing of concrete in accordance with Section 1905.6.2 and strength test acceptance criteria shall comply with the provisions of ACI 318~~, Section 5.6.3~~.

1905.6.4 Field-cured specimens. Where required by the building official to determine adequacy of curing and protection of concrete in the structure, specimens shall be prepared, cured, tested and test results evaluated for acceptance in accordance with ACI 318~~, Section 5.6.4~~.

1905.6.5 Low-strength test results. Where any strength test ~~(see ACI 318, Section 5.6.2.4)~~ falls below the specified value of f'_c, the provisions of ACI 318~~, Section 5.6.5~~, shall apply.

1905.7 Preparation of equipment and place of deposit. Prior to concrete being placed, the space to receive the concrete and the equipment used to deposit it shall comply with ACI 318~~, Section 5.7~~.

1905.8 Mixing. Mixing of concrete shall be performed in accordance with ACI 318~~, Section 5.8~~.

1905.9 Conveying. The method and equipment for conveying concrete to the place of deposit shall comply with ACI 318~~, Section 5.9~~.

1905.10 Depositing. The depositing of concrete shall comply with the provisions of ACI 318~~, Section 5.10~~.

1905.11 Curing. The length of time, temperature and moisture conditions for curing of concrete shall be in accordance with ACI 318~~, Section 5.11~~.

1905.12 Cold weather requirements. Concrete to be placed during freezing or near-freezing weather shall comply with the requirements of ACI 318~~, Section 5.12~~.

1905.13 Hot weather requirements. Concrete to be placed during hot weather shall comply with the requirements of ACI 318~~, Section 5.13~~.

1906.1 Formwork. The design, fabrication and erection of forms shall comply with ACI 318~~, Section 6.1~~.

1906.2 Removal of forms, shores and reshores. The removal of forms and shores, including from slabs and beams (except where cast on the ground), and the installation of reshores shall comply with ACI 318~~, Section 6.2~~.

1906.3 Conduits and pipes embedded in concrete. Conduits, pipes and sleeves of any material not harmful to concrete and within the limitations of ACI 318~~, Section 6.3~~, are permitted to be embedded in concrete with approval of the registered design professional.

1906.4 Construction joints. Construction joints, including their location, shall comply with the provisions of ACI 318~~, Section 6.4~~.

1907.1 Hooks. Standard hooks on reinforcing bars used in concrete construction shall comply with ACI 318~~, Section 7.1~~.

1907.2 Minimum bend diameters. Minimum reinforcement bend diameters utilized in concrete construction shall comply with ACI 318~~, Section 7.2~~.

1907.3 Bending. The bending of reinforcement shall comply with ACI 318~~, Section 7.3~~.

1907.4 Surface conditions of reinforcement. The surface conditions of reinforcement shall comply with the provisions of ACI 318~~, Section 7.4~~.

1907.5 Placing reinforcement. The placement of reinforcement, including tolerances on depth and cover, shall comply with the provisions of ACI 318~~, Section 7.5~~. Reinforcement shall be accurately placed and adequately supported before concrete is placed.

1907.6 Spacing limits for reinforcement. The clear distance between reinforcing bars, bundled bars, tendons and ducts shall comply with ACI 318~~, Section 7.6~~.

1907.7.1 Cast-in-place concrete (nonprestressed). Minimum specified concrete cover shall be provided for reinforcement in nonprestressed, cast-in-place concrete construction in accordance with ACI 318~~, Section 7.7.1~~.

1907.7.2 Cast-in-place concrete (prestressed). The minimum specified concrete cover for prestressed and nonprestressed reinforcement, ducts and end fittings in cast-in-place prestressed concrete shall comply with ACI 318~~, Section 7.7.2~~.

1907.7.3 Precast concrete (manufactured under plant control conditions). The minimum specified concrete cover for prestressed and nonprestressed reinforcement, ducts and end fittings in precast concrete manufactured under plant control conditions shall comply with ACI 318~~, Section 7.7.3~~.

1907.7.4 Bundled bars. The minimum specified concrete cover for bundled bars shall comply with ACI 318~~, Section 7.7.4~~.

1907.7.5 Headed shear stud reinforcement. For headed shear stud reinforcement, the minimum specified concrete cover shall comply with ACI 318~~, Section 7.7.5~~.

1907.7.6 Corrosive environments. In corrosive environments or other severe exposure conditions, prestressed and nonprestressed reinforcement shall be provided with additional protection in accordance with ACI 318~~, Section 7.7.6~~.

1907.8 Special reinforcement details for columns. Offset bent longitudinal bars in columns and load transfer in structural steel cores of composite compression members shall comply with the provisions of ACI 318~~, Section 7.8~~.

1907.9 Connections. Connections between concrete framing members shall comply with the provisions of ACI 318~~, Section 7.9~~.

1907.10 Lateral reinforcement for compression members. Lateral reinforcement for concrete compression members shall comply with the provisions of ACI 318~~, Section 7.10~~.

1907.11 Lateral reinforcement for flexural members. Lateral reinforcement for compression reinforcement in concrete flexural members shall comply with the provisions of ACI 318~~, Section 7.11~~.

1907.12 Shrinkage and temperature reinforcement. Reinforcement for shrinkage and temperature stresses in concrete members shall comply with the provisions of ACI 318~~, Section 7.12~~.

1907.13 Requirements for structural integrity. The detailing of reinforcement and connections between concrete members shall comply with the provisions of ACI 318~~, Section 7.13,~~ to improve structural integrity.

1909.1 Scope. The design and construction of structural plain concrete, both cast-in-place and precast, shall comply with the minimum requirements of Section 1909 and <u>the structural plain concrete requirements of ACI 318</u>~~, Chapter 22,~~ as modified in Section 1908.

1909.3 Joints. Contraction or isolation joints shall be provided to divide structural plain concrete members into flexurally discontinuous elements in accordance with ACI 318~~, Section 22.3~~.

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

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

1909.5 Precast members. The design, fabrication, transportation and erection of precast, structural plain concrete elements shall be in accordance with ACI 318~~, Section 22.9~~.

1909.6 Walls. In addition to the requirements of this section, structural plain concrete walls shall comply with the applicable requirements of ACI 318~~, Chapter 22~~.

PART II – IRC BUILDING/ENERGY

Revise as follows:

R402.2 Concrete. Concrete shall have a minimum specified compressive strength of f'_c, as shown in Table R402.2. Concrete subject to moderate of severe weathering as indicated in Table R301.2(1) shall be air entrained as specified in Table R402.2. The maximum weight of fly ash, other pozzolans, silica fume, slag or blended cements that is included in concrete mixtures for garage floor slabs and for exterior porches, carport slabs and steps that will be exposed to deicing chemicals shall not exceed the percentages of the total weight of cementitious materials specified in <u>the durability requirements</u> ~~Section 4.2.3~~ of ACI 318. Materials used to produce concrete and testing thereof shall comply with the applicable standards listed in <u>the material requirements</u> ~~Chapter 3~~ of ACI 318 or ACI 332.

Reason:
PART I- ACI is in the process of completely reorganizing ACI 318, *Building Code Requirements for Structural Concrete*. This process was formally initiated in the spring of 2008 and was scheduled to be completed by January 1, 2014 so that it would be available for reference by the 2015 IBC. This 2015 IBC schedule was assumed to be the same as traditional ICC meeting schedules.

The reorganized ACI 318 will be significantly different in structure than 318-08. The code will include several new chapters that are based on member design. The 318-08 chapters will be significantly reworked to support the member chapters. Some provisions have been divided sometimes into several chapters and at other provisions combined.

The new schedules released in February 2009 by the ICC require that reorganized ACI 318 be completed by January 3, 2012 for changes to the IBC (Group A) that require textual changes to the body of the IBC. ACI committee 318 will likely not be done with the revisions by that time and thus not have a revised standard to submit for the 2015 IBC if it remains in Group A. ICC's CP 28-05 states that, if no textual changes are required and that the change to the code is in reference only, the revision may be considered by the administrative code committee prior to the Final Action Hearing in Group B.

In this code change proposal, ACI proposes to remove all references to specific sections in ACI 318 from the IBC, but not change the technical intent of any provision. This removal would allow ACI to submit an administrative change in Group B and give ACI Committee 318 one more year to complete their task of reorganization.

In some cases, the referenced section numbers that are removed are replaced with language from ACI 318 (usually the section heading) that will allow the user to easily locate the appropriate section. In some cases, the referenced information in ACI 318 is transcribed into the IBC. The use of words in place of section numbers may require some editorial revisions in the 2012/2013 ICC cycle, if the headings are changed through the reorganization.

The following explanations are given to for the revised Sections stated above:

Table 1704.3 and Table 1704.4	Specific section references are removed.
Table 1704.4	Specific section references are removed.
1704.4.1	Reference to Chapter 3 is removed. "Materials" is the heading of Chapter 3 in ACI 318; therefore, no additional replacement language is needed. Section 3.5.2 is removed and replaced with language that would identify the source information in ACI 318.
1708.2	Replaced the referenced section with language that would identify the source information in ACI 318. The last sentence repeats the welding requirements of IBC 1704.4.1, therefore, it is being removed.
1808.8.2	Reference to Section 7.7.4 is removed and sentence is reworded to preserve the intent of the provision.
Table 1808.8.2	Replaced the referenced section with language that would identify the source information in ACI 318.
1808.8.5	Reference to Chapter 6 is removed. Formwork is the heading of Chapter 6 in ACI 318; therefore, no additional replacement language is needed.
1808.8.6	Replaced the referenced section with language that would identify the source information in ACI 318. Also, technical information from Section 21.14.4.4(a) of ACI 318 is transcribed to Exception 2 in order to remove the section reference.
1810.2.4.1	Replaced the referenced section with language that would identify the source information in ACI 318.
1810.3.2.1.2	Technical information from Section 21.6.4.4 of ACI 318 is transcribed to 1810.3.2.1.2 in order to remove the section reference.
1810.3.8.3.3	Reference to Chapter 21 is removed. Earthquake-Resistant Structures is the heading of Chapter 21 in ACI 318; therefore, no additional replacement language is needed. Section 12.14.3 is removed and replaced with language that would identify the source information in ACI 318.
1810.3.9.4.2.1	Replaced the referenced section with language that would identify the source information in ACI 318. Also, technical information from Section 21.14.4.4(a) of ACI 318 is transcribed to 1810.3.9.4.2.1 in order to remove the section reference.
1810.3.9.4.2.2	Replaced the referenced section with language that would identify the source information in ACI 318.
1810.3.11.1	Reference to section is removed. The sentence is self-explanatory without adding the section reference.
1901.4	Section 21.12.3.4 of ACI 318 states the same information that is given in Item 11. No need for additional referencing.
Section 1904	References to sections are removed. This chapter repeats headings and technical content directly from ACI 318. No additional language is necessary in order to remove the section reference.
Section 1905	References to sections are removed. This chapter repeats headings and technical content directly from ACI 318. No additional language is necessary in order to remove the section reference.
Section 1906	References to sections are removed. This chapter repeats headings and technical content directly from ACI 318. No additional language is necessary in order to remove the section reference.
Section 1907	References to sections are removed. This chapter repeats headings and technical content directly from ACI 318. No additional language is necessary in order to remove the section reference.
1909.1	Replaced the referenced chapter with language that would identify the source information in ACI 318.
1909.3, 1909.4, 1909.5 & 1909.6	References to sections are removed. These sections repeat headings and technical content directly from ACI 318. No additional language is necessary in order to remove the section reference.

PART II- ACI is in the process of completely reorganizing ACI 318, *Building Code Requirements for Structural Concrete*. This process was formally initiated in the spring of 2008 and was scheduled to be completed by January 1, 2014 so that it would be available for reference by the 2015 ICC Codes. This 2015 ICC schedule was assumed to be the same as traditional ICC meeting schedules.

The reorganized ACI 318 will be significantly different in structure than 318-08. The code will include several new chapters that are based on member design. The 318-08 chapters will be significantly reworked to support the member chapters. Some provisions have been divided sometimes into several chapters and at other provisions combined.

The new schedules released in February 2009 by the ICC require that reorganized ACI 318 be completed by January 3, 2012 for changes to the IBC (Group A) that require textual changes to the body of the IBC. ACI committee 318 will likely not be done with the revisions by that time and thus not have a revised standard to submit for the 2015 IBC if it remains in Group A. ICC's CP 28-05 states that, if no textual changes are required and that the change to the code is in reference only, the revision may be considered by the administrative code committee prior to the Final Action Hearing in Group B.

In this code change proposal, ACI proposes to remove all references to specific sections in ACI 318 from the IBC and IRC, but not change the technical intent of any provision. This removal would allow ACI to submit an Administrative Change in Group B and give ACI Committee 318 one more year to complete their task of reorganization.

In some cases, the referenced section numbers that are removed are replaced with language from ACI 318 (usually the section heading) that will allow the user to easily locate the appropriate section. In some cases, the referenced information in ACI 318 is transcribed into the ICC code. The use of words in place of section numbers may require some editorial revisions in the 2012/2013 ICC cycle, if the headings are changed through the reorganization.

The following explanations are given to for the revised Section stated above:

R402.2	Reference to Section 4.2.3 is removed and replaced with the chapter heading, "Durability Requirements," to aid the user in locating the source information in ACI 318. Note reference to Section 4.2.3 is incorrect in the current IRC. ACI 318 now uses a classification system from which the minimum cementitious material is determined. A more general reference to the chapter is more appropriate. Reference to Chapter 3 is removed and replaced with the chapter heading, "Materials ," to aid the user in locating the source information in ACI 318.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: SENECAL-S4-1901.4

S163–09/10

1908.1-1908.1.10, 1912.1

Proponent: Matthew Senecal, PE, American Concrete Institute, representing American Concrete Institute

Revise as follows:

1908.1 General. ~~The text of~~ ACI 318 shall be modified as indicated in Sections 1908.1.1 through 1908.1.10.

1908.1.1 ~~**ACI 318, Section 2.2**~~ **Definitions.** ~~Modify existing definitions and add the following definitions to ACI 318, Section 2.2.~~The following definitions either modify or are in addition to the definitions in ACI 318:

DESIGN DISPLACEMENT. Total lateral displacement expected for the design-basis earthquake*, as specified by Section 12.8.6 of ASCE 7*.

DETAILED PLAIN CONCRETE STRUCTURAL WALL. ~~*A wall complying with the requirements of Chapter 22, including 22.6.7.*~~ *A structural plain concrete wall in accordance with ACI 318, including the requirements of Section 1908.1.7 of this code.*

ORDINARY PRECAST STRUCTURAL WALL. ~~*A precast wall complying with the requirements of Chapters 1 through 18.*~~ *A precast wall complying with the requirements ACI 318, except the requirements for earthquake-resistant structures shall not apply.*

ORDINARY REINFORCED CONCRETE STRUCTURAL WALL. ~~A *cast-in-place* wall complying with the requirements of Chapters 1 through 18.~~ *A cast-in-place wall complying with the requirements ACI 318, except the requirements for earthquake-resistant structures shall not apply.*

ORDINARY STRUCTURAL PLAIN CONCRETE WALL. ~~A wall complying with the requirements of Chapter 22, *excluding 22.6.7.*~~*A structural plain concrete wall in accordance with ACI 318, excluding Section 1908.1.7 of this code.*

SPECIAL STRUCTURAL WALL. A cast-in-place or precast wall complying with the requirements for earthquake-resistant structures of ACI 318 ~~21.1.3 through 21.1.7, 21.9 and 21.10~~, as applicable, in addition to the requirements for ordinary reinforced concrete structural walls *or ordinary precast structural walls, as applicable. Where ASCE 7 refers to a "special reinforced concrete structural wall," it shall be deemed to mean a "special structural wall."*

WALL PIER. *A wall segment with a horizontal length-to-thickness ratio of at least 2.5, but not exceeding 6, whose clear height is at least two times its horizontal length.*

1908.1.2 ~~**ACI 318, Section 21.1.1.**~~ **Structural systems.** ~~Modify ACI 318 Sections 21.1.1.3 and 21.1.1.7 to read as follows:~~ Structural systems designated as part of the seismic-force-resisting system shall be restricted to those permitted by ASCE 7. Structural plain concrete is prohibited in structures assigned to Seismic Design Category C, D, E, or F, except as permitted by Section 1908.1.8 of this code.

~~*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.*~~

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

1908.1.3 ~~**ACI 318, Section 21.4.**~~ **Intermediate precast structural walls.** ~~Modify ACI 318, Section 21.4, by renumbering Section 21.4.3 to become 21.4.4 and adding new Sections 21.4.3, 21.4.5 and 21.4.6 to read as follows:~~ Intermediate precast structural walls shall satisfy the requirements for earthquake-resistant structures of ACI 318 and the following:

~~21.4.3~~ 1. Connections that are designed to yield shall be capable of maintaining 80 percent of their design strength at the deformation induced by the design displacement or shall use Type 2 mechanical splices.

~~21.4.4 – Elements of the connection that are not designed to yield shall develop at least 1.5 S_y.~~

~~21.4.6~~ 2. Wall segments with a horizontal length-to-thickness ratio less than 2.5 shall be designed as columns.

~~21.4.5~~ 3. Wall piers not designed as part of a moment frame shall have transverse reinforcement designed to resist the shear forces required to be resisted by intermediate moment frames ~~determined from 21.3.3~~. Spacing of transverse reinforcement shall not exceed 8 inches (203 mm). Transverse reinforcement shall be extended beyond the pier clear height for at least 12 inches (305 mm).

Exceptions:

1. *Wall piers that satisfy ~~21.13~~ the requirements of ACI 318 for members not designated as part of the seismic-force-resisting system.*
2. *Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers and such segments have a total stiffness of at least six times the sum of the stiffnesses of all the wall piers.*

1908.1.4 ~~**ACI 318, Section 21.9.**~~ **Special structural walls and coupling beams.** ~~Modify ACI 318, Section 21.9, by adding new Section 21.9.10 to read as follows:~~ Special structural walls, coupling beams, wall piers and wall segments shall satisfy the requirements for earthquake-resistant structures of ACI 318 and the following:

~~*21.9.10 – Wall piers and wall segments.*~~

~~21.9.10.1 –~~ 1. Wall piers not designed as a part of a special moment frame shall have transverse reinforcement designed to satisfy the requirements in Item 2. ~~21.9.10.2.~~

Exceptions:

1. *Wall piers that satisfy ~~21.13~~ the requirements of ACI 318 for members not designated as part of the seismic-force-resisting system.*
2. *Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers and such segments have a total stiffness of at least six times the sum of the stiffnesses of all the wall piers.*

~~21.9.10.2 –~~ 2. Transverse reinforcement with seismic hooks at both ends shall be designed to resist the shear forces <u>required to be resisted by special moment frames subjected to bending and axial load in accordance with ACI 318</u> ~~determined from 21.6.5.1~~. Spacing of transverse reinforcement shall not exceed 6 inches (152 mm). Transverse reinforcement shall be extended beyond the pier clear height for at least 12 inches (305 mm).

~~21.9.10.3 –~~ 3. Wall segments with a horizontal length-to-thickness ratio less than 2.5 shall be designed as columns.

1908.1.5 ~~ACI 318, Section 21.10.~~ <u>Special precast structural walls.</u> ~~Modify ACI 318, Section 21.10.2, to read as follows:~~ <u>Special structural walls constructed using precast concrete shall satisfy the requirements for earthquake-resistant structures of ACI 318. In addition, connections that are designed to yield shall be capable of maintaining 80 percent of their design strength at the deformation induced by the design displacement or shall use Type 2 mechanical splices.</u>

~~21.10.2 – Special structural walls constructed using precast concrete shall satisfy all the requirements of 21.9 for cast-in-place special structural walls in addition to Sections 21.4.2 *through* 21.4.4.~~

1908.1.6 ~~ACI 318, Section 21.12.1.1.~~ <u>Foundations resisting earthquake-induced forces</u>. ~~Modify ACI 318, Section 21.12.1.1, to read as follows:~~ <u>In structures assigned to Seismic Design Category D, E, or F, foundations resisting earthquake-induced forces or transferring earthquake-induced forces between a structure and ground shall satisfy the requirements for earthquake-resistant structures of ACI 318, unless modified by Chapter 18 of this code.</u>

~~21.12.1.1 – Foundations resisting earthquake-induced forces or transferring earthquake-induced forces between a structure and ground shall comply with the requirements of Section 21.12 and other applicable provisions of ACI 318 *unless modified by Chapter 18 of the International Building Code*.~~

1908.1.7 ~~ACI 318, Section 22.6.~~ <u>Detailed plain concrete structural walls.</u> ~~Modify ACI 318, Section 22.6, by adding new Section 22.6.7 to read as follows:~~ <u>Detailed plain concrete structural walls shall satisfy the requirements for an ordinary structural concrete wall in ACI 318 and the following:</u>

~~22.6.7 – Detailed plain concrete structural walls.~~

~~22.6.7.1 – Detailed plain concrete structural walls are walls conforming to the requirements of ordinary structural plain concrete walls and 22.6.7.2.~~

~~22.6.7.2 – Reinforcement shall be provided as follows:~~

~~(a)~~ <u>1.</u> Vertical reinforcement of at least 0.20 square inch (129 mm^2) in cross-sectional area shall be provided continuously from support to support at each corner, at each side of each opening and at the ends of walls. The continuous vertical bar required beside an opening is permitted to substitute for one of the two No. 5 bars required by <u>Section 1909.6.3</u>~~22.6.6.5~~.

~~(b)~~ <u>2.</u> Horizontal reinforcement at least 0.20 square inch (129 mm^2) in cross-sectional area shall be provided:

1. Continuously at structurally connected roof and floor levels and at the top of walls;
2. At the bottom of load-bearing walls or in the top of foundations where doweled to the wall; and
3. At a maximum spacing of 120 inches (3048 mm).

Reinforcement at the top and bottom of openings, where used in determining the maximum spacing specified in Item 3 above, shall be continuous in the wall.

1908.1.8 ~~ACI 318, Section 22.10.~~ <u>Structural plain concrete in earthquake-resisting structures.</u> Delete <u>the requirements for structural plain concrete in earthquake-resisting structures in</u> ACI 318, ~~Section 22.10,~~ and ~~replace with~~ <u>satisfy</u> the following:

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

~~22.10.1 –~~ 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 Section 1909.6.3. ~~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 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. *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.*

1908.1.9 ~~ACI 318, Section D.3.3.~~ Anchoring to concrete. ~~Modify ACI 318, Sections D.3.3.4 and D.3.3.5 to read as follows:~~ In structures assigned to Seismic Design Category C, D, E, or F, anchors to concrete shall satisfy the earthquake-resisting requirements for anchorage in ACI 318 and Section 1908.1.10 of this code.

~~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 the anchor steel ductility or attachment ductility requirement of ACI 318 ~~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 the anchor steel ductility or attachment ductility requirement of ACI 318 ~~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.*~~

1908.1.10 ~~ACI 318, Section D.4.2.2~~. Anchor diameter and length limits. ~~Delete ACI 318, Section D.4.2.2, and replace with the following:~~ Delete the ACI 318, 2 inch diameter limit for computation of shear and tension breakout strength and 25 inch length limit for computation of tension breakout strength and replace with the following:

~~*D.4.2.2 — The concrete breakout strength requirements for anchors in tension shall be considered satisfied by the design procedure of D.5.2 provided Equation D-8 is not used for anchor embedments exceeding 25 inches. The concrete breakout strength requirements for anchors in shear with diameters not exceeding 2 inches shall be considered satisfied by the design procedure of D.6.2. For anchors in shear with diameters exceeding 2 inches, shear anchor reinforcement shall be provided in accordance with the procedures of D.6.2.9.*~~

1. Tension concrete breakout strength shall not be computed by the following equation where the anchor length exceeds 25 inches.

$$N_b = 16\lambda\sqrt{f'_c}\,h_{ef}^{\frac{5}{3}}$$

2. For anchors with diameters exceeding 2 inches, the anchor reinforcement shall be used to resist shear instead of the concrete breakout strength.

1912.1 Scope. The provisions of this section shall govern the strength design of anchors installed in concrete for purposes of transmitting structural loads from one connected element to the other. Headed bolts, headed studs and hooked (J- or L-) bolts cast in concrete and expansion anchors and undercut anchors installed in hardened concrete shall be designed in accordance with the anchoring to concrete requirements ~~Appendix D~~ of ACI 318 as modified by Sections 1908.1.9 and 1908.1.10~~, provided they are within the scope of Appendix D~~.

The strength design of anchors that are not within the scope of ~~Appendix D of~~ ACI 318, and as amended in Sections 1908.1.9 and 1908.1.10, shall be in accordance with an approved procedure.

Reason: ACI is in the process of completely reorganizing ACI 318, Building Code Requirements for Structural Concrete. This process was formally initiated in the spring of 2008 and was scheduled to be completed by January 1, 2014 so that it would be available for reference by the 2015 IBC. This 2015 IBC schedule was assumed to be the same as traditional ICC meeting schedules.

The reorganized ACI 318 will be significantly different in structure than 318-08. The code will include several new chapters that are based on member design. The 318-08 chapters will be significantly reworked to support the member chapters. Some provisions have been divided sometimes into several chapters and at other provisions combined.

The new schedules released in February 2009 by the ICC require that reorganized ACI 318 be completed by January 3, 2012 for changes to the IBC (Group A) that require textual changes to the body of the IBC. ACI committee 318 will likely not be done with the revisions by that time and thus not have a revised standard to submit for the 2015 IBC if it remains in Group A. ICC's CP 28-05 states that, if no textual changes are required and that the change to the code is in reference only, the revision may be considered by the administrative code committee prior to the Final Action Hearing in Group B.

In this code change proposal, ACI proposes to remove all references to specific sections in ACI 318 from the IBC, but not change the technical intent of any provision. This removal would allow ACI to submit an administrative change in Group B and give ACI Committee 318 one more year to complete their task of reorganization.

In some cases, the referenced section numbers that are removed are replaced with language from ACI 318 (usually the section heading) that will allow the user to easily locate the appropriate section. In some cases, the referenced information in ACI 318 is transcribed into the IBC. The use of words in place of section numbers may require some editorial revisions in the 2012/2013 ICC cycle, if the headings are changed through the reorganization.

The following explanations are given to for the revised Sections stated above:

1908.1	Section 1908 is reworked to state the exception to ACI 318; in place of providing a textual edit of ACI 318 provisions.
1908.1.1	Replaced the referenced section with language that would identify the source information in ACI 318.
1908.1.2	In the revised section 21.1.1.3, the only additional requirement is the restriction on the use of structural plain concrete in SDC C. This restriction has been reworded. The statement that Chapter 21 does not apply to SDC A is explicitly stated in the Section 21.1.7 of ACI 318 and therefore is not an exception and may be deleted. In the revised section 21.1.7, ASCE 7 is identified as the standard that defines seismic-force-resisting systems. This statement has been reworded to introduce the section. Ordinary precast structural walls are defined in the definitions section and can be deleted at this location. The addition of the word "structural" in item d was addressed in errata to ACI 318.
1908.1.3	The section has been reworked, see explanation in 1908.1. Replaced referenced sections with language that would identify the source information in ACI 318.
1908.1.4	The section has been reworked, see explanation in 1908.1. Replaced referenced sections with language that would identify the source information in ACI 318.
1908.1.5	The section has been reworked, see explanation in 1908.1. The requirement from 1908.1.3 was added here for clarity.
1908.1.6	The section has been reworked, see explanation in 1908.1. Replaced referenced sections with language that would identify the source information in ACI 318.
1908.1.7	The section has been reworked, see explanation in 1908.1. Reference to 22.6.6.5 of ACI 318 was replaced by a reference to 1909.6.3 in the IBC.
1908.1.8	The section has been reworked, see explanation in 1908.1. Reference to 22.6.6.5 of ACI 318 was replaced by a reference to 1909.6.3 in the IBC.
1908.1.9	The section has been reworked, see explanation in 1908.1.
1908.1.10	Technical information from ACI 318 is transcribed to the IBC in order to remove the section reference
1912.1	Replaced the referenced section with language that would identify the source information in ACI 318.

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

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

ICCFILENAME: SENECAL-S5-1908.1

S164–09/10
1908.1.2

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-liked 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.

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

ICCFILENAME: ROBINSON-S1-1908.1.2

S165–09/10
1908.1.3

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

Revise as follows:

1908.1.3 ACI 318, Section 21.4. Modify ACI 318, Section 21.4, by renumbering Section 21.4.3 to become 21.4.4 and adding new Sections 21.4.3, 21.4.5, ~~and~~ 21.4.6 and 21.4.7 to read as follows:

21.4.3 – Connections that are designed to yield shall be capable of maintaining 80 percent of their design strength at the deformation induced by the design displacement or shall use Type 2 mechanical splices.

21.4.4 – Elements of the connection that are not designed to yield shall develop at least 1.5 S_y.

21.4.5 - Wall piers in Seismic Design Category D, E or F shall comply with Section 1908.1.4 of the International Building Code.

~~21.4.5~~ 21.4.6– Wall piers not designed as part of a moment frame in buildings assigned to SDC C shall have transverse reinforcement designed to resist the shear forces determined from 21.3.3. Spacing of transverse reinforcement shall not exceed 8 inches (203 mm). Transverse reinforcement shall be extended beyond the pier clear height for at least 12 inches (305mm).

Exceptions:

1. *Wall piers that satisfy 21.13.*
2. *Wall piers along a wall line within a story where other shear wall segments provide lateral support to the wall piers and such segments have a total stiffness of at least six times the sum of the stiffnesses of all the wall piers.*

~~21.4.6~~ 21.4.7 **–***Wall segments with a horizontal length-to-thickness ratio less than 2.5 shall be designed as columns.*

Reason: The design provision for wall pier detailing was originally introduced by SEAOC in 1987 to legacy Uniform Building Code and was included in 1988 UBC through 1997 UBC. The wall pier detailing provision prescribed under Section 1908.1.4 was intended for high seismic zones equivalent to current SDC D, E or F. 1908.1.3 was added as a complement of wall pier detailing in SDC C (formerly seismic zones 2A and 2B under legacy model code.) ACI 318 Commentary R 21.1.1 emphasized "*it is essential that structures assigned to higher SDC's possess a higher degree of toughness*", and further encourages practitioners to use special structural wall system in regions of high seismic risk. ASCE 7 Table 12.2-1 permits intermediate precast structural wall system in SDC D, E or F. Current Section 1908.1.3 does not limit to just structures assigned to SDC C. The required shear strength under 21.3.3, referenced in current Sec. 21.4.5, is based on V_u under either nominal moment strength or two times the code prescribed earthquake force. The required shear strength in 21.6.5.1, referenced in Sec. 21.9.10.2 (IBC 1908.1.4), is based on the probable shear strength, V_e under the probable moment strength, M_{pr}. In addition, the spacing of required shear reinforcement is 8 inches on center under current 21.4.5 instead of 6 inches on center with seismic hooks at both ends under 21.9.10.2. Requirement of wall pier under 21.9.10.2 would enhance better ductility.

Current practice in commercial buildings constructed using precast panels wall system have large window and door openings and/or narrow wall piers. Wall panels varying up to three stories high with openings resembles wall frame which is not currently recognized under any of the defined seismic-force resisting systems other than consideration of structural wall system. Conformance to special structural wall system design and detailing of wall piers ensures minimum life safety performance in resisting earthquake forces for structures in SDC D, E or F. Proposed modification separates wall piers designed for structures assigned to SDC C from those assigned to 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.

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

ICCFILENAME: ROBINSON-S2-1908.1.3

S166–09/10
1908.1.8

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 71/2 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 <u>Group U occupancies</u> ~~detached one-and two-family dwellings three stories or less in height~~ constructed with stud-bearing walls, plain concrete footings without longitudinal reinforcement supporting walls are permitted.
2. <u>In structures assigned to Seismic Design Categories D, E and F,</u> ~~For~~ foundation systems consisting of a plain concrete footing and a plain concrete stemwall, a minimum of one <u>No. 4</u> 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.

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

ICCFILENAME: MAIEL-S5-1908.1.8

S167–09/10
1908.1.9

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 <u>D.3.3.1,</u> D3.3.4 and D3.3.5<u>, and add Section D.3.3.7</u> to read as follows:

<u>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.</u>

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.

<u>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:</u>

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

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.

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

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

S168–09/10
1908.1.9

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 <u>delete and replace D.3.3.6 and add D.3.3.7</u> 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 an side edge distance of $1^{3}/_{4}$ 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 $0.75\Phi V_n$.

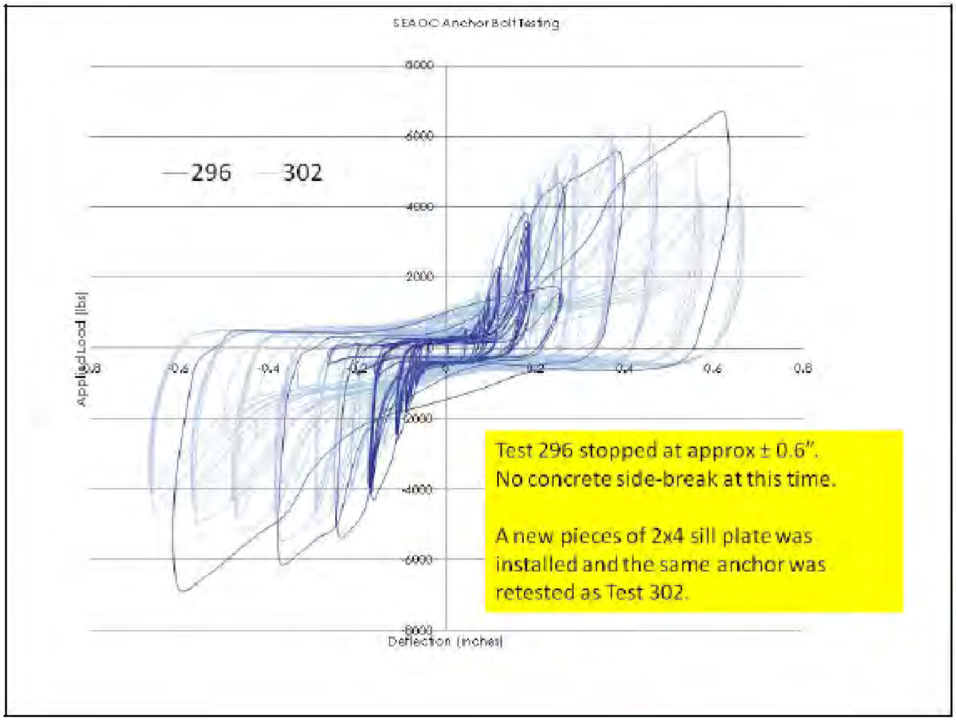


Figure A1 - Result of cycle test specimen 296 and 302

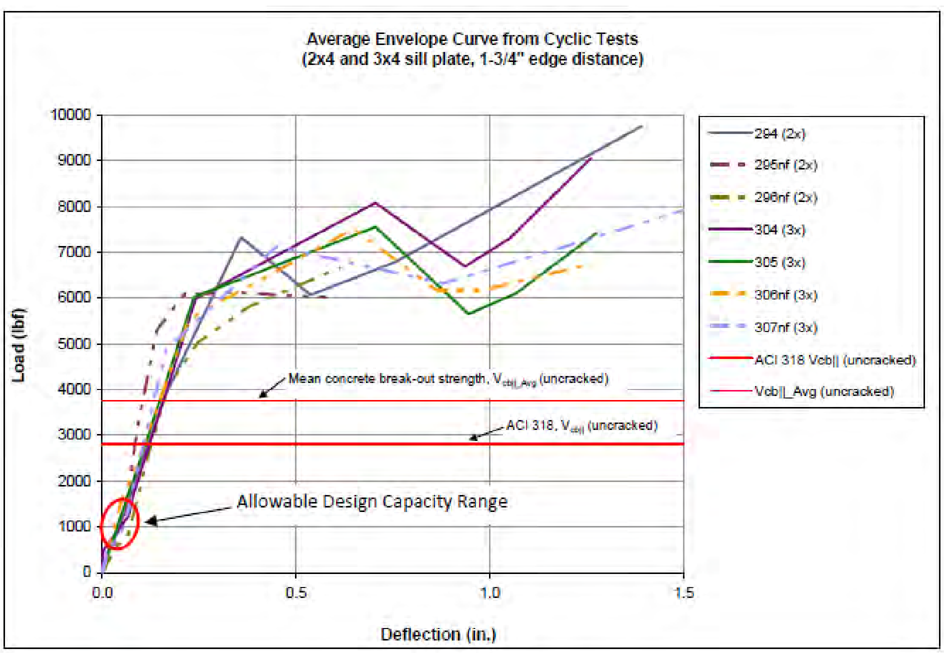


Fig. A2 – Average envelope curve of cyclic tests for concrete anchor of 2x4 and 3x4 sill plate

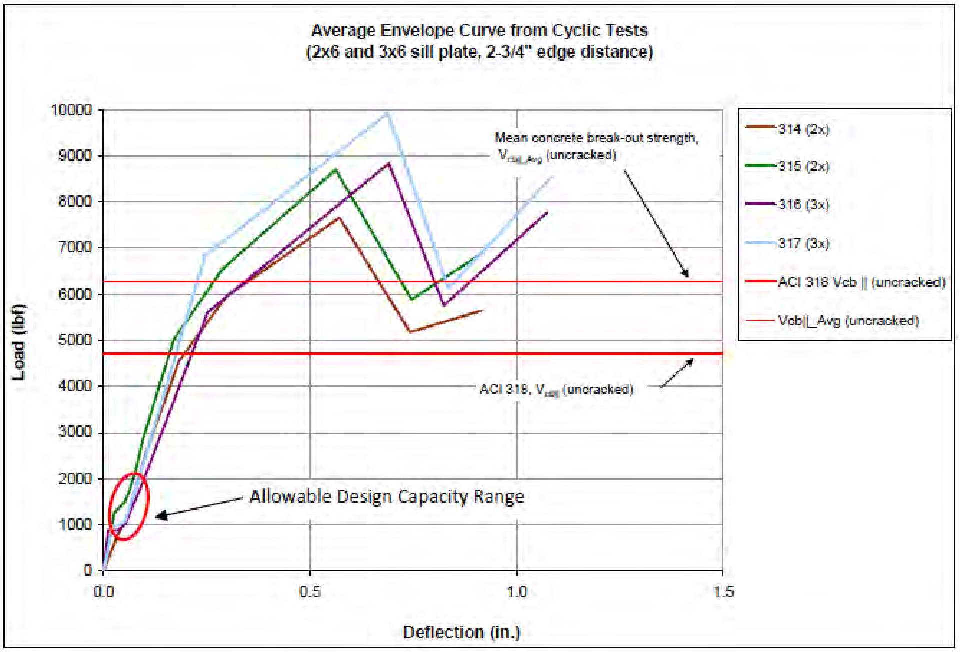


Fig. A3 – Average envelope curve of cyclic tests for concrete anchor of 2x6 and 3x6 sill plate

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.

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

ICCFILENAME: ROBINSON-S3-1908.1.9

S169–09/10

1910.2 (New)

Proponent: Daniel J. Walker, PE, Thomas Associates, Inc. representing the National Sunroom Association

Add new text as follows:

1910.2 Patio cover slab and footing requirements. In areas with a frost depth of zero, a *patio cover* shall be permitted to be supported on a concrete slab on grade without footings, provided the slab conforms to the provisions of Chapter 19 of this code, is not less than 3-1/2 inches (89 mm) thick and further provided that the columns do not support loads in excess of 750 pounds (3.36 kN) per column.

Reason: This language has long been included in Appendix I of the code. The requirements are specific to this section and therefore should be included here.

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

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

ICCFILENAME: Walker-S2-1910.2

S170-09/10
1911.1

Proponent: Richard L. Hess, Hess Engineering Inc., representing self

Revise as follows:

1911.1 Scope. The provisions of this section shall govern the allowable stress design of headed bolts and headed stud anchors cast in normal-weight concrete for purposes of transmitting structural loads from one connected element to the other. These provisions do not apply to anchors installed in hardened concrete or where load combinations include earthquake loads or effects <u>for structures other than buildings of light-frame construction that are no more than three stories above grade plane plus basement</u>. The bearing area of headed anchors shall be not less than one and one-half times the shank area. Where strength design is used, or where load combinations include earthquake loads or effects <u>for structures other than buildings of light-frame construction that are no more than three stories above grade plane plus basement</u>, the design strength of anchors shall be determined in accordance with Section 1912. Bolts shall conform to ASTM A 307 or an approved equivalent.

Reason: Experience of over fifty years in earthquake-prone California has demonstrated that light frame buildings of three stories plus basement or less have not experienced any failure of the anchor bolts used to secure shearwalls in these buildings. In addition to this well-documented field experience, testing of light frame walls with cyclical lateral forces has shown that these connections are ductile because of yielding of the light frame sill plates before any failure of the concrete can occur.

Cost Impact: This code change proposal will substantially decrease unnecessary direct design and construction costs as well as plan checking costs due to the confusion that has been created among practicing engineers and building officials by the requirement that light frame anchor bolts be designed in accordance with provisions of ACE 318 Appendix D, which was not based on evidence of its applicability to this type of construction or on testing of anchorage of light frame elements to concrete foundations.

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

FILENAME: HESS-S1-1911.1

S171–09/10
2101.2, 2101.2.7 (New), Chapter 35; IRC R606.1, R606.1.1, R606.12.1, R606.12.3.1, Chapter 44

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards and Phil Samblanet, The Masonry Society, representing The Masonry Society.

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC-STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

1. Add new text as follows:

<u>**2101.2.7 Direct design.** Masonry designed by the direct design method shall comply with the provisions of TMS 403.</u>

2. Revise as follows:

2101.2 Design methods. Masonry shall comply with the provisions of one of the following design methods in this chapter as well as the requirements of Sections 2101 through 2104. Masonry designed by the allowable stress design provisions of Section 2101.2.1, the strength design provisions of Section 2101.2.2<u>,</u> ~~or~~ the prestressed masonry provisions of Section 2101.2.3<u>, or the direct design requirements of Section 2101.2.7</u> shall comply with Section 2105.

3. Add standard to Chapter 35 as follows:

TMS
<u>403-09 Direct Design Handbook for Masonry Structures</u>

PART II – IRC BUILDING/ENERGY

1. Revise as follows:

R606.1 General. Masonry construction shall be designed and constructed in accordance with the provisions of this section, TMS 403, or in accordance with the provisions of TMS 402/ACI 530/ASCE 5.

R606.1.1 Professional registration not required. When the empirical design provisions of TMS 402/ACI 530/ASCE 5 Chapter 5, the provisions of TMS 403, or the provisions of this section are used to design masonry, project drawings, typical details and specifications are not required to bear the seal of the architect or engineer responsible for design, unless otherwise required by the state law of the *jurisdiction* having authority.

R606.12.1 General. Masonry structures and masonry elements shall comply with the requirements of Sections R606.12.2 through R606.12.4 based on the seismic design category established in Table R301.2(1). Masonry structures and masonry elements shall comply with the requirements of Section R606.12 and Figures R606.11(1), R606.11(2) and R606.11(3) or shall be designed in accordance with TMS 402/ACI 530/ASCE 5, or TMS 403.

R606.12.3.1 Design requirements. Masonry elements other than those covered by Section R606.12.2.2.2 shall be designed in accordance with the requirements of Chapter 1 and Sections 2.1 and 2.3 of ACI TMS 402/ACI 530/ASCE 5 and shall meet the minimum reinforcement requirements contained in Sections R606.12.3.2 and R606.12.3.2.1. Otherwise, masonry shall be designed in accordance with TMS 403.

> **Exception:** Masonry walls limited to one *story* in height and 9 feet (2743 mm) between lateral supports need not be designed provided they comply with the minimum reinforcement requirements of Sections R606.12.3.2 and R606.12.3.2.1.

2. Add standard to Chapter 44 as follows:

TMS
403-09 Direct Design Handbook for Masonry Structures

Reason: This modification proposes to introduce a simplified design method for single story, concrete masonry buildings based on a new, mandatory language reference standard TMS 403, *Direct Design Handbook for Masonry Structures*. The methodology used to develop the *Direct Design Handbook for Masonry Structures* is based upon the strength design provisions of the 2005 and 2008 editions of the TMS 402/ACI 530/ASCE 5, *Building Code Requirements for Masonry Structures* and the factored combinations of dead, roof live, wind, seismic, snow, and rain loads in accordance with the 2005 edition of ASCE 7, *Minimum Design Loads for Buildings and Other Structures*.

The genesis of this new design standard was conceived in response to concerns from the design community that structural design and loading requirements have become too complicated, particularly for relatively small, non-complicated structures. The direct design procedure is a table-based structural design method that permits the user, following a specific series of steps, to design and specify relatively simple, single-story concrete masonry bearing-wall structures. While simpler to implement, the direct design method does limit a designer's flexibility to only those configurations addressed by the standard and does introduce slightly more conservatism compared to conventional design approaches as a result of the conditions and assumptions inherent to the design approach.

A few of the key design limitations include:

- Snow – the ground snow load is limited to 60 lb/ft^2.
- Wind – the 3-second gust basic wind speed is limited to 150 mph; wind exposure category is limited to B or C; and site topography is limited such that K_{zt} = 1.0.
- Seismic – the mapped spectral acceleration for short and 1-second periods is limited to 3.0*g* and 1.25*g*, respectively and limited to site classes A, B, C, or D.
- Construction – wall are limited to single story, single wythe, 8-inch concrete masonry with a maximum height of 30 feet.
- Roof – while the design of roofs is not covered by the direct design method, roof diaphragms are required to be flexible, have rectangular dimensions with an aspect ratio not exceeding 4-to-1, with a maximum plan dimension of 200 feet.
- Reinforcement – all reinforcing bars are limited to No. 5 Grade 60.

While the *Direct Design Handbook for Masonry Structures* cannot be used to design a masonry structure of any configuration, it is intended to capture many of the simple loadbearing masonry structures commonly designed today. Those interested in reviewing a draft of the Direct Design Handbook for Masonry Structures are encouraged to download a working draft of the document at the following link: http://www.masonrysociety.org/html/resources/TMS-403/TMS403.htm

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, TMS 403-09, 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.

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: THOMPSON-SAMBLANET-S2-2101.2

S172–09/10
2101.3

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

Revise as follows:

2101.3 Construction documents. The *construction documents* shall show all of the items required by this code including the following:

1. Specified size, grade, type and location of reinforcement, anchors and wall ties.
2. Reinforcing bars to be welded and welding procedure.
3. Size and location of structural elements.
4. Provisions for dimensional changes resulting from elastic deformation, creep, shrinkage, temperature and moisture.
5. Loads used in the design of masonry.
6. Specified compressive strength of masonry at stated ages or stages of construction for which masonry is designed, except where specifically exempted by this code.
7. Details of anchorage of masonry to structural members, frames and other construction, including the type, size and location of connectors.
8. Size and <u>permitted</u> location of conduits, pipes and sleeves.
9. The minimum level of testing and inspection as defined in Chapter 17, or an itemized testing and inspection program that meets or exceeds the requirements of Chapter 17.

Reason: During the 2007/2008 hearings where the above language was approved (S175-07/08), a valid point was raised regarding Item 8 that requires the location of conduits, pipes, and sleeves be indicated. Typically, plans show the general or permitted location of such items combined with typical details (in this case, the mechanical/electrical/plumbing plans) for the construction or installation of such items, which are often located on different sheets of the plans. While minor, the above modification will continue to allow the flexibility afforded by this long-standing practice used when generating plans.

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

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

ICCFILENAME: THOMPSON-S2-2101.3

S173–09/10
2102.1

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards and Phil Samblanet, The Masonry Society.

1. Revise as follows:

2102.1 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.

AREA.

~~**Bedded**. The area of the surface of a masonry unit that is in contact with mortar in the plane of the joint.~~

Gross cross-sectional. The area delineated by the out-to-out specified dimensions of masonry in the plane under consideration.

Net cross-sectional. The area of masonry units, grout and mortar crossed by the plane under consideration based on out-to-out specified dimensions.

2. Delete without substitution:

~~**CONNECTOR.** A mechanical device for securing two or more pieces, parts or members together, including anchors, wall ties and fasteners.~~

~~**COVER.** Distance between surface of reinforcing bar and edge of member~~

3. Revise as follows:

DIMENSIONS.

~~**Actual.** The measured dimension of a masonry unit or element.~~

Nominal. The specified dimension plus an allowance for the joints with which the units are to be laid. Thickness is given first, followed by height and then length.

Specified. The dimensions specified for the manufacture or construction of masonry, masonry units, joints or any other component of a structure.

4. Delete without substitution:

~~**GROUTED MASONRY.**~~

~~**Grouted hollow-unit masonry.** That form of grouted masonry construction in which certain designated cells of hollow units are continuously filled with grout.~~

~~**Grouted multiwythe masonry.** That form of grouted masonry construction in which the space between the wythes is solidly or periodically filled with grout.~~

~~**HEIGHT, WALLS.** The vertical distance from the foundation wall or other immediate support of such wall to the top of the wall.~~

5. Revise as follows:

MASONRY UNIT. Brick, tile, stone, glass block or concrete ~~block~~ masonry unit conforming to the requirements specified in Section 2103.

~~**Clay.** A building unit larger in size than a brick, composed of burned clay, shale, fired clay or mixtures thereof.~~

~~**Concrete.** A building unit or block larger in size than 12 inches by 4 inches by 4 inches (305 mm by 102 mm by 102 mm) made of cement and suitable aggregates.~~

Hollow. A masonry unit whose net cross-sectional area in any plane parallel to the load-bearing surface is less than 75 percent of its gross cross-sectional area measured in the same plane.

Solid. A masonry unit whose net cross-sectional area in every plane parallel to the load-bearing surface is 75 percent or more of its gross cross-sectional area measured in the same plane.

6. Delete without substitution:

~~**SHELL.** The outer portion of a hollow masonry unit as placed in masonry.~~

~~**TIE, LATERAL.** Loop of reinforcing bar or wire enclosing longitudinal reinforcement.~~

~~**TILE.** A ceramic surface unit, usually relatively thin in relation to facial area, made from clay or a mixture of clay or other ceramic materials, called the body of the tile, having either a "glazed" or "unglazed" face and fired above red heat in the course of manufacture to a temperature sufficiently high enough to produce specific physical properties and characteristics.~~

~~**WEB.** An interior solid portion of a hollow masonry unit as placed in masonry.~~

(Definitions not shown are unchanged)

Reason: The terms "actual dimension", "grouted hollow-unit masonry", "grouted multiwythe masonry", "wall height", "shell", "lateral tie", and "web" are not used in Chapter 21. Several of these terms, however, (such as "actual dimension", "shell" and "wall height"), are used in other IBC chapters and the definitions in Section 2102 may be inappropriate for such use and as such should be deleted from the masonry chapter. If needed for other chapters, they should be defined for the specific use intended.

The definitions of "connector" and "cover" are inappropriate for their current use in Chapter 21. These terms are used in other masonry standards, but conflict with the usage as given in Chapter 21. For example, the definition of connector conflicts with the usage in IBC Sections 2113.11, 2113.15 and 2113.17. Likewise the definition of cover conflicts with IBC section 2113.18. As such, these definitions should be deleted to avoid additional confusion. If needed, more appropriate definitions could be considered, but the use of these terms in the noted sections is appropriate and understandable without definitions.

The definitions for "clay masonry unit", "concrete masonry units", and "tile" are inappropriate and should be deleted as they do not include all such units permitted by IBC Section 2103. For example, concrete masonry units smaller than the size noted in the definition are permitted by ASTM standards. More appropriate definitions of these terms are found in the referenced standards that are cited where these terms are used in the IBC.

The term "bedded area" is used only in Section 2105.3.2 and is defined in the standard cited in that section for the determination of compressive strength and is thus not needed in the IBC.

The term concrete block in the definition of masonry unit is being updated to the more appropriate term concrete masonry unit.

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

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

ICCFILENAME: THOMPSON-SAMBLANET-S4-2102 DEFINITIONS

S174–09/10

2102.1

Proponent: Jason Thompson, National Concrete Masonry Association, representing Masonry Alliance for Codes and Standards and Phil Samblanet, The Masonry Society.

Revise as follows:

2102.1 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.

BRICK.

Calcium silicate (sand lime brick). ~~A masonry unit made of sand and lime.~~ A pressed and subsequently autoclaved unit that consists of sand and lime, with or without the inclusion of other materials.

Clay or shale. ~~A masonry unit made of clay or shale, usually formed into a rectangular prism while in the plastic state and burned or fired in a kiln.~~ A solid or hollow masonry unit of clay or shale, usually formed into a rectangular prism, then burned or fired in a kiln; brick is a ceramic product.

Concrete. ~~A masonry unit having the approximate shape of a rectangular prism and composed of inert aggregate particles embedded in a hardened cementitious matrix.~~ A concrete masonry unit made from portland cement, water, and suitable aggregates, with or without the inclusion of other materials.

COLLAR JOINT. Vertical longitudinal ~~joint~~ space between wythes of masonry or between masonry wythe and backup construction that is permitted to be filled with mortar or grout~~.~~

DIMENSIONS.

Actual. The measured dimension of a masonry unit or element.

Nominal. The specified dimension plus an allowance for the joints with which the units are to be laid. Nominal dimensions are usually stated in whole numbers. Thickness is given first, followed by height and then length.

Specified. ~~The~~ Dimensions specified for the manufacture or construction of ~~masonry, masonry~~ a unit~~s~~, joint~~s~~ or ~~any other component of a structure~~ element.

MORTAR. A ~~plastic~~ mixture consisting of ~~*approved*~~ cementitious materials, fine aggregates<u>,</u> ~~and~~ water<u>, with or without admixtures, that is</u> used to ~~bond~~ <u>construct unit</u> masonry <u>assemblies</u> ~~or other structural units~~.

TIE, WALL. ~~A~~ <u>Metal</u> connector that connects wythes of masonry walls together.

WALL. A vertical element with a horizontal length-to-thickness ratio greater than three, used to enclose space.

Cavity wall. Awall built of masonry units or of concrete, or a combination of these materials, arranged to provide an airspace within the wall, and in which the inner and outer parts of the wall are tied together with metal ties.

Composite wall. A wall built of a combination of two or more masonry units bonded together, one forming the backup and the other forming the facing elements.

Dry-stacked, surface-bonded walls. A wall built of concrete masonry units where the units are stacked dry, without mortar on the bed or head joints, and where both sides of the wall are coated with a surface-bonding mortar.

Masonry-bonded hollow wall. A <u>multiwythe</u> wall built of masonry units ~~so~~ arranged ~~as~~ to provide an air space <u>between the wythes</u> ~~within the wall, and in which the facing and backing of the wall are~~ <u>and with the wythes</u> bonded together with masonry units.

Parapet wall. The part of any wall entirely above the roof line.

(Definitions not shown are unchanged)

Reason: The proposed modifications make these definitions consistent with those in ASTM Standards and the TMS 402 and TMS 602 standards and with their use in the IBC.

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

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

ICCFILENAME: THOMPSON-SAMBLANET-S5-2102 DEFINITIONS

S175–09/10
2103.5 (New), Chapter 35

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

1. Add new text as follows:

<u>**2103.5 Architectural cast stone.** Architectural cast stone shall conform to ASTM C1364.</u>

(Renumber subsequent sections accordingly)

2. Add standard to Chapter 35 as follows:

ASTM
<u>C1364-07</u> <u>Standard Specification for Architectural Cast Stone</u>

Reason: Architectural cast stone products have been used successfully in construction for decades. In 1997, ASTM published ASTM C1364, a specification covering the minimum physical requirements, sampling, testing, and visual inspection of the manufactured cast stone intended to replicate natural stone. To ensure that these products being specified and installed meet minimum established requirements for compressive strength, freeze thaw durability, aesthetics, shrinkage, etc., this modification proposes to introduce a reference to ASTM C1364 into the IBC to help ensure consistent and safe use of these products.

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

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM C1364-07, 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.

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

ICCFILENAME: THOMPSON-S1-2103.5

S176–09/10
2107.1, 2107.2

Proponent: Tom Young, PE, Northwest Concrete Masonry Association, representing Masonry Alliance for Codes and Standards

Revise as follows:

2107.1 General. The design of masonry structures using allowable stress design shall comply with Section 2106, Section 2107.2 and the requirements of Chapters 1 and 2 of TMS 402/ACI 530/ASCE 5 except as modified by Sections ~~2107.2~~ 2107.3 through 2107.5.

2107.2 ~~TMS 402/ACI 530/ASCE 5, Section 2.1.2,~~ Load combinations. ~~Delete Section 2.1.2.1.~~ Structures and portions thereof shall be designed to resist the most critical effects resulting from the load combinations of Section 1605.3. When using the alternative basic load combinations of Section 1605.3.2 that include wind or seismic loads, allowable stresses are permitted to be increased by one-third.

Reason: The IBC provides load combinations in Chapter 16 therefore there is no need to delete section 2.1.2.1 of TMS 402/ACI 530/ASCE 5. This code provision causes confusion for designers and is not necessary.

The additional language directs the code user to the allowable stress design load combinations in Chapter 16 and clarifies the permissible one-third increase in allowable stresses when using the alternative load combinations.

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

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

ICCFILENAME: YOUNG-S1-2107.1

S177–09/10
2107.3, 2107.3.1 (New)

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

Revise as follows:

2107.3 TMS 402/ACI 530/ASCE 5, Section 2.1.9.7.1.1, lap splices. ~~Modify~~ In lieu of Section 2.1.9.7.1.1, it shall be permitted to design lap splices in accordance with Section 2107.3.1 ~~as follows~~:

~~2.1.9.7.1.1~~ 2107.3.1 Lap splices, The minimum length of lap splices for reinforcing bars in tension or compression, l_d, shall be

$$l_d = 0.002 d_b f_s \qquad \textbf{(Equation 21-1)}$$

For SI: $l_d = 0.29 d_b f_s$

but not less than 12 inches (305 mm). In no case shall the length of the lapped splice be less than 40 bar diameters.

where:

d_b = Diameter of reinforcement, inches (mm).

f_s = Computed stress in reinforcement due to design loads, psi (MPa).

In regions of moment where the design tensile stresses in the reinforcement are greater than 80 percent of the allowable steel tension stress, F_s, the lap length of splices shall be increased not less than 50 percent of the minimum required length. Other equivalent means of stress transfer to accomplish the same 50 percent increase shall be permitted. Where epoxy coated bars are used, lap length shall be increased by 50 percent.

Reason: The masonry industry testing of lap splices has progressed. Some testing is still on-going. In the meantime, engineers in some portions of the country would prefer to use the provisions of the TMS 402 document. This code change proposal removes the complete prohibition of section 2.9.1.7.1.1, and replaces it with two alternative design approaches. This dual path is already used in other portions of the IBC.

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

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

ICCFILENAME: HUSTON-S4-2107.3

S178–09/10
2108.3

Proponent: Matthew Senecal, PE, representing American Concrete Institute

Revise as follows:

2108.3 TMS 402/ACI 530/ASCE 5, Section 3.3.3.4, splices. Modify items (b) and (c) of Section 3.3.3.4 as follows:

3.3.3.4 (b). A welded splice shall have the bars butted and welded to develop at least 125 percent of the yield strength, f_y, of the bar in tension or compression, as required. Welded splices shall be of ASTM A 706 steel reinforcement. Welded splices shall not be permitted in plastic hinge zones of intermediate or special reinforced walls or special moment frames of masonry.

3.3.3.4 (c). Mechanical splices shall be classified as Type 1 or 2 according to ~~Section 21.2.6.1~~ <u>in accordance with the requirements for mechanical splices in special moment frames and special structural walls</u> of ACI 318. Type 1 mechanical splices shall not be used within a plastic hinge zone or within a beam-column joint of intermediate or special reinforced masonry shear walls or special moment frames. Type 2 mechanical splices are permitted in any location within a member.

Reason: ACI is in the process of completely reorganizing ACI 318, *Building Code Requirements for Structural Concrete*. This process was formally initiated in the spring of 2008 and was scheduled to be completed by January 1, 2014 so that it would be available for reference by the 2015 IBC. This 2015 IBC schedule was assumed to be the same as traditional ICC meeting schedules.

The reorganized ACI 318 will be significantly different in structure than 318-08. The code will include several new chapters that are based on member design. The 318-08 chapters will be significantly reworked to support the member chapters. Some provisions have been divided sometimes into several chapters and at other provisions combined.

The new schedules released in February 2009 by the ICC require that reorganized ACI 318 be completed by January 3, 2012 for changes to the IBC (Group A) that require textual changes to the body of the IBC. ACI committee 318 will likely not be done with the revisions by that time and thus not have a revised standard to submit for the 2015 IBC if it remains in Group A. ICC's CP 28-05 states that, if no textual changes are required and that the change to the code is in reference only, the revision may be considered by the administrative code committee prior to the Final Action Hearing in Group B.

In this code change proposal, ACI proposes to remove all references to specific sections in ACI 318 from the IBC, but not change the technical intent of any provision. This removal would allow ACI to submit an administrative change in Group B and give ACI Committee 318 one more year to complete their task of reorganization.

In some cases, the referenced section numbers that are removed are replaced with language from ACI 318 (usually the section heading) that will allow the user to easily locate the appropriate section. In some cases, the referenced information in ACI 318 is transcribed into the IBC. The use of words in place of section numbers may require some editorial revisions in the 2012/2013 ICC cycle, if the headings are changed through the reorganization.

The following explanations are given to for the revised Sections stated above:

2108.3	Replaced the referenced section with language that would identify the source information in ACI 318. Note that the ACI 318 reference section number is not correct since it should be Section 21.1.6.

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

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

ICCFILENAME: SENECAL-S7-2108.3

S179–09/10
2109.1.1, 2308.2, 2308.2.1

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. <u>Section 5.1.2.2 of TMS 402/ACI 530/ASCE 5 shall be modified as follows:</u>

> <u>**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.</u>

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 <u>130</u> ~~100~~ miles per hour (mph) (~~44~~ <u>58</u> m/s) (3-second gust).

   Exception: Wind speeds shall not exceed <u>140</u> ~~110~~ mph (~~48.4~~ <u>63</u> 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 ~~100~~ 130 mph (3-second gust). Where the basic wind speed exceeds 130 ~~100~~ 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&PAWFCM 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 in increase in areas where these methods would be allowed.

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

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

ICCFILENAME: Stafford-S1-2109

S180–09/10
2113.1

Proponent: Jim Buckley, Buckley Rumford Co., representing the Masonry Alliance for Codes and Standards (MACS) & Clay Flue Lining Institute (CFLI)

Revise as follows:

2113.1 Definition. A masonry chimney is a chimney constructed of ~~concrete or masonry~~ solid masonry units, hollow masonry units grouted solid, stone or concrete, hereinafter referred to as "masonry." Masonry chimneys shall be constructed, anchored, supported and reinforced as required in this chapter.

Reason: To make language same as for fireplaces and smoke chambers and to be consistent with IRC Section R1003.1 and definitions in Chapters 3 and 4.

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

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

ICCFILENAME: BUCKLEY-S1-2113.1

S181–09/10
2113.20

Proponent: Jim Buckley, Buckley Rumford Co., representing the Masonry Alliance for Codes and Standards (MACS) & Clay Flue Lining Institute (CFLI)

Revise as follows:

2113.20 Chimney fireblocking. All spaces between chimneys and floors and ceilings through which chimneys pass shall be fireblocked with noncombustible material securely fastened in place. The fireblocking of spaces between wood joists, beams or headers shall be ~~to a depth of 1 inch (25 mm) and shall only~~ self-supporting or be placed on strips of metal or metal lath laid across the spaces between combustible material and the chimney.

Reason: To be consistent with IRC Section R1003.19.

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

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

ICCFILENAME: BUCKLEY-S4-2113.20

S182-09/10

2113.9.1 (New), 2113.9.3 (New); IRC R1003.9.1 (New), R1003.9.3 (New)

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

Proponent: Jim Buckley, Buckley Rumford Co., representing the Masonry Alliance for Codes and Standards (MACS) & Clay Flue Lining Institute (CFLI)

PART I – IBC STRUCTURAL

Add new text as follows:

2113.9.1 Chimney caps. Masonry chimneys shall have a concrete, metal or stone cap, sloped to shed water, a drip edge and a caulked bond break around any flue liners in accordance with ASTM C 1283.

~~2113.9.1~~ 2113.9.2 Spark arrestors.

(No change to current text)

2113.9.3 Rain caps. Where a masonry or metal rain cap is installed on a masonry chimney, the net free area under the cap shall not be less than four times the net free area of the outlet of the chimney flue it serves.

PART II – IRC BUILDING/ENERGY

Add new text as follows:

R1003.9.1 Chimney caps. Masonry chimneys shall have a concrete, metal or stone cap, sloped to shed water, a drip edge and a caulked bond break around any flue liners in accordance with ASTM C 1283.

~~R1003.9.1~~ R1003.9.2 Spark arrestors. *(No change to current text)*

R1003.9.3 Rain caps. Where a masonry or metal rain cap is installed on a masonry chimney, the net free area under the cap shall not be less than four times the net free area of the outlet of the chimney flue it serves.

Reason: New language to include provision for commonly used chimney caps and rain caps. This language will reference and be consistent with ASTM C1283 and C315.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: BUCKLEY-S2-2113.9

S183–09/10
2113.12

Proponent: Jim Buckley, Buckley Rumford Co., representing the Masonry Alliance for Codes and Standards (MACS) & Clay Flue Lining Institute (CFLI)

Revise as follows:

2113.12 Clay flue lining (installation). Clay flue liners shall be installed in accordance with ASTM C 1283 and extend from a point not less than 8 inches (203 mm) below the lowest inlet or, in the case of fireplaces, from the top of the smoke chamber to a point above the enclosing walls. The lining shall be carried up vertically, with a maximum slope no greater than 30 degrees (0.52 rad) from the vertical.

Clay flue liners shall be laid in medium-duty non-water soluble refractory mortar conforming to ASTM C 199 with tight mortar joints left smooth on the inside and installed to maintain an air space or insulation not to exceed the thickness of the flue liner separating the flue liners from the interior face of the chimney masonry walls. Flue lining shall be supported on all sides. Only enough mortar shall be placed to make the joint and hold the liners in position.

Reason: To require non-water soluble refractory mortar which won't wash out of the joints in chimneys which are exposed to weather. Also to be consistent with IRC Section R1003.12

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

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

ICCFILENAME: BUCKLEY-S3-2113.12

S184–09/10
2204.2.1

Proponent: Bonnie Manley, representing American Iron and Steel Institute

Revise as follows:

2204.2.1 Anchor rods. Anchor rods shall be set in accordance with the *construction documents*~~accurately to the pattern and dimensions called for on the plans~~. The protrusion of the threaded ends through the connected material shall ~~be sufficient to fully~~ engage the threads of the nuts, but shall not be greater than the length of the threads on the bolts.

Reason: The modifications to this section are intended to be editorial in nature. In the first sentence, the defined term “construction documents” replaces the undefined term “plans”. In the second sentence, vague language is eliminated.

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

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

ICCFILENAME: Manley-S2-2204.2.1

S185–09/10
2206.5 (New)

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

Add new text as follows:

2206.5 Steel joist incidental loads. In addition to the loads required by Table 1607.1 and the loads required in Section 2206.2, steel joists shall be designed for an additional incidental load of 400 pounds (1.779 kN), of which one single concentrated load of up to 300 pounds (1.335 kN) shall be placed between any two top chord panel points and a single concentrated load of up to 100 pounds (0.445 kN) shall be placed between any two bottom chord panel points.

(Renumber subsequent sections)

Reason: Many, if not most, steel joist roof structures which are currently in service have miscellaneous incidental loads suspended from the joist's top or bottom chords. Some joist manufactures have an allowance for such loads, others do not. There are manufacturers of hardware specifically designed to allow for the suspension of such loads. This code change proposal is intended to provide an allowance for such incidental loads

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

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

ICCFILENAME: HUSTON-S3-2206.5

S186–09/10
2208.1

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 <u>additional</u> 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.

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

ICCFILENAME: Manley-S4-2208.1

S187–09/10
2208.1, Chapter 35

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 <u>additional</u> 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~~ <u>11</u> Specification for Design, Testing and Utilization of Industrial Steel Storage Rack<u>, 2011</u>

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.

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

ICCFILENAME: Manley-S5-2208.1

S188–09/10
1604.3.3, 2209.2.1

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.

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

ICCFILENAME: HUSTON-S8-2209.2

S189–09/10
2209.1 through 2209.1.1.3, 2209.2 through 2209.3.6 (New)

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. <u>Where required, the seismic design of cold-formed steel structures shall be in accordance with the additional provisions of Section 2209.2.</u>

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

~~2209.2.1~~ <u>2209.1.1.1</u> 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~~ <u>2209.1.1.2</u> 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 ~~2209.2.2.1~~ <u>2209.1.1.2.1</u>.

~~2209.2.2.1~~ <u>2209.1.1.2.1</u> 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 11/2 inches (38 mm).

~~2209.2.3~~ <u>2209.1.1.3</u> 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

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

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, , 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 0.05*h* drift limit in Section D1.3 of AISI S110 to 0.03*h* in order to more closely align with the 0.025*h* 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 0.05*h* 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.

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

ICCFILENAME: Manley-S3-2209

S190–09/10
2209.2.1

Proponent: Roy H. Reiterman, PE, representing Wire Reinforcement Institute, Inc, Technical Consultant for WRI

Revise as follows:

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 <u>and ACI 318.</u>

Reason: To eliminate confusion on whether steel reinforcement is required for the above elevated slab section. We are proposing the above add/change for the next 2012 Code edition and succeeding IBC Code sections. Section 2209.2.2 is all right as it appears in the current IBC Code and as it references 2209.2.2.1

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

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

ICCFILENAME: RETIERMAN-S1-2209

S191–09/10

2209.2.1, 2209.2.1.1 (New) Chapter 35

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/m^3) 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.

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

ICCFILENAME: Sputo-S1-2209.2.1

S192–09/10
2209.2.1, 2209.2.1.1 (New) Chapter 35

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 <u>permitted to be</u> designed and constructed in accordance with ~~ASCE 3~~ <u>ANSI/SDI-C1.0 as modified in Section 2209.2.1.1.</u>

<u>**2209.2.1.1 ANSI/SDI-C1.0 Section 2.4B7a.** Replace Section 2.4B7a of ANSI/SDI-C1.0 with the following:</u>

<u>a. Temperature and shrinkage effects in the concrete shall be controlled by one of the following alternatives:</u>

<u>1. Welded wire reinforcement or reinforcing bars with a minimum area of 0.00075 times the area of the concrete above the deck (per foot or meter of width), but not be less than the area provided by 6 x 6 – W1.4 x W1.4 (152 x 152 – MW9 x MW9) welded wire reinforcement. Reinforcing shall be properly supported and shall not rest in contact with the steel deck.</u>

2. Add standard to Chapter 35 as follows:

SDI
<u>C1.0-10</u> <u>Standard for Composite Steel Floor Deck</u>

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 C-2010 (currently under development as an update to ANSI/SDI C1.0) is proposed for inclusion because it is the current standard for the design of composite steel deck. This standard, once completed, will be 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 2.4B7a 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 2.4B7a – Text as it appears in SDI-C-2010:

a. Temperature and shrinkage effects in the concrete shall be controlled by one of the following alternatives:

1. Welded wire reinforcement or reinforcing bars with a minimum area of 0.00075 times the area of the concrete above the deck (per foot or meter of width), but not be less than the area provided by 6 x 6 – W1.4 x W1.4 (152 x 152 – MW9 x MW9) welded wire reinforcement. Reinforcing shall be properly supported and shall not rest in contact with the steel deck.
2. Concrete specified in accordance with ASTM C1116, Type I, containing steel fibers meeting the criteria of ASTM A820, Type I , Type II, or Type V, at a minimum addition rate of 25 lb/cu yd (14.8 kg/cu meter)
3. Concrete specified in accordance with ASTM C1116, Type III, containing macrosynthetic fibers made from virgin polyolefin, having an equivalent diameter between 0.012 inches (0.3 mm) and 0.050 inches (1.25 mm), and having a minimum aspect ratio (length/equivalent diameter) of 50, at a minimum addition rate of 4 lb./cu yd (2.4 kg/m3).

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 C1.0-10, 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.

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

ICCFILENAME: Sputo-S2-2209.2.1-2

S193–09/10
2209.2.1

Proponent: Thomas Sputo, Ph.D., PE, SE, Steel Deck Institute

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 remaining sections)

Reason: This section is proposed for deletion because ASCE 3-91 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 standards activities rules require standards to be reaffirmed at intervals not to exceed 5 years. Additionally, this standard is out-of-print and is therefore not readily available to code officials, designers, or users of the code.

In lieu of this section and a reference standard, a designer would rely on alternate methods as permitted by IBC 104.11.

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

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

ICCFILENAME: Sputo-S3-2209.2.1-3

S194–09/10
2209.2.2, 2209.2.2.1

Proponent: Thomas Sputo, Ph.D., PE, SE, Steel Deck Institute

Revise as follows:

2209.2.2 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 2209.2.2.1.~~

Delete without substitution:

~~**2209.2.2.1 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 11/2 inches (38 mm).~~

Reason: 2209.2.2.1 deleted the last sentence of ANSI/SDI-NC1.0 which read "Randomly distributed fibers or fibrous admixtures shall not be substituted for welded wire fabric tensile reinforcement. This sentence does not contradict and is in compliance with requirements of ACI 318, as referenced earlier in Section 2.4B1 of ANSI/SDI-NC1.0. As such, the deleted sentence should not be deleted.
Section 2.4B1 – Text as it appears in SDI-NC1.0:

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). Randomly distributed fibers or fibrous admixtures shall not be substituted for welded wire fabric tensile reinforcement.

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

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

ICCFILENAME: Sputo-S4-2209.2.2

S195–09/10
202, 2302.1, 2304.11.2.3

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

Revise as follows:

SECTION 202
DEFINITIONS

NATURALLY DURABLE WOOD. ~~See Section 2302.1~~ <u>The heartwood of the following species except for the occasional piece with corner sapwood, provided 90 percent or more of the width of each side on which it occurs is heartwood</u>.

Decay resistant. ~~See Section 2302.1~~ <u>Redwood, cedar, black locust and black walnut</u>

Termite resistant. ~~See Section 2302.1~~ <u>Redwood, Alaska yellow cedar, Eastern red cedar and both heartwood and all sapwood of Western red cedar</u>.

2302.1 Definitions. The following words and terms, shall, for the purposes of this chapter, have the meanings shown herein.

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

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

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

2304.11.2.3 Exterior walls below grade. Wood framing members and furring strips attached directly to the interior of exterior masonry or concrete walls below grade shall be of ~~approved~~ naturally durable or preservative-treated wood.

Reason: The placement of the definition of "naturally durable wood" in Section 2302.1 limits its applicability to Chapter 23 as specified in Section 2302.1. Naturally durable wood, however, is also specified in Sections 1507.8.5 and 1507.9.8 as well as Tables 1507.8, 1507.8.5, 1507.8.7, 1507.9.6 and 1507.9.8. Relocating the definition of "naturally durable wood" from Section 2302.1 to Section 202 will expand its applicability to account for all instances of the term in the IBC. The definition is also revised editorially so that the text is nonmandatory.

In Section 2304.11.2.3, "approved" before "naturally durable" is deleted due to redundancy. Since the term is defined, meeting the definition is seen as sufficient since procedures for approval by the building official are specified in Chapter 1. The deletion will also make Section 2304.11.2.3 consistent with all other instances of "naturally durable" in the IBC of which none are preceded by "approved." In addition to the sections and tables noted above, refer to Sections 2304.11.1 through 2304.11.2.2, 2304.11.2.4 through 2304.11.4, 2304.11.4.2, 2304.11.5 and 2304.11.6.

References to "naturally durable" and "naturally durable wood" are also found in the 2009 IRC, including a definition for "naturally durable wood" in Section R202. According to Section R201.1, the definitions in Section R202 are applicable throughout the IRC in the same manner as the definitions in IBC Section 202 are applicable throughout the IBC. Thus, the changes in this proposal for the IBC are not needed in the IRC and will make the IBC consistent with the IRC with respect to naturally durable wood.

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

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

ICCFILENAME: Brazil-S38-2302.1

S196–09/10
2302.1

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

1. Revise as follows:

STRUCTURAL COMPOSITE LUMBER. Structural members manufactured using wood elements bonded together with exterior adhesives. Examples of structural composite lumber are:

Laminated veneer lumber (LVL). A composite of wood veneer elements with wood fibers primarily oriented along the length of the member. <u>Veneer thickness shall not exceed 0.25 inches (6.4 mm).</u>

Parallel strand lumber (PSL). A composite of wood strand elements with wood fibers primarily oriented along the length of the member. <u>The least dimension of the strands shall not exceed 0.25 inches (6.4 mm) and the average length shall be a minimum of 300 times the least dimension.</u>

<u>**Laminated strand lumber (LSL).** A composite of wood strand elements with wood fibers primarily oriented along the length of the member. The least dimension of the strands shall not exceed 0.10 inches (2.54 mm) and the average length shall be a minimum of 150 times the least dimension.</u>

<u>**Oriented strand lLumber (OSL).** A composite of wood strand elements with wood fibers primarily oriented along the length of the member. The least dimension of the strands shall not exceed 0.10 inches (2.54 mm) and the average length shall be a minimum of 75 times the least dimension.</u>

Reason: ASTM Standard D5456 recognizes 4 types of structural composite lumber. This proposal adds the two types missing from the existing definition and makes them consistent with ASTM D5456

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

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

ICCFILENAME: KEITH-S2-2302.1 Definitions

S197–09/10
2302.1

Proponent: Harvey B. Manbeck, PE, Manbeck Engineering, Inc., representing the National Frame Building Association

Add new definition as follows:

2302.1 Definitions. The following words and terms shall, for the purposes of this chapter, have the meanings shown herein.

POST-FRAME BUILDING SYSTEM. A post-frame building system is characterized by primary structural frames of posts as columns and trusses or rafters as roof framing. The roof framing is attached to the posts, either directly or indirectly through structural headers. Posts are typically graded lumber, graded timbers, laminated lumber or fabricated of composite or hybrid material. Posts are embedded in the soil and supported on isolated footings, or are attached to the top of piers, concrete or masonry walls, slabs-on-grade, or other suitable foundations. Secondary framing members, purlins in the roof and girts in the walls, are attached to the primary framing members to provide lateral support and to transfer sheathing loads, both in-plane and out-of-plane, to the posts and roof framing. Structures are sheathed with a wide variety of materials, including metal and wood structural panels or other suitable materials.

Reason: Post-frame foundations, metal-clad, wood-framed diaphragm panels and design procedures, and mechanically laminated posts are referenced Section 2306.1 and Chapter 35 of the current edition of the IBC. However, nowhere are post-frame building systems defined in the IBC. This causes post-frame building systems to be confused with other wood framed building systems. Addition of the proposed definition would eliminate such confusion.

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

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

ICCFILENAME: MANBECK-S1-2302

S198–09/10
2303.1.1, 2303.1.1.1 (New), 2303.1.1.2 (New)

Proponent: Sam Francis, American Forest & Paper Association

Revise as follows:

2303.1.1 Sawn lumber. Sawn lumber used for load-supporting purposes, including end-jointed or edge-glued lumber, machine stress-rated or machine-evaluated lumber, shall be identified by the grade *mark* of a lumber grading or inspection agency that has been approved by an accreditation body that complies with DOC PS 20 or equivalent. Grading practices and identification shall comply with rules published by an agency approved in accordance with the procedures of DOC PS 20 or equivalent procedures.

2303.1.1.1 Certificate of Inspection. In lieu of a grade mark on the material, a certificate of inspection as to species and grade issued by a lumber grading or inspection agency meeting the requirements of this section is permitted to be accepted for precut, remanufactured or rough-sawn lumber and for sizes larger than 3 inches (76 mm) nominal thickness.

2303.1.1.2 End-jointed lumber. Approved end-jointed lumber is permitted to be used interchangeably with solid-sawn members of the same species and grade. End-jointed lumber used in an assembly required elsewhere in this code to have a fire resistance rating shall have the designation "Heat Resistant Adhesive" or "HRA" included in its grade mark.

Reason: The American Lumber Standards Committee (ALSC) recently added elevated-temperature performance requirements for end-jointed lumber adhesives intended for use in fire resistance-rated assemblies. End-jointed lumber manufactured with adhesives which meet the new requirements is being designated as "Heat Resistant Adhesive" or "HRA" on the grade stamp. Heat Resistant Adhesives are required to be qualified in accordance with one of two new ASTM standards, *D7374-08 Practice for Evaluating Elevated Temperature Performance of Adhesives Used in End-Jointed Lumber* and *D7470-08 Practice for Evaluating Elevated Temperature Performance of End-Jointed Lumber Studs*. End-jointed lumber manufactured with a Heat Resistant Adhesive under an auditing program of an ALSC-accredited grading agency is allowed to carry the HRA mark on the grade-stamp. End-jointed lumber manufactured with an adhesive not qualified as a Heat Resistant Adhesive will be designated as "Non-Heat Resistant Adhesive" or "non-HRA" on the grade stamp. Lumber carrying the HRA mark is permitted to be used interchangeably with solid-sawn members of the same species and grade in fire-rated applications.

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

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

ICCFILENAME: FRANCIS-S2-2303.1.1.1

S199–09/10
2303.1.4, 2304.6.2, Table 2306.3, Chapter 35; IRC R604.1, Table R703.4, Chapter 44

Proponent: Edward L. Keith, PE, representing APA – the Engineered Wood Association

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

1. Revise as follows:

2303.1.4 Wood structural panels. Wood structural panels, when used structurally (including those used for siding, roof and wall sheathing, subflooring, diaphragms and built-up members), shall conform to the requirements for their type in DOC PS 1, DOC ~~or~~ PS 2 or ANSI/APA PRP 210. Each panel or member shall be identified for grade and glue type by the trademarks of an approved testing and grading agency. Wood structural panel components shall be designed and fabricated in accordance with the applicable standards listed in Section 2306.1 and identified by the trademarks of an *approved* testing and inspection agency indicating conformance with the applicable standard. In addition, wood structural panels when permanently exposed in outdoor applications shall be of exterior type, except that wood structural panel roof sheathing exposed to the outdoors on the underside is permitted to be interior type bonded with exterior glue, Exposure 1.

2304.6.2 Interior paneling. Softwood wood structural panels used for interior paneling shall conform to the provisions of Chapter 8 and shall be installed in accordance with Table 2304.9.1. Panels shall comply with DOC PS 1, DOC ~~or~~ PS 2 or ANSI/APA PRP 210. Prefinished hardboard paneling shall meet the requirements of CPA/ANSI A135.5. Hardwood plywood shall conform to HPVA HP-1.

TABLE 2306.3
ALLOWABLE SHEAR (POUNDS PER FOOT) FOR WOOD STRUCTURAL PANEL SHEAR WALLS WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHER PINE[a] FOR WIND OR SEISMIC LOADING[b, h, i, j, l]

PANEL GRADE	MINIMUM NOMINAL PANEL THICKNESS (inch)	MINIMUM FASTENER PENETRATION IN FRAMING (inches)	PANELS APPLIED DIRECT TO FRAMING: Nail (common or galvanized box) or staple size[k h]	Fastener spacing at panel edges (inches): 6	4	3	2[e d]	PANELS APPLIED OVER ½" or 5/8" GYPSUM SHEATHING: Nail (common or galvanized box) or staple size[k h]	Fastener spacing at panel edges (inches): 6	4	3	2[e d]
Sheathing, plywood siding[g] except Group 5 species, ANSI/APA PRP 210 siding	5/16[c] or 1/4[c]	1 ¼	6d (2"x0.113" common, 2"x0.099" galvanized box)	180	270	350	450	8d (2 ½" x 0.131" common, 2 ½" x 0.113" galvanized box)	180	270	350	450
		1	1 ½ 16 Gage	145	220	295	375	2 16 Gage	110	165	220	285
	3/8	1 ¼	6d (2"x0.113" common, 2"x0.099" galvanized box)	200	300	390	510	8d (2 ½" x 0.131" common, 2 ½" x 0.113" galvanized box)	200	300	390	510
		1 3/8	8d (2 ½" x 0.131" common, 2 ½" x 0.113" galvanized box)	220[d]	320[d]	410[d]	530[d]	10d (3" x 0.148" common, 3" x 0.128" galvanized box)	260	380	490[f]	640
		1	1 ½ 16 Gage	140	210	280	360	2 16 Gage	140	210	280	360
	7/16	1 3/8	8d (2 ½" x 0.131" common, 2 ½" x 0.113" galvanized box)	240[d]	350[d]	450[d]	585[d]	10d (3" x 0.148" common, 3" x 0.128" galvanized box)	260	380	490[f]	640
		1	1 ½ 16 Gage	155	230	310	395	2 16 Gage	140	210	280	360
	15/32	1 3/8	8d (2 ½" x 0.131" common, 2 ½" x 0.113" galvanized box)	260	380	490	640	10d (3" x 0.148" common, 3" x 0.128" galvanized box)	260	380	490[f]	640
		1 ½	10d (3" x 0.148" common, 3" x 0.128" galvanized box)	310	460	600[f]	770	-	-	-	-	-
		1	1 ½ 16 Gage	170	255	335	430	2 16 Gage	140	210	280	360
	19/32	1 ½	10d (3" x 0.148" common, 3" x 0.128" galvanized box)	340	510	665[f]	870	-	-	-	-	-
		1	1 3/4 16 Gage	185	280	375	475	-	-	-	-	-
			Nail Size (galvanized casing)					Nail Size (galvanized casing)				
	5/16[c]	1 ¼	6d (2" x 0.099")	140	210	275	360	8d (2 ½" x 0.113")	140	210	275	360
	3/8[c]	1 3/8	8d (2 ½" x 0.113")	160	240	310	410	10d (3" x 0.128")	160	240	310[f]	410

(Remainder of table and footnotes unchanged)

2. Add new standard to Chapter 35 as follows:

APA
ANSI/APA PRP 210-08 Standard for Performance-Rated Engineered Wood Siding

PART II – IRC BUILDING/ENERGY

1. Revise as follows:

R604.1 Identification and grade. Wood structural panels shall conform to DOC PS 1, ~~or~~ DOC PS 2 or ANSI/APA PRP 210; or, when manufactured in Canada, CSA 0437 or CSA 0325. All panels shall be identified by a grade mark or certificate of inspection issued by an approved agency.

2. Revise as follows:

TABLE R703.4
WEATHER–RESISTANT SIDING ATTACHMENT AND MINIMUM THICKNESS

				TYPE OF SUPPORTS FOR THE SIDING MATERIAL AND FASTENERS [b,c,d]					
SIDING MATERIAL	NOMINAL THICKNESS[a] (INCHES)	JOINT TREATMENT	WATER-RESISTIVE BARRIER REQUIRED	Wood or wood structural panel sheathing	Fiberboard sheathing into stud	Gypsum sheathing into stud	Foam plastic sheathing into stud	Direct to Studs	Number or spacing of fasteners
Wood structural panel[i], ANSI/APA PRP 210, siding[i] (exterior grade)	3/8 – 1/2	Note p	YES	0.099 NAIL–2"	0.113 NAIL–21/2"	0.113 NAIL–21/2"	0.113 nail[v]	0.099 NAIL–2"	6" PANEL EDGES, 12" INTER. SUP.

(Remainder of table and footnotes unchanged)

3. Add new standard to Chapter 44 as follows:

APA
ANSI/APA PRP 210-08 Standard for Performance-Rated Engineered Wood Siding

Reason:
PART I- This proposal recognized a new national consensus standard for wood structural panel siding products. Such products were formerly covered under several national standards such as APA PRP-108 and others. The industry developed this standard via the ANSI consensus process. Siding products manufactured to this standard have been developed specifically for wall covering/weather proofing applications, carry an Exterior exposure durability classification, and panels have equivalent shear performance on a thickness by thickness basis when nailed in accordance with Table 2306.3. Copies of this new ANSI standard have been provide to the committee and can be downloaded free of charge at the apawood.org website (Form number J350).
PART II- This proposal recognized a new national consensus standard for wood structural panel siding products. Such products were formerly covered under several industry standards such as APA PRP-108 and others. The industry developed this standard via the ANSI consensus process. Siding products manufactured to this standard have been developed specifically for wall covering/weather proofing applications and carry an Exterior exposure durability classification. Copies of this new ANSI standard have been provided to the committee and can be downloaded free of charge at the apawood.org website (Form number J350).

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/APA PRP 210-08, 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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: KEITH-S1-2303.1.4

S200–09/10
2303.1.4; IRC R503.2.1, R503.2.1.1, R602.3, R803.2.1

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

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC-STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

2303.1.4 Wood structural panels. Wood structural panels, when used structurally (including those used for siding, roof and wall sheathing, subflooring, diaphragms and built-up members), shall conform to the requirements for their type in DOC PS 1 or PS 2. Each panel or member shall be identified for grade, ~~and glue type~~ bond classification, and Performance Class by the trademarks of an *approved* testing and grading agency. The Performance Class value shall be used as the "nominal panel thickness" or "panel thickness" whenever referenced in this code. Wood structural panel components shall be designed and fabricated in accordance with the applicable standards listed in Section 2306.1 and identified by the trademarks of an *approved* testing and inspection agency indicating conformance with the applicable standard. In addition, wood structural panels when permanently exposed in outdoor applications shall be of ~~exterior~~ Exterior type, except that wood structural panel roof sheathing exposed to the outdoors on the underside is permitted to be ~~interior type bonded with exterior glue,~~ Exposure 1 type.

PART II – IRC BUILDING/ENERGY

Revise as follows:

R503.2.1 Identification and grade. Wood structural panel sheathing used for structural purposes shall conform to DOC PS 1, DOC PS 2 or, when manufactured in Canada, CSA O437 or CSA O325. All panels shall be identified for grade, bond classification, and Performance Class by a grade mark of certificate or inspection issued by an approved agency. The Performance Class value shall be used as the "nominal panel thickness" or "panel thickness" whenever referenced in this code.

R503.2.1.1 Subfloor and combined subfloor underlayment. Where used as subflooring or combination subfloor underlayment, wood structural panels shall be of one of the grades specified in Table R503.2.1.1(1). When sanded plywood is used as combination subfloor underlayment, the grade, bond classification, and Performance Class shall be as specified in Table R503.2.1.1(2).

R602.3 Design and construction. Exterior walls of wood-frame construction shall be designed and constructed in accordance with the provisions of this chapter and Figures R602.3(1) and R602.3.(2) or in accordance with AF&PA's NDS. Components of exterior walls shall be fastened in accordance with Tables R602.3(1) through R602.3(4). Structural wall sheathing shall be fastened directly to structural framing members. Exterior wall coverings shall be capable of resisting the wind pressures listed in Table R301.2(2) adjusted for height and exposure using Table R301.2(3). Wood structural panel sheathing used for exterior walls shall conform to DOC PS 1, DOC PS 2 or, when manufactured in Canada, CSA O437 or CSA O325. All panels shall be identified for grade, bond classification, and Performance Class by a grade mark or certificate of inspection issued by an approved agency and shall conform to the requirements of Table R602.3(3).

R803.2.1 Identification and grade. Wood structural panels shall conform to DOC PS 1, DOC PS 2 or, when manufactured in Canada, CSA O437 or CSA O325, and shall be identified for grade, bond classification, and Performance Class by a grade mark or certificate of inspection issued by an approved agency. Wood structural panels shall comply with the grades specified in Table R503.2.1.1(1).

Reason: (IBC & IRC) This is a nomenclature change that reflects the newest versions of National Standards PS 1 and PS 2. Wood structural panels are required to be in conformance to DOC PS 1 and PS 2 in the code. The PS 1 and PS 2 consensus standard committees have revised both standards to include the terminologies of "bond classification" to reference glue type and "Performance Classes" to reference the thicknesses tolerance consistent with the nominal panel thicknesses in the IBC. This change proposal updates the code to the nomenclature that appears on the trademark of wood structural panels in the field in accordance with DOC PS 1 and PS 2. This is not a technical change.

(IRC)In Section R602.3, the description of wood structural panel was added as it shows up in Chapters 5 and 8 where wood structural panels are also specified. This was done to make the code read consistently between similar sections.

Cost Impact: This will **not** impact the cost of construction.

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: Keith-S4-2303.1.4

S201-09/10

2303.2, 2303.2.1, 2303.2.2, 2303.2.3; IRC R802.1.3, R802.1.3.1, R802.1.3.2, R802.1.3.3

Proponent: Joe Holland and Dave Bueche, representing Hoover Treated Wood Products

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDER FOR THE STRUCTURAL COMMITTEE.

PART I- IBC STRUCTURAL

1. Revise as follows:

2303.2 Fire-retardant-treated wood. *Fire-retardant-treated wood* (FRTW) is a pressure treated ~~any~~ wood product ~~which, when impregnated with chemicals by a pressure process or other means during manufacture, shall have, when~~ tested in accordance with ASTME 84 or UL723~~,~~.FRTW shall have a *listed* flame spread index of 25 or less and show no evidence of significant progressive combustion when the test is continued for an additional 20-minute period. ~~Additionally, t~~The flame front shall not progress more than 101/2 feet (3200 mm) beyond the centerline of the burners at any time during the test.

2. Delete without substitution:

~~**2303.2.1 Pressure process.** For wood products impregnated with chemicals by a pressure process, the process shall be performed in closed vessels under pressures not less than 50 pounds per square inch gauge (psig) (345 kPa).~~

~~**2303.2.2 Other means during manufacture.** For wood products produced by other means during manufacture, the treatment shall be an integral part of the manufacturing process of the wood product. The treatment shall provide permanent protection to all surfaces of the wood product.~~

~~**2303.2.3 Testing.** For wood products produced by other means during manufacture, other than a pressure process, all sides of the wood product shall be tested in accordance with and produce the results required in Section 2303.2. Wood structural panels shall be permitted to test only the front and back faces.~~

(Renumber remaining sections)

PART II- IRC BUILDING/ENERGY

1. Revise as follows:

R802.1.3 Fire-retardant-treated wood. *Fire-retardant-treated wood* (FRTW) is a pressure treated ~~any~~ wood product ~~which, when impregnated with chemicals by a pressure process or other means during manufacture, shall have, when~~ tested in accordance with ASTME 84~~,~~.FRTW shall have a *listed* flame spread index of 25 or less and show no evidence of significant progressive combustion when the test is continued for an additional 20-minute period. ~~Additionally, t~~The flame front shall not progress more than 10.5 feet (3200 mm) beyond the centerline of the burners at any time during the test

2. Delete without substitution:

~~**R802.1.3.1 Pressure process.** For wood products impregnated with chemicals by a pressure process, the process shall be performed in closed vessels under pressures not less than 50 pounds per square inch gauge (psig) (345 kPa).~~

~~**R802.1.3.2 Other means during manufacture.** For wood products produced by other means during manufacture, the treatment shall be an integral part of the manufacturing process of the wood product. The treatment shall provide permanent protection to all surfaces of the wood product.~~

~~**R802.1.3.3 Testing.** For wood products produced by other means during manufacture, other than a pressure process, all sides of the wood product shall be tested in accordance with and produce the results required in Section 2303.2. Wood structural panels shall be permitted to test only the front and back faces.~~

(Renumber remaining sections)

Reason: Revision is more concise. Present section is wordy. In the fifty years of recognition of FRTW in the code there is no wood product meeting the requirement of FRTW where adding the fire retardant to the wood is done during manufacture. This provision creates interpretation problems in the field. Revision will improve enforcement of section. "Pressure process" and "other means during manufacturer" are no longer used; delete Sections 2303.2.1 through 2303.2.3.

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

PART I- IBC STRUCTURAL

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

PART II-IRC BUILDING/ENERGY

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

FILENAME: Holland-Bueche-S1-2303.2-RB-2-R802.1.3

S202–09/10
2304.8.4.2, 2304.8.5.2

Proponent: Jeff Linville, PE, representing American Institute of Timber Construction

Revise as follows:

2304.8.4.2 Nailing. Each piece of decking shall be toenailed at each support with one 16d common nail through the tongue and face-nailed with one 16d common nail. <u>Other nailing patterns are permitted where justified by an engineering analysis. Predrilled holes are permitted to be used to prevent splitting.</u>

2304.8.5.2 Nailing. Each piece shall be toenailed at each support with one 40d common nail and face-nailed with one 60d common nail. <u>Other nailing patterns are permitted where justified by an engineering analysis. Predrilled holes are permitted to be used to prevent splitting.</u> Courses shall be spiked to each other with 8 inch (203 mm) spikes at maximum intervals of 30 inches (762 mm) through predrilled edge holes penetrating to a depth of approximately 4 inches (102 mm). One spike shall be installed at a distance not exceeding 10 inches (254 mm) from the end of each piece.

Reason: Sections 2304.8.4.2 and 2304.8.5.2 are proposed to be revised to allow other than the prescriptive nailing patterns if justified by engineering analysis. Additionally, these sections are further revised to allow predrilled holes to prevent splitting, such as when nails are placed near the ends of boards.

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

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

ICCFILENAME: LINVILLE-S1-2304.8.4.2

S203–09/10

2304.9.5, 2304.9.5.1, 2304.9.5.2, 2304.9.5.3, 2304.9.5.4; IRC R317.3, R317.3.1, R 317.3.2, R317.3.3, R317.3.4

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

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

2304.9.5 Fasteners and connectors in contact with preservative-treated and fire-retardant-treated wood. Fasteners, including nuts and washers, and connectors in contact with *preservative-treated* and *fire-retardant-treated wood* shall be in accordance with Sections 2304.9.5.1 through 2304.9.5.4. The coating weights for zinc-coated fasteners shall be in accordance with ASTM A 153.

2304.9.5.1 Fasteners and connectors for preservative-treated wood. Fasteners, including nuts and washers, in contact with *preservative-treated wood* shall be of hot-dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper. Fasteners other than nails, timber rivets, wood screws and lag screws shall be permitted to be of mechanically deposited zinc-coated steel with coating weights in accordance with ASTM B 695, Class 55 minimum. Connectors that are used in exterior applications and in contact with *preservative-treated wood* shall have coating types and weights in accordance with the treated wood or connector manufacturer's recommendations. In the absence of manufacturer's recommendations, a minimum of ASTM A 653, type G185 zinc-coated galvanized steel, or equivalent, shall be used.

> **Exception:** Plain carbon steel fasteners, including nuts and washers, in SBX/DOT and zinc borate *preservative-treated wood* in an interior, dry environment shall be permitted.

2304.9.5.2 Fastenings for wood foundations. Fastenings, including nuts and washers, for wood foundations shall be as required in AF&PA PWF.

2304.9.5.3 Fasteners for fire-retardant-treated wood used in exterior applications or wet or damp locations. Fasteners, including nuts and washers, for *fire-retardant-treated wood* used in exterior applications or wet or damp locations shall be of hot-dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper. Fasteners other than nails, timber rivets, wood screws and lag screws shall be permitted to be of mechanically deposited zinc-coated steel with coating weights in accordance with ASTM B 695, Class 55 minimum.

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

PART II – IRC BUILDING/ENERGY

Revise as follows:

R317.3 Fasteners and connectors in contact with preservative-treated and fire-retardant-treated wood. Fasteners, including nuts and washers, and connectors in contact with preservative-treated wood and fire-retardant-treated wood shall be in accordance with this section. The coating weights for zinc-coated fasteners shall be in accordance with ASTM A 153.

R317.3.1 Fasteners for preservative-treated wood. Fasteners, including nuts and washers, for preservative-treated wood shall be of hot-dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper. Coating types and weights for connectors in contact with preservative-treated wood shall be in accordance with the connector manufacturer's recommendations. In the absence of manufacturer's recommendations, a minimum of ASTM A 653 type G185 zinc-coated galvanized steel, or equivalent, shall be used.

Exceptions:

1. One-half-inch (12.7 mm) diameter or greater steel bolts.
2. Fasteners other than nails and timber rivets shall be permitted to be of mechanically deposited zinc coated steel with coating weights in accordance with ASTM B 695, Class 55 minimum.
3. Plain carbon steel fasteners in SBX/DOT and zinc borate preservative-treated wood in an interior, dry environment shall be permitted.

R317.3.2 Fastenings for wood foundations. Fastenings, including nuts and washers, for wood foundations shall be as required in AF&PA PWF.

R317.3.3 Fasteners for fire-retardant-treated wood used in exterior applications or wet or damp locations. Fasteners, including nuts and washers, for fire-retardant-treated wood used in exterior applications or wet or damp locations shall be of hot-dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper. Fasteners other than nails and timber rivets shall be permitted to be of mechanically deposited zinc-coated steel with coating weights in accordance with ASTM B 695, Class 55 minimum.

R317.3.4 Fasteners for fire-retardant-treated wood used in interior applications. Fasteners, including nuts and washers, for fire-retardant-treated wood used in interior locations shall be in accordance with the manufacturer's recommendations. In the absence of the manufacturer's recommendations, Section R317.3.3 shall apply.

Reason: The purpose of this proposal is to clarify the requirements for fasteners in treated wood. The nuts and washers that accompany a bolt (or other fastener) in pressure-treated or fire-retardant treated wood could also be exposed to the corrosive action of the treatments. It therefore makes sense for the nut and washer to have the same coating, or other approved coating or protection, as the fastener with which they are associated. Additionally, the language allowing carbon steel fasteners when borates are used in interior dry applications is added to the IRC. Similar language was successfully added to the IBC last cycle.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: Ehrlich-S3-2304.9.5

S204–09/10

2304.9.5.1, 2304.9.5.3

Proponent: Randall Shackelford, PE, representing Simpson Strong-Tie Co.

1. Revise as follows:

2304.9.5.1 Fasteners and connectors for preservative-treated wood. Fasteners in contact with *preservative-treated wood* shall be of hot dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper. Fasteners other than nails~~,~~and timber rivets~~, wood screws and lag screws~~ shall be permitted to be of mechanically deposited zinc coated steel with coating weights in accordance with ASTM B 695, Class 55 minimum. Connectors that are used in exterior applications and in contact with *preservative-treated wood* shall have coating types and weights in accordance with the treated wood or connector manufacturer's recommendations. In the absence of manufacturer's recommendations, a minimum of ASTM A 653, type G185 zinc-coated galvanized steel, or equivalent, shall be used.

Exception: Plain carbon steel fasteners in SBX/DOT and zinc borate *preservative-treated wood* in an interior, dry environment shall be permitted.

2304.9.5.3 Fasteners for fire-retardant-treated wood used in exterior applications or wet or damp locations. Fasteners for *fire-retardant-treated wood* used in exterior applications or wet or damp locations shall be of hot-dipped zinc-coated galvanized steel, stainless steel, silicon bronze or copper. Fasteners other than nails~~,~~ and timber rivets~~, wood screws and lag screws~~ shall be permitted to be of mechanically deposited zinc-coated steel with coating weights in accordance with ASTM B 695, Class 55 minimum.

Reason: The purpose of this code change is to permit wood screws and lag screws that are galvanized in accordance with ASTM B 695, Class 55 to be used in contact with fire-retardant and preservative-treated wood. This will bring the IBC into agreement with the fasteners in treated wood requirements of the IRC.

Mechanical galvanizing is very common for screws, and in fact is possibly the only way to deposit a thick zinc coating on them. Wood screws and lag screws are frequently installed in pre-drilled holes, so abrasion of the finish is not the same problem as for nails.

Currently, all fasteners in treated wood are permitted to be hot dipped galvanized with a coating weight in accordance with ASTM A 153. This results in a zinc coating weight on the fastener of one to 1.25 ounces of zinc per square foot, and a minimum thickness of 43 to 53 microns, depending on the diameter of the fastener. See Class C and D in copy of the standard below:

TABLE 1 Thickness or Weight [Mass] of Zinc Coating for Various Classes of Material

NOTE 1— Length of the piece, stated in Classes B-1, B-2, and B-3, refers to the finished dimension of the piece after fabrication.

Class of Material	Weight [Mass] of Zinc Coating, oz/ft² [g/m²] of Surface, Minimum		Coating Thickness, mils [microns], Minimum	
	Average of Specimens Tested	Any Individual Specimen	Average of Specimens Tested	Any Individual Specimen
Class A—Castings—Malleable Iron, Steel	2.00 [610]	1.80 [550]	3.4 [86]	3.1 [79]
Class B—Rolled, pressed, and forged articles (except those which would be included under Classes C and D):				
B-1—3⁄16 in. [4.76 mm] and over in thickness and over 15 in. [381 mm] in length	2.00 [610]	1.80 [550]	3.4 [86]	3.1 [79]
B-2—under 3⁄16 in. [4.76 mm] in thickness and over 15 in. [381 mm] in length	1.50 [458]	1.25 [381]	2.6 [66]	2.1 [53]
B-3—any thickness and 15 in. [381 mm] and under in length	1.30 [397]	1.10 [336]	2.2 [56]	1.9 [48]
Class C—Fasteners over 3⁄8 in. [9.52 mm] in diameter and similar articles. Washers 3⁄16 in. and 1⁄4 in. [4.76 and 6.35 mm] in thickness	1.25 [381]	1.00 [305]	2.1 [53]	1.7 [43]
Class D—Fasteners 3⁄8 in. [9.52 mm] and under in diameter, rivets, nails and similar articles. Washers under 3⁄16 in. [4.76 mm] in thickness	1.00 [305]	0.85 [259]	1.7 [43]	1.4 [36]

ASTM B 695, Class 55 will also provide a minimum thickness of 53 µm of zinc on the fastener. (See table below from ASTM B 695).

3. Classification

3.1 *Classes*—Zinc coatings are classified on the basis of thickness, as follows:

Class	Minimum Thickness, µm
110	107
80	81
70	69
65	66
55	53
50	50
40	40
25	25
12	12
8	8
5	5

Since the corrosion resistance of a galvanized coating is directly proportional to the amount of zinc on the fastener, providing an equivalent amount of zinc by mechanical galvanizing will provide an equivalent corrosion resistance to hot dip galvanizing to ASTM A 153. Therefore the alternative galvanizing standard should be accepted.

Further, Section X1 of the Appendix of ASTM B 695 states the following:

> "X1.2 Zinc coatings are usually applied to provide corrosion resistance. The performance of a zinc coating depends largely on its thickness, the supplementary treatment if any, and the kind of environment to which it is exposed. The seven heaviest classes of coatings offer suitable alternatives to hot-dip galvanizing."

Class 55 is the fifth heaviest so it certainly falls within the seven heaviest.

Cost Impact: This proposal will not increase costs. It could decrease costs where more economical mechanically galvanized fasteners can be used instead of hot dipped galvanized fasteners.

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

ICCFILENAME: SHACKELFORD-S2-2304.9.5.1

S205–09/10
2304.11.6

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

Revise as follows:

Section 2304.11.6 Termite protection. In geographical areas where hazard of termite damage is known to be very heavy, wood floor framing exposed to the ground in crawl spaces or unexcavated areas located within the periphery of the building foundation and exposed framing of exterior decks or balconies, shall be of durable species (termite resistant) or preservative treated in accordance with AWPA U1 for the species, product preservative and end use or provided with approved methods of termite protection.

Reason: This change intends to clarify that the wood floor framing that needs to be durable species or preservative treated wood are limited to those interior floors with exposure to soil instead of all floors in the building. In addition exposed exterior decks or balcony framing are specifically added. Other provisions address wood in contact with concrete or close to grade for all termite hazard regions.

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

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

ICCFILENAME: MAIEL-S1-2304.11.6

S206–09/10

2304.13 (New)

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

Add new text as follows:

2304.13 Buildings with prefabricated wood I-joists or trusses for floor and roof systems. All buildings using prefabricated wood I-joists or trusses for the floor or roof system shall be marked in accordance with all of the following:

1. A placard shall be attached to the building on the front of the structure in the vicinity of the front entrance and in a visible location. Additional placards shall be applied to each side of the structure in a visible location.
2. Building placards shall be 8 inches (203 mm) high by 24 inches (610 mm) long with a white background, black letters and a black border. The letters and border shall be easily visible and readable at 10 feet (3048 mm).
3. The placard shall state: *"This building uses prefabricated wood I-joists or trusses for the floor or roof system. Proceed with caution if entering during a fire emergency"*

Reason: Section 2304 of the International Building Code outlines the general construction requirements for buildings using wood materials. This includes provisions applicable to engineered wood products such as wood I-joists and trusses. Though suitable for structural loads, buildings constructed of wood I-joist or trusses pose an inherent risk to the fire service when responding to a fire emergency. There have been numerous instances where fire conditions have brought on structural failure of the floor or roof systems sooner than that experienced with structures constructed of conventional wood framed construction. This early failure under fire conditions has the potential to trap fire fighters within the building while performing interior fire attacks. This proposal will require buildings constructed with wood I-joists and trusses for the floor or roof system to have placards placed on the building to warn emergency responders of these construction features and allow them to proceed with caution if entering the building during a fire emergency.

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

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

ICCFILENAME: Thompson-S3-2304.13 (new)

S207–09/10

2302.1, 2303.1, 2303.1.12 (New), 2304.13 (New), CHAPTER 35; IRC R317.4.1 (New)

Proponent: John Woestman, The Kellen Company representing the Composite Lumber Manufacturers Association (CLMA)

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

1. Add new definition as follows:

WOOD PLASTIC COMPOSITE. A composite material made primarily from wood or cellulose-based materials, and plastic.

2. Revise as follows:

2303.1 General. Structural sawn lumber; end-jointed lumber; prefabricated wood I-joists; structural glued laminated timber; wood structural panels, fiberboard sheathing (when used structurally); hardboard siding (when used structurally); particleboard; preservative-treated wood; structural log members; structural composite lumber; round timber poles and piles; fire-retardant-treated wood; hardwood plywood; wood trusses; wood plastic composite exterior deck components; joist hangers; nails; and staples shall conform to the applicable provisions of this section.

3. Add new text as follows:

2303.1.12 Wood plastic composite exterior deck, railing, and stairway components. Structural capacities for exterior wood plastic composite deck boards, stair treads, handrails and guardrail systems shall be determined in accordance with ASTM D 7032.

2304.13 Wood plastic composite exterior deck, railing, and stairway components. Exterior wood plastic composite deck boards, deck boards used as stair treads, handrails and guardrail systems shall meet the applicable requirements of ASTM D 7032, and bear a label indicating the required performance levels and demonstrating compliance with ASTM D 7032,

4. Add new standard to Chapter 35 as follows:

ASTM

D 7032-08 Standard Specification for Establishing Performance Ratings For Wood-Plastic Composite Deck Boards and Guardrail Systems (Guards or Handrails)

5. Add new text as follows:

2304.13.1 Labeling. Labels for deck boards and stair treads shall include the allowable maximum load and span. Labels for handrails and guardrail systems shall indicate the allowable maximum span.

6. Add new text as follows:

2304.13.2 Installation. Wood plastic composite deck components shall be installed in accordance with the manufacturer's instructions.

PART II – IRC BUILDING/ENERGY

1. Add new text as follows:

R317.4.1 Labeling. Labels for deck boards and stair treads shall include the allowable maximum load and span. Labels for handrails and guardrail systems shall indicate the allowable maximum span.

2. Revise as follows:

~~R317.4.1~~ **R317.4.2** **Installation.** Wood/plastic composites shall be installed in accordance with the manufacturer's instructions.

Reason:
(Part I, items 1-4) The IBC is currently silent regarding requirements for wood plastic composite exterior deck components. The Composite Lumber Manufacturers Association (CLMA) seeks to make it easier for code officials to enforce the IBC and to make it easier for deck builders to comply with the code by incorporating requirements for wood plastic composite decking into the IBC.

This code change proposes to include requirements for wood plastic composite exterior deck components in Chapter 23 of the IBC, which is the most appropriate chapter of the IBC for these products. Section 2301.2 refers to elements or systems "constructed partially or wholly of wood or wood-based products". No other IBC chapter incorporates wood-based products of this type.

Wood plastic composite exterior deck components are constructed partially of wood-based material (as are particleboard and composite panels; included in Chapter 23), and partially of resin bonded by heat and pressure (as are several materials included in Chapter 23, such as particleboard). CLMA reviewed Chapter 26, Plastic, but concluded that wood plastic composite exterior deck components are much more closely aligned with the methods of distribution and application to the materials included in Chapter 23 than those in Chapter 26. Moreover, the ASTM standard governing wood plastic composite decking (ASTM D 7032) has been developed by and continues to be maintained by the ASTM D7 committee on wood. For these reasons, this proposal includes revisions to Chapter 23.

This CLMA proposal complements language in the 2009 IRC which defines "wood plastic composite" and requires wood plastic composite deck boards, stair treads, handrails and guardrail systems to bear a label indicating the required performance levels and demonstrating compliance to ASTM D 7032. This labeling requirement, by definition of "label" in the IBC, includes 3rd-party certification and ongoing quality assurance and will help to assure the code official that wood-plastic composite decking will meet the performance provisions in the IBC.

Complying with ASTM D 7032 verifies the wood plastic composite materials are appropriate for use as deck components and includes deck-related performance evaluations such as flexural tests, ultraviolet resistance tests, freeze-thaw resistance tests, bio-deterioration tests, fire performance tests, creep recovery tests, mechanical fastener holding tests, and slip resistance tests. The standard also includes consideration of the effects of temperature and moisture, concentrated loads, and fire propagation tests.

This code change for the IBC will make it faster and easier to verify that a deck constructed of wood plastic composite material complies with the code.

(Part I, items 5) This item adds a new subject matter in 2304.13.1. This new requirement specifies that the load and span information is required on the labels.

This item will make it faster and easier to verify that a deck constructed of wood plastic composite material complies with the code. The wood plastic composite deck boards and stair treads are to have a label indicating the span rating (i.e. 100 lbs/ft^2 at 16" O.C.). Handrails and guardrail systems will be similarly labeled. The load and span information will improve the ability to verify compliance to the code.

(Part I, items 6) This item adds a new subject matter in 2304.13.2 which requires that wood plastic composite deck components be installed per the manufacturer's instructions.

As with most engineered building components, wood plastic composite deck components should be required to be installed per the manufacturer's instructions. It's important that wood plastic composite deck components be installed as intended by the manufacturer.

(Part II, IRC) This CLMA proposal complements language proposed for IBC (see Part I, item 5).

This code change for the IRC will make it faster and easier to verify that a deck constructed of wood plastic composite material complies with the code. The wood plastic composite deck boards and stair treads are to have a label indicating the span rating (i.e. 100 lbs/ft^2 at 16" O.C.) in addition to confirming compliance to ASTM D7032. Handrails and guardrail systems will be similarly labeled with their span rating (distance between support posts).

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

Analysis: A review of the standard(s) proposed for inclusion in the code, ASTM D7032-08, 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.

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: WOESTMAN-S7-2304.13.2

S208–09/10

2305, 2306, 2307, 2308.11.2, 2308.12.2, 1704.6.1

Proponent: Brad Douglas, American Forest & Paper Association

1. Revise as follows:

2305.1 General. Structures using wood-frame shear walls ~~and~~ or wood-frame diaphragms to resist wind, seismic ~~and~~ or other lateral loads shall be designed and constructed in accordance with AF&PA SDPWS and the applicable provisions of Sections 2305, 2306 and 2307.

2305.2 Diaphragm deflection. The deflection of wood-frame diaphragms shall be determined in accordance with AF&PA SDPWS. The deflection (Δ) of a blocked wood structural panel diaphragm uniformly fastened throughout with staples is permitted to be calculated by using ~~the following~~ Equation 23-1. If not uniformly fastened, the constant 0.188 (For SI: 1/1627) in the third term shall be modified ~~accordingly~~ by an approved method.

$$\Delta = \frac{5vL^3}{8EAb} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\Sigma(\Delta_c X)}{2b}$$

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

where:

A = Area of chord cross section, in square inches (mm^2).
B = Diaphragm width, in feet (mm).
E = Elastic modulus of chords, in pounds per square inch (N/mm^2).
e_n = Staple deformation, in inches (mm) [see Table 2305.2(1)].
Gt = Panel rigidity through the thickness, in pounds per inch (N/mm) of panel width or depth [see Table 2305.2(2)].
L = Diaphragm length, in feet (mm).
V = Maximum shear due to design loads in the direction under consideration, in pounds per linear foot (plf) *(N/mm).*
Δ = The calculated deflection, in inches (mm).
$\sum(\Delta_c X)$ = Sum of individual chord-splice slip values on both sides of the diaphragm, each multiplied by its distance to the nearest support.

2305.3 Shear wall deflection. The deflection of wood-frame shear walls shall be determined in accordance with AF&PA SDPWS. The deflection (Δ) of a blocked wood structural panel shear wall uniformly fastened throughout with staples is permitted to be calculated by the use of the following equation:

$$\Delta = \frac{8vh^3}{EAb} + \frac{vh}{Gt} + 0.75he_n + d_a\frac{h}{b}$$

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

where:

A = Area of boundary element cross section in square inches (mm^2) (vertical member at shear wall boundary).
b = Wall width, in feet (mm).
d_a = Vertical elongation of overturning anchorage (including fastener slip, device elongation, anchor rod elongation, etc.) at the design shear load (v).
E = Elastic modulus of boundary element (vertical member at shear wall boundary), in pounds per square inch (N/mm^2).
e_n = Staple deformation, in inches (mm) [see Table 2305.2(1)].
Gt = Panel rigidity through the thickness, in pounds per inch (N/mm) of panel width or depth [see Table 2305.2(2)].
h = Wall height, in feet (mm).
v = Maximum shear due to design loads at the top of the wall, in pounds per linear foot (N/mm).
Δ = The calculated deflection, in inches (mm).

2306.1 Allowable stress design. The ~~structural analysis~~ design and construction of wood elements in structures using *allowable stress design* shall be in accordance with the following applicable standards:

(No change to list of allowable stress design standards)

2306.2 Wood-frame diaphragms. Wood-frame diaphragms shall be designed and constructed in accordance with AF&PA SDPWS. Where panels are fastened to framing members with staples, requirements and limitations of AF&PA SDPWS shall be met and the allowable shear values set forth in Table 2306.2(1) or 2306.2(2) shall be permitted. The allowable shear values in Tables 2306.2(1) and 2306.2(2) are permitted to be increased 40 percent for wind design.

2. Delete without substitution:

~~**2306.2.1 Wood structural panel diaphragms.** Wood structural panel diaphragms shall be designed and constructed in accordance with AF&PA SDPWS. Wood structural panel diaphragms are permitted to resist horizontal forces, using the allowable shear capacities set forth in Table 2306.2.1(1) or 2306.2.1(2). The allowable shear capacities in Tables 2306.2.1(1) and 2306.2.1(2) are permitted to be increased 40 percent for wind design.~~

~~**2306.2.2 Single diagonally sheathed lumber diaphragms.** Single diagonally sheathed lumber diaphragms shall be designed and constructed in accordance with AF&PA SDPWS.~~

~~**2306.2.3 Double diagonally sheathed lumber diaphragms.** Double diagonally sheathed lumber diaphragms shall be designed and constructed in accordance with AF&PA SDPWS.~~

3. Revise as follows:

~~**2306.2.4**~~ **2306.2.1 Gypsum board diaphragm ceilings.** Gypsum board diaphragm ceilings shall be in accordance with Section 2508.5.

TABLE ~~2306.2.1(1)~~ 2306.2(1)
ALLOWABLE SHEAR VALUES (POUNDS PER FOOT) FOR WOOD STRUCTURAL PANEL DIAPHRAGMS UTILIZING STAPLES WITH FRAMING OF DOUGLAS-FIR-LARCH, OR SOUTHERN PINE[a] FOR WIND OR SEISMIC LOADING [~~h~~ f]

PANEL GRADE	~~COMMON NAIL SIZE OR~~ STAPLE [~~f~~ d] LENGTH AND GAGE	MINIMUM FASTENER PENETRATION IN FRAMING (inches)	MINIMUM NOMINAL PANEL THICKNESS (inch)	MINIMUM NOMINAL WIDTH OF FRAMING MEMBER AT ADJOINING PANEL EDGES AND BOUN-DARIES [~~g~~ e] (inches)	BLOCKED DIAPHRAGMS				UNBLOCKED DIAPHRAGMS	
					Fastener spacing (inches) at diaphragm boundaries (all cases) at continuous panel edges parallel to load (Cases 3,4), and at all panel edges (Cases 5, 6)[b]				Fasteners spaced 6" max. at supported edges[b]	
					6	4	2 1/2[c]	2[c]	Case 1 (No unblocked edges or continuous joints parallel to load)	All other configurations (Cases 2, 3, 4, 5 and 6)
					Fastener spacing (inches) at other panel edges (Cases 1, 2, 3 and 4)					
					6	6	4	3		
Structural I grades	~~8d (2½" x 0.131)~~	~~1 3/8~~	3/8	~~2~~	~~270~~	~~360~~	~~530~~	~~600~~	~~240~~	~~180~~
				~~3~~	~~300~~	~~400~~	~~600~~	~~675~~	~~265~~	~~200~~
	1½ 16 Gage	1		2	175	235	350	400	155	115
				3	200	265	395	450	175	130
	~~10d[d] (3" x 0.148")~~	~~1 ½~~	15/32	~~2~~	~~320~~	~~425~~	~~640~~	~~730~~	~~285~~	~~215~~
				~~3~~	~~360~~	~~480~~	~~720~~	~~820~~	~~320~~	~~240~~
	1½ 16 Gage	1		2	175	235	350	400	155	120
				3	200	265	395	450	175	130
Sheathing, single floor and other grades covered in DOC PS1 and PS2	~~6d[e] (2" x 0.113)~~	~~1 ¼~~	3/8	~~2~~	~~185~~	~~250~~	~~375~~	~~420~~	~~165~~	~~125~~
				~~3~~	~~210~~	~~280~~	~~420~~	~~475~~	~~185~~	~~140~~
	~~8d (2½" x 0.131)~~	~~1 3/8~~		~~2~~	~~240~~	~~320~~	~~480~~	~~545~~	~~215~~	~~160~~
				~~3~~	~~270~~	~~360~~	~~540~~	~~610~~	~~240~~	~~180~~
	1½ 16 Gage	1		2	160	210	315	360	140	105
				3	180	235	355	400	160	120
	~~8d (2½" x 0.131)~~	~~1 3/8~~	7/16	~~2~~	~~255~~	~~340~~	~~505~~	~~575~~	~~230~~	~~170~~
				~~3~~	~~285~~	~~380~~	~~570~~	~~645~~	~~255~~	~~190~~
	1½ 16 Gage	1		2	165	225	335	380	150	110
				3	190	250	375	425	165	125
	~~8d (2½" x 0.131)~~	~~1 3/8~~	15/32	~~2~~	~~270~~	~~360~~	~~530~~	~~600~~	~~240~~	~~180~~
				~~3~~	~~300~~	~~400~~	~~600~~	~~675~~	~~265~~	~~200~~
	~~10d[d] (3" x 0.148")~~	~~1 ½~~		~~2~~	~~290~~	~~385~~	~~575~~	~~655~~	~~255~~	~~190~~
				~~3~~	~~325~~	~~430~~	~~650~~	~~735~~	~~290~~	~~215~~
	1½ 16 Gage	1		2	160	210	315	360	140	105
				3	180	235	355	405	160	120
	~~10d[d] (3" x 0.148")~~	~~1 ½~~	19/32	~~2~~	~~320~~	~~425~~	~~640~~	~~730~~	~~285~~	~~215~~
				~~3~~	~~360~~	~~480~~	~~720~~	~~820~~	~~320~~	~~140~~
	1½ 16 Gage	1		2	175	235	350	400	155	115
				3	200	265	395	450	175	130

For SI: 1 inch =- 25.4 mm, 1 pound per foot = 14.59 N/m.

(No Change to Diaphragm Figure Case 1 through Case 6)

a. For framing of other species: (1) Find specific gravity for species of lumber in AF&PA NDS. (2) For staples find shear value from table above for Structural I panels (regardless of actual grade) and multiply value by 0.82 for species with specific gravity of 0.42 or greater, or 0.65 for all other species. ~~(3) For nails find shear value from table above for nail size for actual grade and multiply value by the following adjustment factor: Specific Gravity Adjustment Factor = [1 - (0.5 - SG)], where SG = Specific Gravity of the framing lumber. This adjustment factor shall not be greater than 1.~~

b. Space fasteners maximum 12 inches o.c. along intermediate framing members (6 inches o.c. where supports are spaced 48 inches o.c.).

c. Framing at adjoining panel edges shall be 3 inches nominal or wider~~, and nails at all panel edges shall be staggered where panel edge nailing is specified 2-1/2 inches o.c. or less~~.

~~d. Framing at adjoining panel edges shall be 3 inches nominal or wider, and nails at all panel edges shall be staggered where both of the following conditions are met: (1) 10d nails having penetration into framing of more than 1-1/2 inches and (2) panel edge nailing is specified 3 inches o.c. or less.~~

~~e. 8d is recommended minimum for roofs due to negative pressures of high winds.~~

~~f.~~d. Staples shall have a minimum crown width of 7/16 inch and shall be installed with their crowns parallel to the long dimension of the framing members.

~~g~~e. The minimum nominal width of framing members not located at boundaries or adjoining panel edges shall be 2 inches.

~~h~~ f. For shear loads of normal or permanent load duration as defined by the AF&PA NDS, the values in the table above shall be multiplied by 0.63 or 0.56, respectively.

TABLE ~~2306.2.1(2)~~ 2306.2(2)
ALLOWABLE SHEAR VALUES (POUNDS PER FOOT) FOR WOOD STRUCTURAL PANEL BLOCKED DIAPHRAGMS UTILIZING MULTIPLE ROWS OF ~~FASTENERS~~ STAPLES (HIGH LOAD DIAPHRAGMS) WITH FRAMING OF DOUGLAS FIR, LARCH, OR SOUTHERN PINE[a] FOR WIND OR SEISMIC LOADING[b, g, h]

PANEL GRADE[c]	~~COMMON NAIL SIZE OR~~ STAPLE[f] GAGE	MINIMUM FASTENER PENETRATION IN FRAMING (inches)	MINIMUM NOMINAL PANEL THICKNESS (inch)	MINIMUM NOMINAL WIDTH OF FRAMING MEMBER AT ADJOINING PANEL EDGES AND BOUNDARIES[e] (inches)	LINES OF FASTENERS	BLOCKED DIAPHRAGMS Cases 1 and 2[d] Fastener Spacing Per Line at Boundaries (inches)					
						4		2 1/2		2	
						Fastener Spacing Per Line at Other Panel Edges (inches)					
						6	4	4	3	3	2
Structural I grades	~~10d common nails~~	~~1 ½~~	~~15/32~~	~~3~~	~~2~~	~~605~~	~~815~~	~~875~~	~~1,150~~	-	-
				~~4~~	~~2~~	~~700~~	~~915~~	~~1,005~~	~~1,290~~	-	-
				~~4~~	~~3~~	~~875~~	~~1,220~~	~~1,285~~	~~1,395~~	-	-
			~~19/32~~	~~3~~	~~2~~	~~670~~	~~880~~	~~965~~	~~1,255~~	-	-
				~~4~~	~~2~~	~~780~~	~~990~~	~~1,110~~	~~1,440~~	-	-
				~~4~~	~~3~~	~~965~~	~~1,320~~	~~1,405~~	~~1,790~~	-	-
			~~23/32~~	~~3~~	~~2~~	~~730~~	~~955~~	~~1,050~~	~~1,365~~	-	-
				~~4~~	~~2~~	~~855~~	~~1,070~~	~~1,210~~	~~1,565~~	-	-
				~~4~~	~~3~~	~~1,050~~	~~1,430~~	~~1,525~~	~~1,800~~	-	-
	14 gage staples	2	15/32	3	2	600	600	860	960	1,060	1,200
				4	3	860	900	1,160	1,295	1,295	1,400
			19/32	3	2	600	600	875	960	1,075	1,200
				4	3	875	900	1,175	1,440	1,475	1,795
Sheathing, single floor and other grades covered in DOC PS1 and PS2	~~10d common nails~~	~~1 ½~~	~~15/32~~	~~3~~	~~2~~	~~525~~	~~725~~	~~765~~	~~1,010~~	-	-
				~~4~~	~~2~~	~~605~~	~~815~~	~~875~~	~~1,105~~	-	-
				~~4~~	~~3~~	~~765~~	~~1,085~~	~~1,130~~	~~1,195~~	-	-
			~~19/32~~	~~3~~	~~2~~	~~650~~	~~860~~	~~935~~	~~1,225~~	-	-
				~~4~~	~~2~~	~~755~~	~~965~~	~~1,080~~	~~1,370~~	-	-
				~~4~~	~~3~~	~~935~~	~~1,290~~	~~1,365~~	~~1,495~~	-	-
			~~23/32~~	~~3~~	~~2~~	~~710~~	~~935~~	~~1,020~~	~~1,335~~	-	-
				~~4~~	~~2~~	~~825~~	~~1,050~~	~~1,175~~	~~1,445~~	-	-
				~~4~~	~~3~~	~~1,020~~	~~1,400~~	~~1,480~~	~~1,565~~	-	-
	14 gage staples	2	15/32	3	2	540	540	735	865	915	1,080
				4	3	735	810	1,005	1,105	1,105	1,195
			19/32	3	2	600	600	865	960	1,065	1,200
				4	3	865	900	1,130	1,430	1,370	1,485
			23/32	4	3	865	900	1,130	1,490	1,430	1,545

For SI: 1 inch =- 25.4 mm.

a. For framing of other species: (1) Find specific gravity for species of framing lumber in AF&PA NDS. (2) For staples, find shear value from table above for Structural I panels (regardless of actual grade) and multiply value by 0.82 for species with specific gravity of 0.42 or greater, or 0.65 for all other species. ~~(3) For nails, find shear value from table above for nail size of actual grade and multiply value by the following adjustment factor: Specific Gravity Adjustment Factor = [1 - (0.5 - SG)], where SG = Specific gravity of the framing lumber. This adjustment factor shall not be greater than 1.~~

b. Fastening along intermediate framing members: Space fasteners a maximum of 12 inches on center, except 6 inches on center for spans greater than 32 inches.

c. Panels conforming to PS 1 or PS 2.

d. This table gives shear values for Cases 1 and 2 as shown in Table ~~2306.2.1(1)~~ 2306.2(1). The values shown are applicable to Cases 3, 4, 5 and 6 as shown in Table ~~2306.2.1(1)~~ 2306.2(1), provided fasteners at all continuous panel edges are spaced in accordance with the boundary fastener spacing.

e. The minimum nominal depth of framing members shall be 3 inches. The minimum nominal width of framing members not located at boundaries or adjoining panel edges shall be 2 inches.

f. Staples shall have a minimum crown width of 7/16 inch, and shall be installed with their crowns parallel to the long dimension of the framing members.

g. High load diaphragms shall be subject to special inspection in accordance with Section 1704.6.1.

h. For shear loads of normal or permanent load duration as defined by the AF&PA NDS, the values in the table above shall be multiplied by 0.63 or 0.56, respectively.

(No Change to Fastener Placement Figure)

4. Delete and substitute as follows:

~~**2306.3 Wood structural panel shear walls.** Wood structural panel shear walls shall be designed and constructed in accordance with AF&PA SDPWS. Wood structural panel shear walls are permitted to resist horizontal forces, using the allowable capacities set forth in Table 2306.3. Allowable capacities in Table 2306.3 are permitted to be increased 40 percent for wind design.~~

2306.3 Wood-frame shear walls. Wood-frame shear walls shall be designed and constructed in accordance with AF&PA SDPWS. Where panels are fastened to framing members with staples, requirements and limitations of AF&PA SDPWS shall be met and the allowable shear values set forth in Table 2306.3(1), 2306.3(2) or 2306.3(3) shall be permitted. The allowable shear values in Tables 2306.3(1) and 2306.3(2) are permitted to be increased 40 percent for wind design.

5. Revise as follows:

TABLE ~~2306.3~~ 2306.3(1)
ALLOWABLE SHEAR VALUES (POUNDS PER FOOT) FOR WOOD STRUCTURAL PANEL SHEAR WALLS UTILIZING STAPLES WITH FRAMING OF DOUGLAS FIR-LARCH OR SOUTHERN PINE[a] FOR WIND OR SEISMIC LOADING [b,~~h,i, j,l~~ f, g]

PANEL GRADE	MINIMUM NOMINAL PANEL THICKNESS (inch)	MINIMUM FASTENER PENETRATION IN FRAMING (inches)	PANELS APPLIED DIRECT TO FRAMING					PANELS APPLIED OVER ½" or 5/8" GYPSUM SHEATHING				
			~~Nail (common or galvanized box) or~~ staple size [~~k~~ h]	Fastener spacing at panel edges (inches)				~~Nail (common or galvanized box) or~~ staple size [~~k~~ h]	Fastener spacing at panel edges (inches)			
				6	4	3	2 [~~e~~ d]		6	4	3	2 [~~e~~ d]
Structural I Sheathing	3/8	~~1 3/8~~	~~8d (2 ½" x 0.131" common, 2 ½" x 0.113" galvanized box)~~	~~230[d]~~	~~360[d]~~	~~460[d]~~	~~610[d]~~	~~10d (3" x 0.148" common, 3" x 0.128" galvanized box)~~	~~280~~	~~430~~	~~550f~~	~~730~~
		1	1 ½ 16 Gage	155	235	315	400	2 16 Gage	155	235	310	400
	7/16	~~1 3/8~~	~~8d (2 ½" x 0.131" common, 2 ½" x 0.113" galvanized box)~~	~~255[d]~~	~~395[d]~~	~~505[d]~~	~~670[d]~~	~~10d (3" x 0.148" common, 3" x 0.128" galvanized box)~~	~~280~~	~~430~~	~~550[f]~~	~~730~~
		1	1 ½ 16 Gage	170	260	345	440	2 16 Gage	155	235	310	400
	15/32	~~1 3/8~~	~~8d (2 ½" x 0.131" common, 2 ½" x 0.113" galvanized box)~~	~~280~~	~~430~~	~~550~~	~~730~~	~~10d (3" x 0.148" common, 3" x 0.128" galvanized box)~~	~~280~~	~~430~~	~~550[f]~~	~~730~~
		1	1 ½ 16 Gage	185	280	375	475	2 16 Gage	155	235	300	400
		~~1 ½~~	~~10d (3" x 0.148" common, 3" x 0.128" galvanized box)~~	~~340~~	~~510~~	~~665[f]~~	~~870~~	~~10d (3" x 0.148" common, 3" x 0.128" galvanized box)~~	-	-	-	-
Sheathing, plywood siding [~~g~~ e] except Group 5 species	5/16[c] or 1/4[c]	~~1 ¼~~	~~6d (2"x0.113" common, 2"x0.099" galvanized box)~~	~~180~~	~~270~~	~~350~~	~~450~~	~~8d (2 ½" x 0.131" common, 2 ½" x 0.113" galvanized box)~~	~~180~~	~~270~~	~~350~~	~~450~~
		1	1 ½ 16 Gage	145	220	295	375	2 16 Gage	110	165	220	285
	3/8	~~1 ¼~~	~~6d (2"x0.113" common, 2"x0.099" galvanized box)~~	~~200~~	~~300~~	~~390~~	~~510~~	~~8d (2 ½" x 0.131" common, 2 ½" x 0.113" galvanized box)~~	~~200~~	~~300~~	~~390~~	~~510~~
		~~1 3/8~~	~~8d (2 ½" x 0.131" common, 2 ½" x 0.113" galvanized box)~~	~~220[d]~~	~~320[d]~~	~~410[d]~~	~~530[d]~~	~~10d (3" x 0.148" common, 3" x 0.128" galvanized box)~~	~~260~~	~~380~~	~~490[f]~~	~~640~~
		1	1 ½ 16 Gage	140	210	280	360	2 16 Gage	140	210	280	360
	7/16	~~1 3/8~~	~~8d (2 ½" x 0.131" common, 2 ½" x 0.113" galvanized box)~~	~~240[d]~~	~~350[d]~~	~~450[d]~~	~~585[d]~~	~~10d (3" x 0.148" common, 3" x 0.128" galvanized box)~~	~~260~~	~~380~~	~~490[f]~~	~~640~~
		1	1 ½ 16 Gage	155	230	310	395	2 16 Gage	140	210	280	360
	15/32	~~1 3/8~~	~~8d (2 ½" x 0.131" common, 2 ½" x 0.113" galvanized box)~~	~~260~~	~~380~~	~~490~~	~~640~~	~~10d (3" x 0.148" common, 3" x 0.128" galvanized box)~~	~~260~~	~~380~~	~~490[f]~~	~~640~~
		~~1 ½~~	~~10d (3" x 0.148" common, 3" x 0.128" galvanized box)~~	~~310~~	~~460~~	~~600[f]~~	~~770~~	-	-	-	-	-
		1	1 ½ 16 Gage	170	255	335	430	2 16 Gage	140	210	280	360
	19/32	~~1 ½~~	~~10d (3" x 0.148" common, 3" x 0.128" galvanized box)~~	~~340~~	~~510~~	~~665[f]~~	~~870~~	-	-	-	-	-
		1	1 3/4 16 Gage	185	280	375	475	-	-	-	-	-
			~~Nail Size (galvanized casing)~~					~~Nail Size (galvanized casing)~~				
	~~5/16[c]~~	~~1 ¼~~	~~6d (2" x 0.099")~~	~~140~~	~~210~~	~~275~~	~~360~~	~~8d (2 ½" x 0.113")~~	~~140~~	~~210~~	~~275~~	~~360~~
	~~3/8[c]~~	~~1 3/8~~	~~8d (2 ½" x 0.113")~~	~~160~~	~~240~~	~~310~~	~~410~~	~~10d (3" x 0.128")~~	~~160~~	~~240~~	~~310[f]~~	~~410~~

For SI: 1 inch =- 25.4 mm, 1 pound per foot = 14.59 N/m.

a. For framing of other species: (1) Find specific gravity for species of lumber in NDS. (2) For staples find shear value from table above for Structural I panels (regardless of actual grade) and multiply value by 0.82 for species with specific gravity of 0.42 or greater, or 0.65 for all other species. ~~(3) For nails find shear value from table above for nail size for actual grade and multiply value by the following adjustment factor: Specific Gravity Adjustment Factor = [1 - (0.5 - SG)], Where SG = Specific Gravity of the framing lumber. This adjustment factor shall not be greater than 1.~~

b. Panel edges backed with 2-inch nominal or wider framing. Install panels either horizontally or vertically. Space fasteners maximum 6 inches on center along intermediate framing members for 3/8-inch and 7/16-inch panels installed on studs spaced 24 inches on center. For other conditions and panel thickness, space fasteners maximum 12 inches on center on intermediate supports.

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

~~d. Allowable shear values are permitted to be increased to values shown for 15/32-inch sheathing with same nailing fastening provided (a) studs are spaced a maximum of 16 inches on center, or (b) if panels are applied with long dimension across studs.~~

~~e~~ d. Framing at adjoining panel edges shall be 3 inches nominal or wider~~, and nails at all panel edges shall be staggered where panel edge nailing is specified 2 inches on center or less~~.

~~f. Framing at adjoining panel edges shall be 3 inches nominal or wider, and nails at all panel edges shall be staggered where both of the following conditions are met: (1) 10d (3" x 0.148") nails having penetration into framing of more than 1-1/2 inches and (2) panel edge nailing is specified 3 inches on center or less.~~

~~g~~ e. Values apply to all-veneer plywood. Thickness at point of fastening on panel edges governs shear values.

~~h~~ f. Where panels are applied on both faces of a wall and ~~nail~~ fastener spacing is less than 6 inches o.c. on either side, panel joints shall be offset to fall on different framing members. Or framing shall be 3-inch nominal or thicker at adjoining panel edges~~ and nails at all panel edges shall be staggered~~.

~~i~~ g. In Seismic Design Category D, E or F, where shear design values exceed 350 pounds per lineal foot, all framing members receiving edge ~~nailing~~ fastening from abutting panels shall not be less than a single 3-inch nominal member, or two 2-inch nominal members fastened together in accordance with Section 2306.1 to transfer the design shear value between framing members. ~~Wood structural panel joint and sill plate nailing shall be staggered at all panel edges.~~ See ~~Sections 4.3.6.1 and 4.3.6.4.2 of~~ AF&PA SDPWS for sill plate size and anchorage requirements.

~~j. Galvanized nails shall be hot-dipped or tumbled.~~

~~k~~ h. Staples shall have a minimum crown width of 7/16 inch and shall be installed with their crowns parallel to the long dimension of the framing members.

~~l~~ i. For shear loads of normal or permanent load duration as defined by the AF&PA NDS, the values in the table above shall be multiplied by 0.63 or 0.56, respectively.

6. Delete without substitution:

~~TABLE 2306.5~~
~~ALLOWABLE SHEAR FOR PARTICLEBOARD SHEAR WALL SHEATHING~~[b]

~~PANEL GRADE~~	~~MINIMUM NOMINAL PANEL THICKNESS (inch)~~	~~MINIMUM NAIL PENETRATION IN FRAMING (inches)~~	~~PANELS APPLIED DIRECT TO FRAMING~~				
			~~Nail size (common or galvanized box)~~	~~Allowable shear (pounds per foot) nail spacing at panel edges (inches)a~~			
				~~6~~	~~4~~	~~3~~	~~2~~
~~M-S "Exterior Glue" and M-2 "Exterior Glue"~~	~~3/8~~	~~11/2~~	~~6d~~	~~120~~	~~180~~	~~230~~	~~300~~
	~~3/8~~	~~11/2~~	~~8d~~	~~130~~	~~190~~	~~240~~	~~315~~
	~~1/2~~			~~140~~	~~210~~	~~270~~	~~350~~
	~~1/2~~	~~15/8~~	~~10d~~	~~185~~	~~275~~	~~360~~	~~460~~
	~~5/8~~			~~200~~	~~305~~	~~395~~	~~520~~

7. Revise as follows:

TABLE ~~2306.6~~ 2306.3(2)
ALLOWABLE SHEAR VALUES (plf) FOR WIND OR SEISMIC LOADING ON SHEAR WALLS OF FIBERBOARD SHEATHING BOARD CONSTRUCTION UTILIZING STAPLES FOR TYPE V CONSTRUCTION ONLY [a,b,c,d,e]

THICKNESS AND GRADE	FASTENER SIZE	ALLOWABLE SHEAR VALUE (pounds per linear foot) ~~NAIL~~ STAPLE SPACING AT PANEL EDGES (inches)[a]		
		4	3	2
1/2"' or 25/32" Structural	~~No. 11 gage galvanized roofing nail 1-1/2" long for 1/2", 1¾ for 25/32" with 3/8" head~~	~~170~~	~~230~~	~~260~~
	No. 11 gage galvanized staple, 7/16" crown[f]	150	200	225
	No. 11 gage galvanized staple, 1" crown[f]	220	290	325

(No change to footnote a)

b. Panel edges shall be backed with 2 inch or wider framing of Douglas fir-larch or Southern pine. For framing of other species: (1) Find specific gravity for species of framing lumber inAF&PANDS. (2) For staples, multiply the shear value from the table above by 0.82 for species with specific gravity of 0.42 or greater, or 0.65 for all other species. ~~(3) For nails, multiply the shear value from the table above by the following adjustment factor: specific gravity adjustment factor = [1-(0.5-SG)], where SG = Specific gravity of the framing lumber.~~

(No change to footnotes c through f)

TABLE ~~2306.7~~ 2306.3(3)
ALLOWABLE SHEAR VALUES FOR WIND OR SEISMIC FORCES FOR SHEAR WALLS OF LATH AND PLASTER OR GYPSUM BOARD WOOD FRAMED WALL ASSEMBLIES UTILIZING STAPLES

TYPE OF MATERIAL	THICKNESS OF MATERIAL	WALL CONSTRUCTION	~~FASTENER~~ STAPLE SPACING[b] MAXIMUM (inches)	SHEAR VALUE [a, ~~e~~ c] (plf)	MINIMUM ~~FASTENER~~ STAPLE SIZE [~~c,d,j,k~~ f, g]
1. Expanded metal or woven wire lath and Portland cement plaster	7/8"	Unblocked	6	180	~~No. 11 gage 1 ½" long, 7/16" head~~ No. 16 gage galv. staple, 7/8" legs
~~2. Gypsum lath, plain or perforated with vertical joints staggered~~	~~3/8" lath and ½" plaster~~	~~Unblocked~~	~~5~~	~~180~~	~~No. 13 gage galv. 1 1/8" long, 19/64" head, plasterboard nail~~
3. Gypsum lath, plain or perforated	3/8" lath and 1/2" plaster	Unblocked	5	100	No. 16 gage galv. staple, 1 1/8" long, ~~0.120" nail, min. 3/8" head, 1 ¼" long~~
4. Gypsum board, gypsum veneer base or water-resistant gypsum backing board	½"	Unblocked [~~f~~ d]	7	75	~~5d cooler (1 5/8" x 0.086") or wallboard 0.120" nail, min. 3/8" head, 1 ½" long~~ No. 16 gage galv. staple, 1 ½" long
		Unblocked [~~f~~ d]	4	110	
		Unblocked	7	100	
		Unblocked	4	125	
		Blocked [~~g~~ e]	7	125	
		Blocked [~~g~~ e]	4	150	
		~~Unblocked~~	~~8/12[h]~~	~~60~~	~~No. 6 1 ½" screws[i]~~
		~~Blocked[g]~~	~~4/16[h]~~	~~160~~	
		~~Blocked[f,g]~~	~~4/12[h]~~	~~155~~	
		~~Blocked[g]~~	~~8/12[h]~~	~~70~~	
		~~Blocked[g]~~	~~6/12[h]~~	~~90~~	
	5/8"	Unblocked [~~f~~ d]	7	115	~~6d cooler (1 7/8" x 0.092") or wallboard 0.120" nail, min 3/8" head, 1 ¾" long~~ No. 16 gage galv. staple, 1 ½" legs, 1 5/8" long
			4	145	
		Blocked [~~g~~ e]	7	145	
			4	175	
		Blocked [~~g~~ e] Two-ply	Base ply: 9 Face ply: 7	250	~~Base ply-6d cooler (1 7/8" x 0.092") or wallboard 1 ¾" x 0.120" nail, min. 3/8" head 1 5/8"~~ No. 16" gage galv. staple, 1 5/8" long ~~Face ply-8d cooler (2 3/8" x 0.113") or wallboard 0.120" nail, min. 3/8" head, 2 3/8" long~~ No. 15 gage galv. staple, 2 ¼" long
		~~Unblocked~~	~~8/12[h]~~	~~70~~	~~No. 6 1 ¼" screws[i]~~
		~~Blocked~~	~~8/12[h]~~	~~90~~	

For SI: 1 inch =- 25.4 mm, 1 foot = 304.8 mm, 1 pound per foot = 14.59N/m.

a. These shear walls shall not be used to resist loads imposed by masonry or concrete walls (see ~~Section 4.1.5 of~~ AF&PA SDPWS). Values shown are for short-term loading due to wind or seismic load. Walls resisting seismic loads shall be subject to the limitations in Section 12.2.1 of ASCE 7. Values shown shall be reduced 25 percent for normal loading.

b. Applies to fastening at studs, top and bottom plates and blocking.

~~c. Alternate fasteners are permitted to be used if their dimensions are not less than the specified dimensions Drywall screws are permitted to substitute for the 5d (1-5/8" x 0.086") and 6d (1-7/8" x 0.092") (cooler) nails listed above, and No. 6-1 1/4 inch Type S or W screws for 6d (1-7/8" x 0.092") (cooler) nails.~~

~~d. For properties of cooler nails, see ASTM C 514.~~

~~e~~ c. Except as noted, shear values are based on a maximum framing spacing of 16 inches on center.

~~f~~ d. Maximum framing spacing of 24 inches on center.

~~g~~ e. All edges are blocked, and edge fastening is provided at all supports and all panel edges.

~~h. First number denotes fastener spacing at the edges; second number denotes fastener spacing at intermediate framing members.~~

~~i. Screws are Type W or S.~~

~~j~~ f. Staples shall have a minimum crown width of 7/16 inch, measured outside the legs, and shall be installed with their crowns parallel to the long dimension of the framing members.

~~k~~ g. Staples for the attachment of gypsum lath and woven-wire lath shall have a minimum crown width of 3/4 inch, measured outside the legs.

8. Delete without substitution:

~~**2306.4 Lumber sheathed shear walls.** Single and double diagonally sheathed lumber shear walls shall be designed and constructed in accordance with AF&PA SDPWS. Single and double diagonally sheathed lumber walls shall not be used to resist seismic forces in structures assigned to *Seismic Design Category* E or F.~~

~~**2306.5 Particleboard shear walls.** Particleboard shear walls shall be designed and constructed in accordance with AF&PA SDPWS. Particleboard shear walls shall be permitted to resist horizontal forces using the allowable shear capacities set forth in Table 2306.5. Allowable capacities in Table 2306.5 are permitted to be increased 40 percent for~~

~~wind design. Particleboard shall not be used to resist seismic forces in structures assigned to *Seismic Design Category* D, E or F.~~

~~**2306.6 Fiberboard shear walls.** Fiberboard shear walls shall be designed and constructed in accordance with AF&PASDPWS. Fiberboard shear walls are permitted to resist horizontal forces, using the allowable shear capacities set forth in Table 2306.6. Allowable capacities in Table 2306.6 are permitted to be increased 40 percent for wind design. Fiberboard shall not be used to resist seismic forces in structures assigned to *Seismic Design Category* D, E or F.~~

~~**2306.7 Shear walls sheathed with other materials.** Shear walls sheathed with portland cement plaster, gypsum lath, gypsum sheathing or gypsum board shall be designed and constructed in accordance with AF&PA SDPWS. Shear walls sheathed with these materials p are permitted to resist horizontal forces using the allowable shear capacities set forth in Table 2306.7. Shear walls sheathed with portland cement plaster, gypsum lath, gypsum sheathing or gypsum board shall not be used to resist seismic forces in structures assigned to *Seismic Design Category* E or F.~~

9. Revise as follows:

2307.1 Load and resistance factor design. The ~~structural analysis~~ design and construction of wood elements and structures using *load and resistance factor design* shall be in accordance with AF&PA NDS and AF&PA SDPWS.

10. Delete without substitution:

~~**2307.1.1 Wood structural panel shear walls.** In *Seismic Design Category* D, E or F, where shear design values exceed 490 pounds per foot (7154 N/m), all framing members receiving edge nailing fastening from abutting panels shall not be less than a single 3-inch (76 mm) nominal member or two 2-inch (51 mm) nominal members fastened together in accordance with AF&PA NDS to transfer the design shear value between framing members. Wood structural panel joint and sill plate nailing shall be staggered at all panel edges. See Sections 4.3.6.1 and 4.3.6.4.3 of AF&PA SDPWS for sill plate size and anchorage requirements.~~

11. Revise as follows:

2308.11.2 Concrete or masonry. Concrete or masonry walls and stone or masonry veneer shall not extend above the basement.

Exceptions:

1. Stone and masonry veneer is permitted to be used in the first two stories above grade plane or the first three stories above grade plane where the lowest story has concrete or masonry walls in Seismic Design Category B, provided that structural use panel wall bracing is used, and the length of bracing provided is 1.5 times the required length as determined in Table 2308.9.3(1).
2. Stone and masonry veneer is permitted to be used in the first story above grade plane or the first two stories above grade plane where the lowest story has concrete or masonry walls in Seismic Design Category B or C.
3. Stone and masonry veneer is permitted to be used in both stories of buildings with two stories above grade plane in Seismic Design Categories B and C provided the following criteria are met:
   - 3.1. Type of brace per Section 2308.9.3 shall be Method 3 and the allowable shear capacity in accordance with ~~Table 2306.3~~ Section 2306.3 shall be a minimum of 350 plf (5108 N/m).
   - 3.2. Braced wall panels in the second story shall be located in accordance with Section 2308.9.3 and not more than 25 feet (7620 mm) on center, and the total length of braced wall panels shall be not less than 25 percent of the braced wall line length. Braced wall panels in the first story shall be located in accordance with Section 2308.9.3 and not more than 25 feet (7620 mm) on center, and the total length of braced wall panels shall be not less than 45 percent of the braced wall line length.
   - 3.3. Hold-down connectors shall be provided at the ends of each braced wall panel for the second story to first story connection with an allowable design of 2,000 pounds (907.0 kg). Hold-down connectors shall be provided at the ends of each braced wall panel for the first story to foundation connection with an allowable capacity of 3,900 pounds (1768 kg). In all cases, the hold down connector force shall be transferred to the foundation.
   - 3.4. Cripple walls shall not be permitted.

2308.12.2 Concrete or masonry. Concrete or masonry walls and stone or masonry veneer shall not extend above a basement.

Exception: Stone and masonry veneer is permitted to be used in the first story above grade plane in Seismic Design Category D provided the following criteria are met:

1. Type of brace in accordance with Section 2308.9.3 shall be Method 3 and the allowable shear capacity in accordance with ~~Table 2306.3~~ Section 2306.3 shall be a minimum of 350 plf (5108 N/m).
2. The bracing of the first story shall be located at each end and at least every 25 feet (7620 mm) o.c. but not less than 45 percent of the braced wall line.
3. Hold-down connectors shall be provided at the ends of braced walls for the first floor to foundation with an allowable capacity of 2,100 pounds (1768 kg).
4. Cripple walls shall not be permitted.

1704.6.1 High-load diaphragms. High-load diaphragms designed in accordance with ~~Table 2306.2.1(2)~~ Section 2306.2 shall be installed with special inspections as indicated in Section 1704.1. The special inspector shall inspect the wood structural panel sheathing to ascertain whether it is of the grade and thickness shown on the approved construction documents. Additionally, the special inspector must verify the nominal size of framing members at adjoining panel edges, the nail or staple diameter and length, the number of fastener lines and that spacing between fasteners in each line and at edge margins agrees with the approved construction documents.

Reason: This proposal coordinates provisions of the IBC with those in the AF&PA consensus standard, *Special Design Provisions for Wind and Seismic* (*SDPWS*).

Item 1 implements consistent use of the term "wood-frame" to describe shear walls and diaphragms in 2305 and adds a reference to *SDPWS* for determining deflection of wood-frame diaphragms and shear walls. Item 2 removes design information for nailed diaphragms and shear walls that is duplicated in *SDPWS*. Revised Sections 2306.2 and 2306.3 clarify that design and construction as well as limitations provided in *SDPWS* are applicable to use of allowable design values for stapled diaphragm and shear wall construction. Table numbers and footnotes are editorially revised to account for removal of allowable design values for nailed diaphragms and shear walls. Item 3, replaces references to a table with a reference to Section 2306.3 to address both stapled and nailed construction. Item 4, the reference to the allowable design value table is replaced by a reference to Section 2306.2 to address both stapled and nailed construction.

Cost Impact: No known impact.

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

ICCFILENAME: FRANCIS-S4-2305.1

S209–09/10
2305.1.2 (New)

Proponent: James E. Russell, City of Palo Alto, representing self

Add new text as follows:

<u>**2305.1.2 Sill plate anchor bolts.** For sill plates of 2x or 3x nominal thickness, the allowable lateral design strength for shear parallel to 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:</u>

1. <u>The maximum anchor bolt diameter is 5/8 inches (16 mm).</u>
2. <u>The anchor bolt is cast-in-place and embedded at least 7 inches (178 mm) into concrete.</u>
3. <u>The anchor bolt is located a minimum of 2-1/2 anchor diameters from any concrete edge that is parallel to the sill plate; and</u>
4. <u>The anchor bolt is located a minimum of 15 anchor diameters from any concrete edge that is perpendicular to the sill plate.</u>

Reason: To clarify that the Section 1908.1.9 modifications to ACI 318 Section D.3.3 establishing shear capacity applicable to small diameter sill plate anchor bolts installed in concrete with certain minimum embedment, edge and end distances, permit the use of the lateral design value of the bolt attaching a wood sill plate to concrete, specified in AF&PA NDS Table 11E. The proposal that adds this information in Section 1908.1.9 specifically refers back to Section 2305, but currently Section 2305 does not contain any information about design of sill plate anchor bolts. Therefore this change is intended to make Section 2305 compatible with 1908.1.9 and to aid the code user by specifically explaining what the change in 1908.1.9 will allow.

Current design provisions require calculation of the capacity of sill plate anchor bolts using the provisions of ACI 318 Appendix D, however, those methods result in shear capacities far smaller than historical values using provisions of earlier codes and standards. Recent experiments specifically focused on this connection have revealed that the actual capacities exceed those historically used and support a return to determining the sill bolt shear capacity based upon its capacity in the wood sill plate member. The experimental data supporting this proposal and a similar proposal to change 2009 IBC Section 1908.1.9, indicate that concrete failure modes do not control the capacity of these connections when certain embedment, edge and end distances are maintained. Therefore, it is proposed that Section 2305 clearly state that the minimum design capacity be based upon the lateral design value of the bolt attaching a wood sill plate to concrete, as determined using AF&PA NDS.

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

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

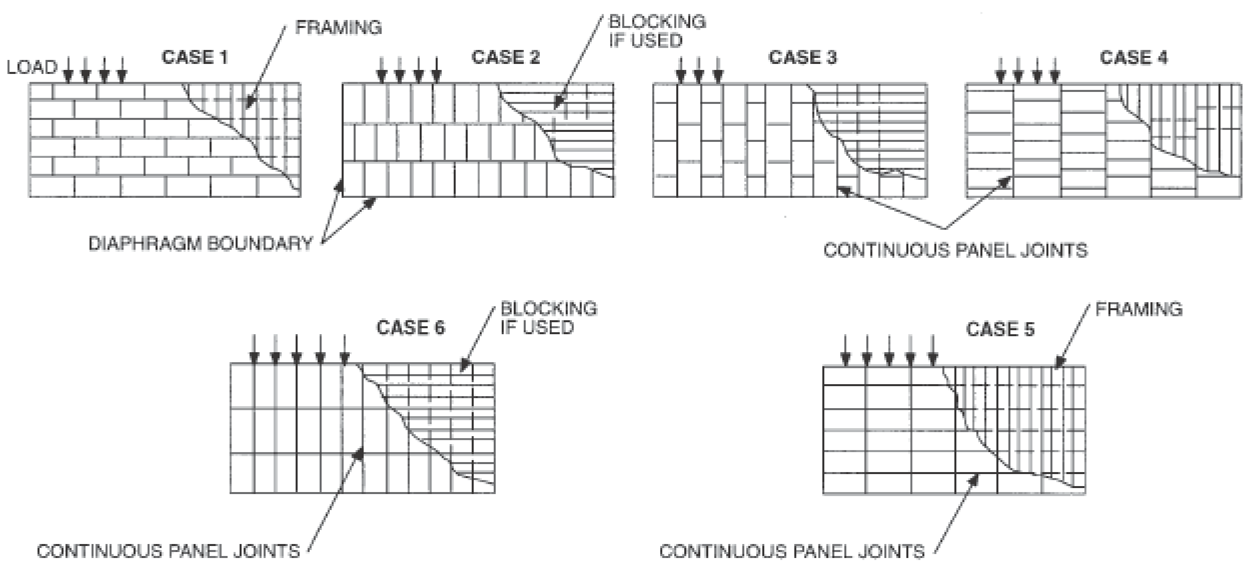
ICCFILENAME: RUSSELL-S1-2503.1.2

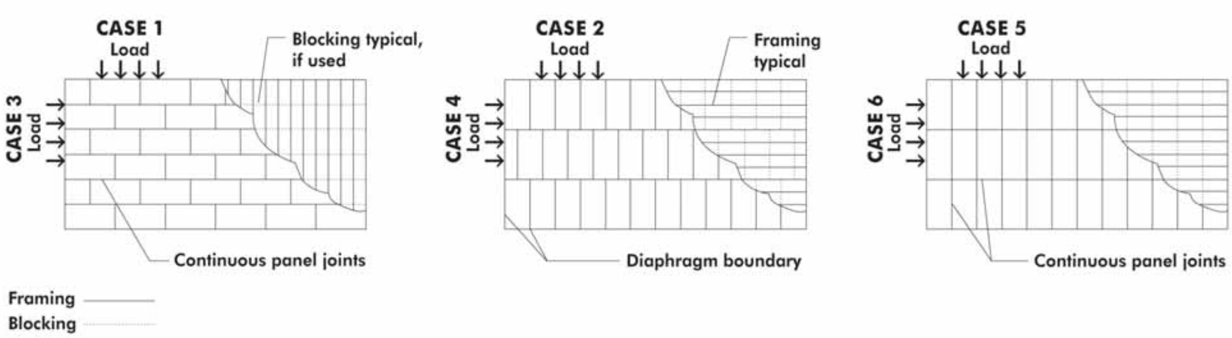
S210–09/10
Table 2306.2.1(1)

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

TABLE 2306.2.1(1)
ALLOWABLE SHEAR (POUNDS PER FOOT) FOR WOOD STRUCTURAL PANEL DIAPHRAGMS WITH FRAMING OF DOUGLAS FIR-LARCH, OR SOUTHERN PINE[a] FOR WIND OR SEISMIC LOADING[h]

Delete and substitute as follows:





Reason: The existing drawings in the 2006 Code are difficult to understand; partially because of the quality of the drawing and because of the improper placement of the annotation lines. The proposed drawing was drawn at a higher resolution to better differentiate between blocking and framing members. A legend was added to assist the user and the annotation lies were more carefully placed. This proposal makes no technical changes to the code.

As the designer is going to be concerned with a specific diaphragm geometry with two loading cases for that diaphragm (one load for each orthogonal direction), we have shown the 3 diaphragm configurations represented by the old 6 figures and added the two appropriate cases for each configuration. We were also able to increase the size of the figure to make them easier to understand. No changes have been made to the case numbers or table content. This provision just better matches the cases with the corresponding diaphragm configurations to simplify the use of the table.

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

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

ICCFILENAME: KEITH-S3-TABLE 2306.2.1(1)

S211–09/10
2308.3.2, 2308.3.2.1 (New), 2308.3.2.2 (New), 2308.12.6

Proponent: Robert Rice, Grants Pass, representing Josephine County Building Safety and Southern Oregon Chapter International Code Council.

Revise as follows:

2308.3.2 Braced wall line connections. Wind and seismic lateral forces shall be transferred from the <u>roof</u> ~~roofs~~ and <u>floor</u> ~~floors~~ <u>diaphragms</u> to braced wall lines and from the braced wall lines in upper stories to the braced wall lines in the story below in accordance with ~~this section~~ <u>Sections 2308.3.2.1 and 2308.3.2.2</u>.

<u>**2308.3.2.1 Bottom plate connection.** Braced wall line bottom plates shall be connected to joists or full-depth blocking below in accordance with Table 2304.9.1, Item 6, or to foundations in accordance with Section 2308.3.3.</u>

<u>**2308.3.2.2 Top plate connection.** Where joists or rafters are used, b</u>raced wall line top plates shall be fastened to joists, rafters or full-depth blocking above in accordance with Table 2304.9.1, Items 11, 12, 15 or 19 as applicable based on the orientation of the joists or rafters to the braced wall line. ~~Braced wall line bottom plates shall be connected to joists or blocking below in accordance with Table 2304.9.1, Item 6, or to foundations in accordance with Section 2308.3.3.~~ <u>Blocking shall be a minimum of 2 inches (51 mm) nominal in thickness and equal to the depth of the joist or rafter at the wall line and shall be fastened to the braced wall line top plate as specified in Table 2304.9.1, Item 11.</u>

> <u>**Exception:** Blocking at rafters need not be full depth when there are no braced wall lines above but shall extend to within 2 inches (51 mm) from the sheathing above.</u>

At exterior gable end walls braced wall panel sheathing in the top *story* shall be extended and fastened to roof framing where the spacing between parallel exterior braced wall lines is greater than 50 feet (15240 mm).

> ~~**Exception:**~~ Where roof trusses are used and are installed perpendicular to an exterior braced wall line, lateral forces shall be transferred from the roof diaphragm to the braced wall by blocking of the ends of the trusses or by other *approved* methods providing equivalent lateral force transfer. Blocking shall be minimum 2 inch (51 mm) nominal thickness and equal to the depth of the truss at the wall line and shall be fastened to the braced wall line top plate as specified in Table 2304.9.1, Item 11

2308.12.6 Irregular structures. *Conventional light-frame construction* shall not be used in irregular portions of structures in *Seismic Design Category* D or E. Such irregular portions of structures shall be designed to resist the forces specified in Chapter 16 to the extent such irregular features affect the performance of the conventional framing system. A portion of a structure shall be considered to be irregular where one or more of the conditions described in Items 1 through 6 below are present.

1. Where exterior braced wall panels are not in one plane vertically from the foundation to the uppermost *story* in which they are required, the structure shall be considered to be irregular [see Figure 2308.12.6(1)].

    Exception: Floors with cantilevers or setbacks not exceeding four times the nominal depth of the floor joists [see Figure 2308.12.6(2)] are permitted to support braced wall panels provided:

    1. Floor joists are 2 inches by 10 inches (51mm by 254 mm) or larger and spaced not more than 16 inches (406 mm) o.c.
    2. The ratio of the back span to the cantilever is at least 2:1.
    3. Floor joists at ends of braced wall panels are doubled.

4. A continuous rim joist is connected to the ends of cantilevered joists. The rim joist is less than 0.058 inch (1.47 mm) (16 galvanized gage) and 11/2 inches (38 mm) wide fastened with six 16d common nails on each side. The metal tie shall have a minimum yield of 33,000 psi (227 MPa).
5. Joists at setbacks or the end of cantilevered joists shall not carry gravity loads from more than a single *story* having uniform wall and roof loads, nor carry the reactions from headers having a span of 8 feet (2438 mm) or more.

2. Where a section of floor or roof is not laterally supported by braced wall lines on all edges <u>and connected in accordance with Section 2308.3.2,</u> the structure shall be considered to be irregular [see Figure 2308.12.6(3)].

   Exception: Portions of roofs or floors that do not support braced wall panels above are permitted to extend up to 6 feet (1829 mm) beyond a braced wall line [see Figure 2308.12.6(4)] <u>provided that the framing members are connected to the braced wall line below in accordance with Section 2308.3.2.</u>

3. Where the end of a required braced wall panel extends more than 1 foot (305 mm)over an opening in the wall below, the structure shall be considered to be irregular. This requirement is applicable to braced wall panels offset in plane and to braced wall panels offset out of plane as permitted by the exception to Item 1 above in this section [see Figure 2308.12.6(5)].

   Exception: Braced wall panels are permitted to extend over an opening not more than 8 feet (2438 mm) in width where the header is a 4-inch by 12-inch (102 mm by 305 mm) or larger member.

4. Where portions of a floor level are vertically offset such that the framing members on either side of the offset cannot be lapped or tied together in an *approved* manner, the structure shall be considered to be irregular [see Figure 2308.12.6(6)].

   Exception: Framing supported directly by foundations need not be lapped or tied directly together.

5. Where braced wall lines are not perpendicular to each other, the structure shall be considered to be irregular [see Figure 2308.12.6(7)].

6. Where openings in floor and roof diaphragms having a maximum dimension greater than 50 percent of the distance between lines of bracing or an area greater than 25 percent of the area between orthogonal pairs of braced wall lines are present, the structure shall be considered to be irregular [see Figure 2308.12.6(8)].

Reason: This code section addresses the connection of braced wall lines to framing above and below to transfer lateral (wind and seismic) forces into the roof and floor diaphragms. This proposal does not add any new requirements. First, in Section 2308.3.2, this proposal separates the top plate connection requirements from the bottom plate connections for clarity. Secondly, in section 2308.12.6, a reference is added to point to the connection requirements in 2308.3.2.

Purpose: As currently written, the text of the code combines top plate and bottom plate connections in the same paragraph. Top plate connection requirements at roofs and ceilings are typically different than connections to floors above. At roofs, rafters or trusses are used and pose different challenges as opposed to flat floor joists. This proposal is intended to make the section read more clearly as well as arrange it to work with another proposal revising this section that will provide prescriptive solutions for connections at the top plate to the roof diaphragm when full-depth, solid blocking will not work or is impractical.

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

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

ICCFILENAME: RICE-S1-2308.3.2

S212–09/10

2308.3.2, Figure 2308.3.2(1) (New), Figure 2308.3.2(2) (New)

Proponent: Robert Rice, Grants Pass, OR, representing Josephine County Building Safety and Southern Oregon Chapter International Code Council.

Revise as follows:

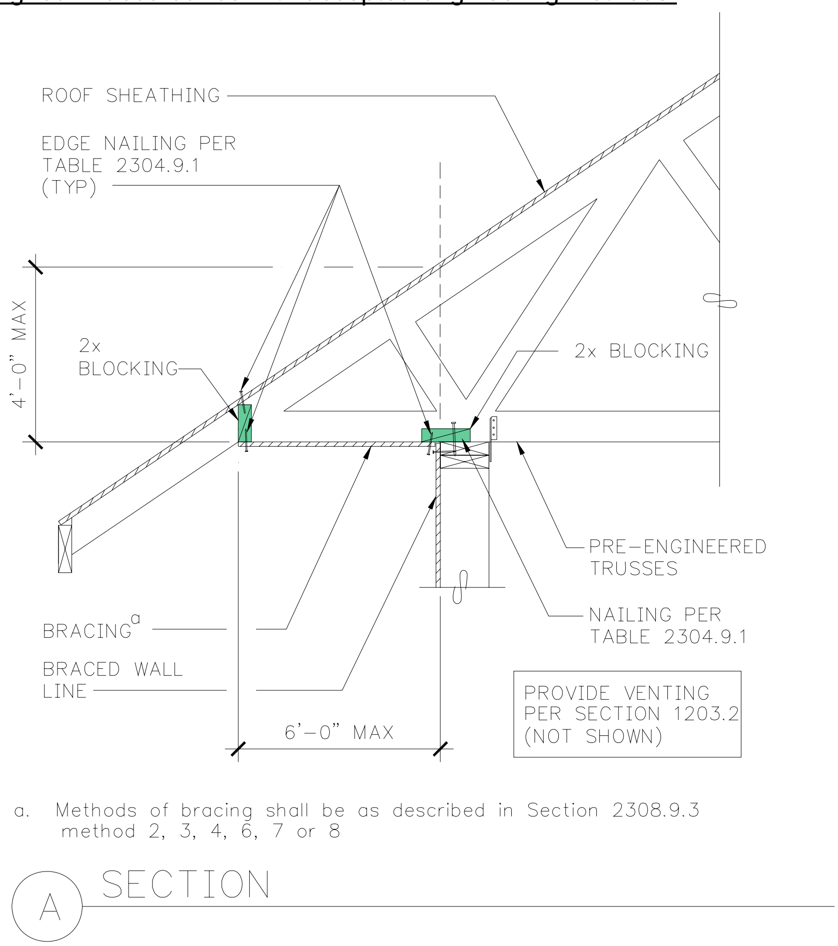
2308.3.2 Braced wall line connections. Wind and seismic lateral forces shall be transferred from the roofs and floors to braced wall lines and from the braced wall lines in upper stories to the braced wall lines in the story below in accordance with is section.

Braced wall line top plates shall be fastened to joists, rafters or full-depth blocking above in accordance with Table 2304.9.1, Items 11, 12, 15 or 19 as applicable based on the orientation of the joists or rafters to the braced wall line. Braced wall line bottom plates shall be connected to joists or blocking below in accordance with Table 2304.9.1, Item 6, or to foundations in accordance with Section 2308.3.3. At exterior gable end walls, braced wall panel sheathing in the top story shall be extended and fastened to roof framing where the spacing between parallel exterior braced wall lines is greater than 50 feet (15 240 mm).

~~**Exception:**~~ Where roof trusses are used and are installed perpendicular to an exterior braced wall line, lateral forces shall be transferred from the roof diaphragm to the braced wall by blocking of the ends of the trusses or by other approved methods providing equivalent lateral force transfer. Blocking shall be a minimum of 2 inches (51 mm) nominal in thickness and equal to the depth of the truss at the wall line and shall be fastened to the braced wall line top plate as specified in Table 2304.9.1, Item 11.

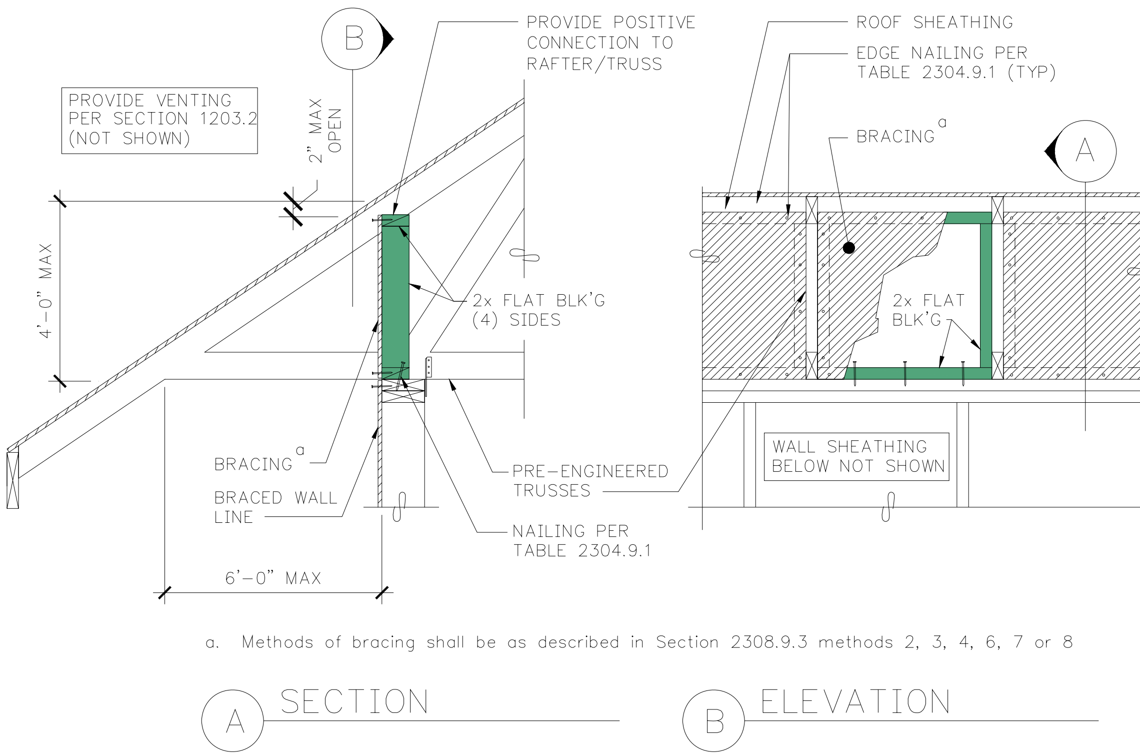
Exceptions:

1. For buildings that are classified as Seismic Design Category A, B or C and the basic wind speed is less than 100 mph (45 m/s) where the framing members are perpendicular to the wall line below and the distance from the top plate to the sheathing above is less than 9 1/4 inches (235 mm) solid blocking need not be provided when the perpendicular framing members or a parallel member such as a continuous rim joist or header is attached to the wall line in accordance with Table 2304.9.1.
2. Where the roof sheathing is greater than 9-1/4 inches (235 mm) above the top plate solid blocking is not required when the framing members are connected in accordance with one of the following methods:
   2.1 In accordance with Figure 2308.3.2 (1)
   2.2 In accordance with Figure 2308.3.2 (2)
   2.3 With full height engineered blocking panels designed for values listed in AF&PA WFCM.
   2.4 Designed in accordance with accepted engineering methods.



For SI: 1 inch = 25.4 mm

FIGURE 2308.3.2 (1)
BRACED WALL PANEL TOP PLATE CONNECTION



For SI: 1 inch = 25.4 mm

FIGURE 2308.3.2 (2)
BRACED WALL PANEL TOP PLATE CONNECTION

Reason: The 2006 IBC had fairly clear wording that the diaphragms need to be connected to the braced wall lines. With the approval of proposal 2008/2009 S224 the 2009 language was modified to make the purpose even more clear in that the connection is required to resist wind and seismic (lateral) forces. This proposal merely provides prescriptive methods to accomplish the connection whether with solid blocking or when solid blocking doesn't work.

In addition, another proposal that I have submitted rearranges the existing section to separate top plate connections from bottom plate connections since roof connections at the top plate differ from conditions where there is floor framing above. The two proposals are intended to work together and are shown at the end of the purpose statement combined as one.

Purpose: The current text of the IBC states the intention of connecting the braced wall line to the roof or floor diaphragm above in section 2308.3.2. A similar version of this proposal was adopted as an Oregon amendment for the adoption of the 2006 IBC (and the recent adoption of the 2009 IBC) and has worked well. Since then, countless hours have gone into developing proposals for both the IRC and the IBC in the 2009 code development process. The proposal for the IRC (which was the main focus) was successful and was approved for the 2009 IRC. The details for that proposal are the same ones submitted for this proposal. During the process of resolving opposition and developing a consensus two main changes were made to the proposals. First, based on engineering reports and historical data, an exception was made for low heel connections (9 ¼") in lower wind and seismic zones to not require the blocking. Second, the details for the high-heel blocking was modified to allow a 2" gap at the top to allow for venting (again, backed up by engineering data). Following the approval for the 2009 IRC an article was published in the Spring 2009 issue of *Wood Design Focus* addressing the issue. The article, "*When is Roof Eave Blocking Required?", states, Because the 2006 IRC lacks clarity on when roof eave blocking is required for lateral force transfer, IRC users and code officials are forced to interpret its intent on a case by case basis, often with varied results." "Fortunately, Section R602.10.6.2 of the 2009 IRC provides a reasonable solution that addresses the above concerns, places reasonable limits on past successful practices, and avoids the pitfalls of the 2006 IRC…..".*

This proposal does not add additional requirements to the code. This proposal clarifies that the connection needs to occur and provides prescriptive solutions when solid blocking in not possible or is impractical.

Per accepted engineering practice for lateral design loads, the floor and roof diaphragms transmit wind and seismic loads into the braced walls (engineered shearwalls or prescriptive braced panels). The fact that the diaphragm needs to be connected to the braced wall line is often not fully understood by plans examiners, inspectors and contractors. The typical requirement that is intended by the code is that solid blocking occur at this connection with the blocking connected to the top plate of the wall to transfer the diaphragm (plf) force to the wall top plates. This is evidenced in the IBC by the exception to irregular structures stating, *"..lateral forces shall be transferred from the roof diaphragm to the braced wall by blocking of the ends of the trusses.."*. In order for the forces to be transferred there has to be a connection capable of transferring the diaphragm shear evenly to the top plates.

The condition that occurs at an increasing rate that brings this issue up is with cantilevered or high stub-heel trusses. In that construction method solid blocking (either with 2x or engineered wood products) is often not possible due to the height of the diaphragm above the top plate of the wall.

Without this clarification of the text it is a connection that may or may not occur based on what I have seen in the field and have discussed with code officials. The blocking that is called for in the code serves three functions. It provides closure to prevent animals, birds, etc. from entering the attic space, it prevents the trusses or rafters from "rolling over" and it transfers the diaphragm forces to the wall. Most code officials, inspectors and contractors understand the first two objectives. However, the latter is a concept that is often not fully understood. This needs to be perceived, understood and implemented in a uniform way.

In addition, rather than identify a problem without providing a solution, my proposal includes two details to accomplish this connection simply. The solutions are, in principle, fundamentally extending the braced wall sheathing to the roof diaphragm either vertically in the truss bays or horizontally through the soffit. No engineering or testing is required since it is just completing the load path with the already defined sheathing and nailing.

Without prescriptive provisions in the current code this condition would require engineering or, as stated in 2308.3.2, Exception to item 1 "..by other approved methods." would be left up to the Authority Having Jurisdiction to determine what is acceptable without any guidance or uniformity between jurisdictions.

Typically, the engineering solution would provide details similar to those included in this proposal. Therefore, the solution and construction costs would not change. Costs would be reduced by eliminating additional costs for engineering where these prescriptive solutions work.

If approved, the two proposals I have submitted for section 2308.3.2 would read as shown below when combined:

2308.3.2 Braced wall line connections. Wind and seismic lateral forces shall be transferred from the roof and floor diaphragms to braced wall lines and from the braced wall lines in upper stories to the braced wall lines in the story below in accordance with this sections 2308.3.2.1 and 2308.3.2.2.
2308.3.2.1 Bottom plate connection. Braced wall line bottom plates shall be connected to joists or full depth blocking below in accordance with Table 2304.9.1, Item 6, or to foundations in accordance with Section 2308.3.3.
2308.3.2.2 Top plate connection. Where joists or rafters are used, braced wall line top plates shall be fastened to joists, rafters or full-depth blocking above in accordance with Table 2304.9.1, Items 11, 12, 15 or 19 as applicable based on the orientation of the joists or rafters to the braced wall line. Blocking shall be a minimum of 2 inches (51 mm) nominal in thickness and equal to the depth of the truss at the wall line and shall be fastened to the braced wall line top plate as specified in Table 2304.9.1, Item 11.

Exception: Blocking at rafters need not be full depth when there are no braced wall lines above but shall extend to within 2 inches (51mm) from the sheathing above.

At exterior gable end walls, braced wall panel sheathing in the top story shall be extended and fastened to roof framing where the spacing between parallel exterior braced wall lines is greater than 50 feet (15 240 mm).

Where roof trusses are used and are installed perpendicular to an exterior braced wall line, lateral forces shall be transferred from the roof diaphragm to the braced wall by blocking of the ends of the trusses or by other approved methods providing equivalent lateral force transfer. Blocking shall be a minimum of 2 inches (51 mm) nominal in thickness and equal to the depth of the truss at the wall line and shall be fastened to the braced wall line top plate as specified in Table 2304.9.1, Item 11.

Exceptions:

1. For Seismic Design Categories C and less and wind speed zones less than 100 mph where the rafters, joists or trusses are perpendicular to the wall line below and the distance from the top plate is less than 9 ¼ inches (235 mm) solid blocking need not be provided when the perpendicular framing members or a parallel member such as a continuous rim joist or header is attached to the wall line per Table 2304.9.1
2. Where the roof sheathing is greater than 9-1/4 inches (235 mm) above the top plate solid blocking is not required when the rafters, joists or trusses are connected in accordance with one of the following methods:
    1. In accordance with Figure 2308.3.2 (1)
    2. In accordance with Figure 2308.3.2 (2)
    3. With full height engineered blocking panels designed for values listed in American Forest and Paper Association (AF&PA) Wood Frame Construction Manual for One- and Two-Family Dwellings (WFCM).
    4. Designed in accordance with accepted engineering methods.

Bibliography: "When is Roof Eave Blocking Required*", by Jay H. Crandell, P.E., Robert Rice, Brian Foley, P.E., and Frank Woeste, PhD, P.E Volume 19, Number 1, Spring 2009 Wood Design Focus, a quarterly publication of Forest Products Society, Madison WI..*

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

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

ICCFILENAME: RICE-S2-2308.3.2

S213–09/10
2308.9.2.3

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

Revise as follows:

2308.9.2.3 Nonbearing walls and partitions. In nonbearing walls and partitions, studs shall be spaced not more than ~~28~~ 24 inches (~~711~~ 609 mm) o.c. and in interior nonbearing walls and partitions, are permitted to be set with the long dimension parallel to the wall. Interior nonbearing partitions shall be capped with no less than a single top plate installed to provide overlapping at corners and at intersections with other walls and partitions. The plate shall be continuously tied at joints by solid blocking at least 16 inches (406 mm) in length and equal in size to the plate or by 1/2-inch by 11/2-inch (12.7 mm by 38 mm) metal ties with spliced sections fastened with two 16d nails on each side of the joint.

Reason: The ICC Structural Committee liked the idea of Code Change Proposal S228-07/08 but thought it was unclear. NCSEA was not the author of S228-07/08, but is now proposing a change to this section to address what we see as a potential safety concern for wind loading. Section 2308.9.2.3 allows 2x studs to be placed flat wise in a wall and be spaced at up to 28" oc. Table 2308.9.1 limits the height of edge wise studs in such a wall to 14 feet for 2x4 nonbearing walls, for example. Our Code Change Proposal is aimed at limiting this construction to interior walls. Tall flat wise stud construction is not appropriate for exterior walls which are subject to wind loads.

We are also recommending that the 28" spacing in Section 2308.9.2.3 should be changed to 24" oc. Table 2308.9.1 limits the maximum spacing of edge wise studs in all non-bearing walls to 24". Turning the stud and using it flat wise in the wall, should not let the stud spacing increase. We also note that in modern construction almost all wall framing is based on modules which fit within dimensions of 48" or 96". A spacing of 24" oc is a module of 48" and 96" but a spacing of 28" oc is not.

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

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

ICCFILENAME: HUSTON-S1-2308.9.2.3

S214–09/10
2308.9.4; IRC R602.9

Proponent: Robert Rice, Grants Pass, representing Josephine County Building Safety and Southern Oregon Chapter International Code Council.

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC-STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

2308.9.4 Cripple walls. Foundation cripple walls shall ~~be framed of studs~~ not <u>be</u> less in size than the <u>required width of the</u> studding above with a minimum length of 14 inches (356 mm), or <u>the wall</u> shall be framed of solid blocking <u>or other approved method to prevent the studs from splitting</u>. Where exceeding 4 feet (1219 mm) in height, such walls shall be framed of studs having the size required for an additional story.

PART II – IRC BUILDING/ENERGY

Revise as follows:

R602.9 Cripple Walls. Foundation cripple walls shall be framed of studs not smaller than <u>required size of</u> the studding above. When exceeding 4 feet (1219 mm) in height, such walls shall be framed of studs having the size required for an additional story.

Cripple walls with a stud height less than 14 inches (356 mm) shall be sheathed on at least one side with a wood structural panel that is fastened to both the top and bottom plates in accordance with Table R602.3(1) <u>or other approved method to prevent studs from splitting</u> or the cripple walls shall be constructed of solid blocking. Cripple walls shall be <u>braced as required for lateral loads per section R602.10.2 and R602.10.11.4 and</u> supported on continuous foundations~~.~~

Reason: There are situations where the wall above is of studs larger than what would be required for structural reasons. In some cases it is to accommodate increased insulation or for tall walls. Typically, a 2x4 cripple wall is structurally sufficient even though the wall above may be 2x 6 for insulation reasons or 2x 8 for tall studs. The words "..required width…" would clear this up.

Regarding the 14" studs, this code section has been modified in the past by Oregon amendment and perhaps been misunderstood by others. The purpose for "Cripple walls with a stud height less than 14 inches…", to be sheathed does not relate to lateral bracing as the Oregon amendment implies,

For example, the Oregon amendment reads as follows:

> *Cripple walls with a stud height less than 14 inches (356 mm)* ***<u>supporting exterior walls or an interior braced wall line which is supported by a continuous foundation as required by Section 602.10.9</u>*** *shall be sheathed on at least one side with a wood structural panel that is fastened to both the top and bottom plates in accordance with Table R602.3(1), or the cripple walls shall be constructed of solid blocking.*

The intention of this code requirement is to ensure structural stability of walls with studs that are short enough to be susceptible to splitting. The 14" limit is due to the fact that, historically, up to 14" dimensional lumber was available to be used as solid blocking in lieu of the short studs. In addition, the proposal states, "or other approved method" since mechanical anchors are currently available that would allow the studs to be attached to the top and bottom plate without damaging the studs.

With the provision contained in R602.9, studs shorter than 14" can be used as long as sheathing is placed on one side of the wall to maintain the integrity of the studs and plates. The text continues, "...or the cripple walls shall be constructed of solid blocking." which would allow a number of products, now available, to be used such as glue-laminated beams (GLB), laminated veneer lumber beams (LVL) or dimensional lumber such as 4x's and 6x's.

The IRC commentary states, "The minimum length of 14 inches for cripple wall studs provides sufficient clear space for required nailing of the framing". The IBC commentary states, "The minimum stud length of 14 inches is based on the length necessary to properly fasten the studs to the foundation wall plate and the double plate above."

In addition, "Section R602.9 Cripple Walls" appears in the wall "framing" portion of the code. Wall "bracing" begins to be addressed in section R602.10. In Section R602.10.2 "Cripple wall bracing" is addressed specifically.

In summary, section "R602.9 Cripple Walls" has nothing to do with lateral bracing.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: RICE-S-2308.9.4

S215–09/10
2308.12.4, Table 2308.12.4

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

Revise as follows:

2308.12.4 Braced wall line sheathing. Braced wall lines shall be braced by one of the types of sheathing prescribed by Table 2308.12.4 as shown in Figure 2308.9.3. The sum of lengths of braced wall panels at each braced wall line shall conform to <u>the required percentage of wall length required to be braced per braced wall line in </u>Table 2308.12.4. Braced wall panels shall be distributed along the length of the braced wall line and start at not more than 8 feet (2438 mm) from each end of the braced wall line. Panel sheathing joints shall occur over studs or blocking. Sheathing shall be fastened to studs, top and bottom plates and at panel edges occurring over blocking. Wall framing to which sheathing used for bracing is applied shall be nominal 2 inch wide [actual 11/2 inch (38 mm)] or larger members.

Cripple walls having a stud height exceeding 14 inches (356 mm) shall be considered a story for the purpose of this section and shall be braced as required for braced wall lines in accordance with <u>the required percentage of wall length required to be braced per braced wall line in </u>Table 2308.12.4. Where interior braced wall lines occur without a continuous foundation below, the length of parallel exterior cripple wall bracing shall be one and one-half times the lengths required by Table 2308.12.4. Where the cripple wall sheathing type used is Type S-W and this additional length of bracing cannot be provided, the capacity of Type S-W sheathing shall be increased by reducing the spacing of fasteners along the perimeter of each piece of sheathing to 4 inches (102 mm) o.c.

TABLE 2308.12.4
WALL BRACING IN SEISMIC DESIGN CATEGORIES D AND E
(Minimum <u>Percentage</u> ~~Length~~ of Wall Bracing per each ~~25 Linear Feet of~~ Braced Wall Line[a])

Condition	SHEATHING TYPE[b]	$S_{DS} < 0.50$	$0.50 \le S_{DS} < 0.75$	$0.50 \le S_{DS} \le 1.00$	$S_{DS} > 1.00$
One Story	G-P[c]	~~10 feet 8 inches~~ <u>43 %</u>	~~14 feet 8 inches~~ <u>59 %</u>	~~18 feet 8 inches~~ <u>75 %</u>	~~25 feet 0 inches~~ <u>100 %</u>
	S-W	~~5 feet 4 inches~~ <u>21 %</u>	~~8 feet 0 inches~~ <u>32 %</u>	~~9 feet 4 inches~~ <u>37 %</u>	~~12 feet 0 inches~~ <u>48 %</u>

(No change to footnotes)

Reason: This proposed change is to add clarity to the IBC. This is necessary to account for real world cases like a 20 ft by 20 ft building or a 40 ft by 40 ft building. This will also assist both the code official and designer to better visualize what the expected wall bracing pattern will appear to be as a percentage by showing the percentage of the wall that is required to be solid.

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

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

ICCFILENAME: FATTAH-S1-2308.12.4

S216–09/10
Table 2308.12.4, 2308.12.4.1 (New)

Proponent: Gregory Mahoney, City of Davis Community Development Department, representing Sacramento Valley Association of Building Officials

1. Add new text as follows:

2308.12.4.1 Alternative bracing. An alternate braced wall panel constructed in accordance with Section 2308.9.3.1 or 2308.9.3.2 is permitted to be substituted for a braced wall panel in Section 2308.9.3 Items 2 through 8. For methods 2, 3, 4, 6, 7 and 8 each 48 inch (1219 mm) section or portion thereof required by Table 2308.12.4 is permitted to be replaced by one alternate braced wall panel constructed in accordance with Section 2308.9.3.1 or 2308.9.3.2. For method 5 each 96 inch (2438 mm) section (applied to one face) or 48 inch (1219 mm) section (applied to both faces) or portion thereof required by Table 2308.12.4 is permitted to be replaced by one alternate braced wall panel constructed in accordance with Section 2308.9.3.1 or 2308.9.3.2.

2. Revise as follows:

TABLE 2308.12.4
WALL BRACING IN SESIMIC DESIGN CATEGORIES D AND E
(Minimum Length of Wall Bracing per each 25 Linear Feet of Braced Wall Line[a])
(No change to table entries

a. Minimum length of panel bracing of one face of the wall for S-W sheathing or both faces of the wall for G-P sheathing; h/w ratio shall not exceed 2:1. For S-W panel bracing of the same material on two faces of the wall, the minimum length is permitted to be one-half the tabulated value but the h/w ratio shall not exceed 2:1 and design for uplift is required. The 2:1 h/w ratio limitation does not apply to alternate braced wall panels constructed in accordance with Section 2308.9.3.1 or 2308.9.3.2.

(No change to footnotes b and c)

Reason: Sections 2308.9.3.1 and 2308.9.3.2 provide for the substitution of braced wall panels listed in 2308.9.3 with Alternative bracing (2308.9.3.1) or Alternate bracing wall panels (2308.9.3.2). Section 2308.12 specifies additional requirements governing the use of braced wall panels constructed in accordance with 2308.9.3 in Seismic Design Categories D and E. The code is not clear regarding the use of alternate braced wall panels constructed in accordance with 2308.9.3.1 or 2308.9.3.2 in Seismic Design Categories D and E.

The purpose of this addition is to clarify that alternate braced wall panels may be substituted for braced wall panels in Seismic Design Categories D and E. In addition the proposed code addition further clarifies that an alternate brace wall panel is equal to one section of brace wall panel or portion thereof when used in Seismic Design Categories D and E.

So if Table 2308.12.4 requires 5'4" of S-W braced wall panel then that section of wall could be replaced by 2 alternate braced wall panels. If Table 2308.12.4 required 8' of S-W braced wall panels then the section of wall could also be replaced by 2 alternate braced wall panels.

The second part of the proposed change is a clarification of Footnote a to Table 2308.12.4. Footnote a to Table 2308.12.4 indicates that the h/w ratio shall not exceed 2:1. Table 2308.12.4 is based on Table 12.4-2 of the 2003 NEHRP Provisions. The NEHRP commentary suggests that the primary concern in limiting the height-to-width is to limit the overturning demand on braced wall panels due to the lack of overturning restraint. However, the NEHRP provisions do not include or address alternative braced wall panels. Since alternate braced wall panels incorporate overturning restraint devices, they should not be subject to the 2:1 limit.

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

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

ICCFILENAME: MAHONEY-S1-T2308.12.4

S217–09/10
2406.1

Proponent: Don Davies representing Utah Chapter of ICC

Revise as follows:

2406.1 Human impact loads. Individual glazed areas, including glass mirrors, in hazardous locations as defined in Section 2406.4 shall comply with Sections 2406.1.1 through 2406.1.4. Wired glass is not permitted in any of the locations where safety glazing is required in Section 2406.4.

Reason: The process of removing wired glazing from hazardous locations has taken several years and has involved changes to both sections 715.5 and 2406.1. Some confusion in Section 715 has been cleared up in the 2009 I.B.C. in sections 715.4.7.4 and 715.5.3 which both reference to chapter 24 for safety glazing. The bottom line is that wired glass is not allowed in those locations. When one turns to chapter 24 it is still unclear to the first time user of the code that wired glass is no longer allowed when a fire-protection rating is required. In the 2006 commentary to I.B.C.

Section 715.4.6.4 second paragraph on page 7-94 it states "Code users should be very aware of the change that first occurred in Chapter 24 of the 2006 code. Earlier editions of the code permitted wired glass to meet a lower level of impact resistance". Changes to Chapter 24 relating to this issue even go back to the 2003 edition of the I.B.C. where only Group E occupancies were identified as locations where wired glass could not be used in Section 2406.1.2. In the 2006 code the reference to wired glass in 2406.1 has been removed leading one to surmise that it is no longer addressed in the code at that location. The current code user shouldn't have to refer to two previous editions of the code to determine how the current code addresses an issue. The current code should stand on its own and be a straight forward document stating what is allowed and what is not. The convoluted path listing test and standards mean nothing to the typical code user. This item is too important to miss due to an oversight.

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

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

ICCFILENAME: Davies-S1-2406.1

S218–09/10

2406.1, 2106.4, 2406.4.1, 2406.4.2 (New), 2406.4.3 (New), 2406.4.4 (New), 2406.4.5 (New), 2406.4.6 (New), 2406.4.7 (New); IRC R308.4, R308.4.1 (New), R308.4.2 (New), R308.4.3 (New), R308.4.4 (New), R308.4.5 (New), R308.4.6 (New), R308.4.7 (New)

Proponent: Roger R. Evans, Park City Municipal Corporation, representing Utah Chapter of Building Officials

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

2406.1 Human impact loads. Individual glazed areas, including glass mirrors, in hazardous locations as defined in Section 2406.4 shall comply with Sections 2406.1.1 through 2406.1.4.

> **Exception:** Mirrors and other glass panels mounted or hung on a surface that provides a continuous backing support.

2406.4 Hazardous locations. The ~~following~~ locations specified in Sections 2306.4.1 through 2406.4.7 shall be considered specific hazardous locations requiring safety glazing materials.~~:~~

1. ~~Glazing in swinging doors except jalousies (see Section 2406.4.1).~~
2. ~~Glazing in fixed and sliding panels of sliding door assemblies and panels in sliding and bifold closet door assemblies.~~
3. ~~Glazing in storm doors.~~
4. ~~Glazing in unframed swinging doors.~~
5. ~~Glazing in doors and enclosures for hot tubs, whirlpools, saunas, steam rooms, bathtubs and showers. Glazing in any portion of a building wall enclosing these compartments where the bottom exposed edge of the glazing is less than 60 inches (1524 mm) above a standing surface.~~

2406.4.1 Glazing in doors. Glazing in all fixed and operable panels of swinging, sliding, and bifold doors shall be considered a hazardous location.

Exceptions:

1. Glazed openings of a size through which a 3-inch diameter (76 mm) sphere is unable to pass.
2. Decorative glazing.
3. Glazing materials used as curved glazed panels in revolving doors.
4. Commercial refrigerated cabinet glazed doors.

~~6.~~ **2406.4.2 Glazing adjacent doors**. Glazing in an individual fixed or operable panel adjacent to a door where the nearest ~~exposed~~ vertical edge of the glazing is within a 24-inch (610 mm) arc of either vertical edge of the door in a closed position and where the bottom exposed edge of the glazing is less than 60 inches (1524 mm) above the walking surface shall be considered a hazardous location.

Exceptions:

1. Decorative glazing.
2. ~~Panels where~~ When there is an intervening wall or other permanent barrier between the door and glazing.
3. Where access through the door is to a closet or storage area 3 feet (914 mm) or less in depth. Glazing in this application shall comply with Section ~~2406.4, Item 7~~ 2406.4.3.
4. Glazing in walls ~~perpendicular to the plane of the door in a closed position, other than the wall towards which the door swings when opened,~~ on the latch side of and perpendicular to the plane of the door in a closed position in one- and two-family *dwellings* or within *dwelling units* in Group R-2.
5. Glazing that is adjacent to the fixed panel of patio doors.

~~7.~~ **2406.4.3 Glazing in windows.** Glazing in an individual fixed or operable panel~~, other than in those locations described in preceding Items 5 and 6, which~~ that, meets all of the following conditions shall be considered a hazardous location:

~~7.~~1. ~~Exposed~~ The exposed area of an individual pane is greater than 9 square feet (0.84 m^2);
~~7.~~2. ~~Exposed~~ The bottom edge of the glazing is less than 18 inches (457 mm) above the floor;
~~7.~~3. ~~Exposed~~ The top edge of the glazing is greater than 36 inches (914 mm) above the floor; and
~~7.~~4. One or more walking surface(s) are within 36 inches (914 mm), measured horizontally ~~of the plane~~ and in a straight line, of the glazing.

Exceptions: ~~Safety glazing for Item 7 is not required for the following installations:~~

~~1. A protective bar 11/2 inches (38 mm) or more in height, capable of withstanding a horizontal load of 50 pounds plf (730 N/m) without contacting the glass, is installed on the accessible sides of the glazing 34 inches to 38 inches (864 mm to 965 mm) above the floor.~~
1. Decorative glazing.
2. When a horizontal rail is installed on the accessible side(s) of the glazing 34 to 38 inches above the walking surface. The rail shall be capable of withstanding a horizontal load of 50 pounds per linear foot (730 N/m) without contacting the glass and be a minimum of 1½ inches (38 mm) in cross sectional height.
~~2~~3. ~~The outboard~~ Outboard panes in insulating glass units or multiple glazing where the bottom exposed edge of the glass is 25 feet (7620 mm) or more above any grade, roof, walking surface or other horizontal or sloped (within 45 degrees of horizontal) (0.78 rad) surface adjacent to the glass exterior.

~~8.~~ **2406.4.4 Glazing in guards and railings.** Glazing in guards and railings, including structural baluster panels and nonstructural in-fill panels, regardless of area or height above a walking surface shall be considered a hazardous location.

~~9. Glazing in walls and fences enclosing indoor and outdoor swimming pools, hot tubs and spas where all of the following conditions are present:~~

~~9.1. The bottom edge of the glazing on the pool or spa side is less than 60 inches (1524 mm) above a walking surface on the pool or spa side of the glazing; and~~
~~9.2. The glazing is within 60 inches (1524 mm) horizontally of the water's edge of a swimming pool or spa.~~

2406.4.5 Glazing and wet surfaces. Glazing in walls, enclosures or fences containing or facing hot tubs, spas, whirlpools, saunas, steam rooms, bathtubs, showers and indoor or outdoor swimming pools where the bottom exposed edge of the glazing is less than 60 inches (1524 mm) measured vertically above any standing or walking surface shall be considered a hazardous location. This shall apply to single glazing and all panes in multiple glazing.

Exception: Glazing that is more than 60 inches (1524 mm), measured horizontally and in a straight line, from the water's edge of a bathtub, hot tub, spa, whirlpool, or swimming pool.

~~10.~~ **2406.4.6 Glazing adjacent stairs and ramps.** Glazing ~~adjacent to~~ where the bottom exposed edge of the glazing is less than 60 inches (1524 mm) above the plane of the adjacent walking surface of stairways, landings between flights of stairs, and ramps shall be considered a hazardous location ~~within 36 inches (914 mm) horizontally of a walking surface; when the exposed surface of the glass is less than 60 inches (1524 mm) above the plane of the adjacent walking surface~~.

Exceptions:

1. The side of a stairway, landing or ramp which has a guard complying with the provisions of Sections 1013 and 1607.7, and the plane of the glass is greater than 18 inches (457 mm) from the railing.
2. Glazing 36 inches (914 mm) or more measured horizontally from the walking surface.

~~11.~~ **2406.4.7 Glazing adjacent the bottom stair landing.** ~~Glazing adjacent to *stairways* within 60 inches (1524 mm) horizontally of the bottom tread of a *stairway* in any direction when the exposed surface of the glass is less than 60 inches (1524 mm) above the nose of the tread.~~ Glazing adjacent the landing at the bottom of a stairway where the glazing is less than 36 inches (914 mm) above the landing and within 60 inches (1524 mm) horizontally of the bottom tread shall be considered a hazardous location.

Exception: ~~Safety glazing for Item 10 or 11 is not required for the following installations where:~~

~~1.~~ The ~~side of a stairway, landing or ramp which has~~ glazing is protected by a guard ~~or handrail, including balusters or in-fill panels,~~ complying with ~~the provisions of~~ Sections 1013 and 1607.7~~;~~ and ~~2. The~~ the plane of the glass is greater than 18 inches (457 mm) from the ~~railing~~ guard.

~~**2406.4.1 Exceptions.** The following products, materials and uses shall not be considered specific hazardous locations:~~

1. ~~Openings in doors through which a 3-inch (76 mm) sphere is unable to pass.~~
2. ~~Decorative glass in Section 2406.4, Item 1, 6 or 7.~~
3. ~~Glazing materials used as curved glazed panels in revolving doors.~~
4. ~~Commercial refrigerated cabinet glazed doors.~~
5. ~~Glass-block panels complying with Section 2101.2.5.~~
6. ~~Louvered windows and jalousies complying with the requirements of Section 2403.5.~~
7. ~~Mirrors and other glass panels mounted or hung on a surface that provides a continuous backing support.~~

PART II – IRC BUILDING/ENERGY

Revise as follows:

R308.4 Hazardous locations. The ~~following~~ locations specified in Sections R308.4.1 through R308.4.7 shall be considered specific hazardous locations for the purposes of glazing:

~~1.~~ **R308.4.1 Glazing in doors.** Glazing in all fixed and operable panels of swinging, sliding and bifold doors shall be considered a hazardous location.

Exceptions:

1. Glazed openings of a size through which a 3-inch diameter (76 mm) sphere is unable to pass.
2. Decorative glazing.

~~2.~~ **R308.4.2 Glazing adjacent doors.** Glazing in an individual fixed or operable panel adjacent to a door where the nearest vertical edge of the glazing is within a 24-inch (610 mm) arc of either vertical edge of the door in a closed position and ~~whose~~ where the bottom exposed edge of the glazing is less than 60 inches (1524 mm) above the floor or walking surface shall be considered a hazardous location.

Exceptions:

1. Decorative glazing.
2. When there is an intervening wall or other permanent barrier between the door and the glazing.
3. Glazing in walls on the latch side of and perpendicular to the plane of the door in a closed position.
4. ~~Glazing adjacent to a door where~~ Where access through the door is to a closet or storage area 3 feet (914 mm) or less in depth. Glazing in this application shall comply with section R308.4.3.
5. Glazing that is adjacent to the fixed panel of patio doors.

~~3.~~ **R308.4.3 Glazing in windows.** Glazing in an individual fixed or operable panel that meets all of the following conditions shall be considered a hazardous location:

~~3.1.~~1. The exposed area of an individual pane is larger than 9 square feet (0.836 m2); and
~~3.2.~~2. The bottom edge of the glazing is less than 18 inches (457 mm) above the floor; and
~~3.3.~~3. The top edge of the glazing is more than 36 inches (914 mm) above the floor; and
~~3.4.~~4. One or more walking surfaces are within 36 inches (914 mm), measured horizontally and in a straight line, of the glazing.

Exceptions:

1. Decorative glazing.
2. When a horizontal rail is installed on the accessible side(s) of the glazing 34 to 38 inches (864 to 965) above the walking surface. The rail shall be capable of withstanding a horizontal load of 50 pounds per linear foot (730 N/m) without contacting the glass and be a minimum of 11/2 inches (38 mm) in cross sectional height.
3. Outboard panes in insulating glass units and other multiple glazed panels when the bottom edge of the glass is 25 feet (7620 mm) or more above grade, a roof, walking surfaces or other horizontal [within 45 degrees (0.79 rad) of horizontal] surface adjacent to the glass exterior.

~~4.~~ **R308.4.4 Glazing in guards and railings.** ~~All glazing in railings regardless of area or height above a walking surface. Included are structural baluster panels and nonstructural infill panels.~~ Glazing in guards and railings, including structural baluster panels and nonstructural in-fill panels, regardless of area or height above a walking surface shall be considered a hazardous location.

~~5. Glazing in enclosures for or walls facing hot tubs, whirlpools, saunas, steam rooms, bathtubs and showers where the bottom exposed edge of the glazing is less than 60 inches (1524 mm) measured vertically above any standing or walking surface.~~

~~**Exception:** Glazing that is more than 60 inches (1524 mm) measured horizontally and in a straight line, from the waters edge of a hot tub, whirlpool or bathtub.~~

~~6. Glazing in walls and fences adjacent to indoor and outdoor swimming pools, hot tubs and spas where the bottom edge of the glazing is less than 60 inches (1524 mm) above a walking surface and within 60 inches (1524 mm), measured horizontally and in a straight line, of the water's edge. This shall apply to single glazing and all panes in multiple glazing.~~

R308.4.5 Glazing and wet surfaces. Glazing in walls, enclosures or fences containing or facing hot tubs, spas, whirlpools, saunas, steam rooms, bathtubs, showers and indoor or outdoor swimming pools where the bottom exposed edge of the glazing is less than 60 inches (1524 mm) measured vertically above any standing or walking surface shall be considered a hazardous location. This shall apply to single glazing and all panes in multiple glazing.

Exception: Glazing that is more than 60 inches (1524 mm), measured horizontally and in a straight line, from the water's edge of a bathtub, hot tub, spa, whirlpool, or swimming pool.

~~7.~~ **R308.4.6 Glazing adjacent stairs and ramps.** Glazing ~~adjacent to~~ where the bottom exposed edge of the glazing is less than 36 inches (914 mm) above the plane of the adjacent walking surface of stairways, landings between flights of stairs, and ramps shall be considered a hazardous location ~~within 36 inches (914 mm) horizontally of a walking surface when the exposed surface of the glazing is less than 60 inches (1524 mm) above the plane of the adjacent walking surface~~.

Exceptions:

1. When a rail is installed on the accessible side(s) of the glazing 34 to 38 inches (864 to 965 mm) above the walking surface. The rail shall be capable of withstanding a horizontal load of 50 pounds per linear foot (730 N/m) without contacting the glass and be a minimum of 11/2 inches (38 mm) in cross sectional height.
2. Glazing 36 inches (914 mm) or more measured horizontally from the walking surface.
~~2. The side of the stairway has a guardrail or handrail, including balusters or in-fill panels, complying with Sections R311.7.6 and R312 and the plane of the glazing is more than 18 inches (457 mm) from the railing; or~~
~~3. When a solid wall or panel extends from the plane of the adjacent walking surface to 34 inches (863 mm) to 36 inches (914 mm) above the walking surface and the construction at the top of that wall or panel is capable of withstanding the same horizontal load as a guard.~~

~~8.~~ **R308.4.7 Glazing adjacent the bottom stair landing.** ~~Glazing adjacent to stairways within 60 inches (1524 mm) horizontally of the bottom tread of a stairway in any direction when the exposed surface of the glazing is less than 60 inches (1524 mm) above the nose of the tread.~~ Glazing adjacent the landing at the bottom of a stairway where the glazing is less than 36 inches (914 mm) above the landing and within 60 inches (1524 mm) horizontally of the bottom tread shall be considered a hazardous location.

Exception~~s~~:

~~1.~~ The ~~side of the stairway has~~ glazing is protected by a guard~~rail or handrail, including balusters or in-fill panels~~, complying with Section~~s R311.7.6 and~~ R312 and the plane of the glass is more than 18 inches (457 mm) from the ~~railing; or~~ guard.

~~2. When a solid wall or panel extends from the plane of the adjacent walking surface to 34 inches (864 mm) to 36 inches (914 mm) above the walking surface and the construction at the top of that wall or panel is capable of withstanding the same horizontal load as a guard.~~

Reason:
PART I- This code change was submitted for the IRC in Palm Springs. While the Committee said they supported the change, they said that the proposal should include the IBC for consistency. There is a companion code change submitted for the IRC. The main purpose of this code change is to eliminate conflicts, create consistency, and ease the use of the safety glazing requirements. Following is a point by point explanation of the changes.

The Exception to Section 2406.1 is relocated from 2406.4.1

Items 1-4 are combined into Section 2406.4.1 and item 5 is relocated to Section 2406.4.5.

Exception 1 to Section 2406.4.2 is from 2406.4.1 and exceptions 4 & 5 provide consistency with the IRC.

Editorial revision in Section 2406.4.3 removes unnecessary language.

Exception 1 to Section 2406.4.3 is from 2406.4.1. Exception 2 is for consistency w/IRC. "Protective bar" implies evaluation by the BO that the "bar" meets certain safety standards. The language already provides a minimum size and rigidity which is sufficient.

Section 2406.4.5 combines all of the language related to hazardous glazing adjacent water, eliminates conflicting or confusing language, and eases interpretation. It also addresses the issue of glazing adjacent a freestanding bathtub by treating it the same as a hot tub, spa or whirlpool. Consistency with the IRC/IBC.

Section 2406.4.6 is largely editorial however there is a difference between the IRC and IBC in that the IRC exempts safety glazing if the glass is in a wall and 34 to 38 inches above the walking surface. No such exception occurs in the IBC so 60 inches is the rule. Exception one is repeated from current item #11. Reference to a handrail with balusters (a guard) is deleted.

(2406.4.7) While the current text says "stairways", clearly what is being regulated is the bottom landing because of the use of the term "horizontally of the bottom tread.. Exception 2 provides the basis for the least restrictive scenario, that being a wall with glazing 34-38 inches above the walking surface.

PART II- This code change was submitted for the IRC in Palm Springs. While the Committee said they supported the change, they said that the proposal should include the IBC for consistency. There is a companion code change submitted for the IBC. The main purpose of this code change is to eliminate conflicts, create consistency, and ease the use of the safety glazing requirements. Following is a point by point explanation of the changes.

Safety glazing requirements have been placed in separate sections, R308.4.1 through R308.4.7, with descriptive titles to aid in use of the code.

R308.4.2 and R308.4.4 are revised for consistency w/IBC

R308.4.5 combines all of the language related to hazardous glazing adjacent water, eliminates conflicting or confusing language, and eases interpretation. It also addresses the issue of glazing adjacent a freestanding bathtub by treating it the same as a hot tub, spa or whirlpool and provides consistency with the IBC.

R308.4.6 is largely editorial but permits unprotected glazing within 36 inches horizontally of the plane of the walking surface. Exception 3 reduces the height when safety glazing is required from 60 inches to 34-38 inches if the glazing is in a wall or panel. Since glazing usually means windows and since windows are typically in a wall, the exception is most always the rule. It is puzzling that such a wall would have an upper limit. Does the glazing become hazardous again if the wall is 40 inches high? An average of 36 inches was chosen as the minimum height for the wall consistent with the height of a guard. Exception 1 exempts safety glazing if a rail is installed 34 to 38 inches high regardless of whether or not there are balusters or in-fill panels and without regard to the proximity of the glass. Since this is the least restrictive exception, it makes no sense to require glazing to be 18 inches from a guard or to require a guard have balusters or in-fill panels in Exception 2. Exception 3 language is incorporated into the main section since the standard installation is for the window to be installed in a wall. This exception says if a wall places the window 34 to 38 inches above the walking surface safety glazing is not required. There is no legitimate reason to have an upper limit.

(R308.4.7) While the current text says "stairways", clearly what is being regulated is the bottom landing because of the use of the term "horizontally of the bottom tread. Exception 2 provides the basis for the least restrictive scenario, that being a wall with glazing 34-38 inches above the walking surface and is incorporated into the main section since the standard installation is for the window to be installed in a wall. The exception says if a wall places the window 34 to 38 inches above the walking surface safety glazing is not required. There is no legitimate reason to have an upper limit.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: EVANS-S1-2406.

S219–09/10
2406.2; IRC R308.3.1

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

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC STRUCTURAL

Revise as follows:

2406.2 Impact Test. Where required by other sections of this code, glazing shall be tested in accordance with CPSC 16 CFR 1201. Glazing shall comply with the test criteria for Category ~~I or~~ II ~~as~~, unless otherwise indicated in Table 2406.2(1)

> **Exception:** Glazing not in doors or enclosures for hot tubs, whirlpools, saunas, steam rooms, bathtubs and showers shall be permitted to be tested in accordance with ANSI Z97.1. Glazing shall comply with the test criteria for Class A ~~or B as~~ , unless otherwise indicated in Table 2406.2(2).

PART II – IRC BUILDING/ENERGY

Revise as follows:

R308.3.1 Impact Test. Where required by other sections of this code, glazing shall be tested in accordance with CPSC 16 CFR 1201. Glazing shall comply with the test criteria for Category ~~I or~~ II ~~as~~, unless otherwise indicated in Table R308.3.1(1).

> **Exception:** Glazing not in doors or enclosures for hot tubs, whirlpools, saunas, steam rooms, bathtubs and showers shall be permitted to be tested in accordance with ANSI Z97.1. Glazing shall comply with the test criteria for Class A ~~or B as~~ , unless otherwise indicated in Table R308.3.1(2).

Reason: Not all of the hazardous locations are indicated in the Table and as such the performance or test criteria for the safety glazing are not specified in the Code. The proposal uses the higher test criteria as the default value and the Table relaxes the requirement for specific applications.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: KOFFELL-S2-2406.2

S220–09/10
2406.4

Proponent: John D. McGee, Binswanger Glass Training Center, representing self

Revise as follows:

2406.4 Hazardous locations. The following shall be considered specific hazardous locations requiring safety glazing materials:

(No change to items 1 through 11)

12. <u>Ceiling mounted glass mirrors panels shall be no larger than 18 square feet (1.672 m^2), shall have CPSC 16 CFR 1201 Category II safety backing applied to back of mirror, and shall apply for annealed, heat strengthened, or tempered glass mirrors. Both adhesive fastening and mechanical retention systems shall be used for all ceiling-mounted glass mirrors. Substrate and mechanical fasteners shall be of suitable strength to support a 4:1 ratio of the weight of the mirror. Design by a registered design professional shall be provided where required by the Building Official. The adhesive shall be applied in accordance with the manufacturer's instructions and the mechanical fastener or retainer shall capture a minimum glass bite of ½-inch (12.7 mm) on all mirror edges. Where rosette-type fasteners are used, holes shall be no closer to the mirror edge than the mirror thickness plus ¼-inch (6.4 mm). All holes drilled into the mirror (i.e., screws, light fixtures, etc.) shall have adequate radius to prevent stress cracks. Screw- and bolt-type fasteners shall be of stainless steel.</u>

Reason: With over 100 nationwide locations, Binswanger Glass regularly receives numerous requests to mount mirrors on ceilings, which is an extremely dangerous practice. The ICC indicates no code exists to address this application, even though poorly mounted ceiling mirrors can kill if they fall off. The revision indicated above is only a starting point for council consideration; detailed assessment by other glazing specialists is urged.

Cost Impact: Variable based on glass thickness.

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

ICCFILENAME: MCGEE-S1-2406.4.DOC

S221–09/10
2407.1.1

Proponent: Thomas B. Zuzik Jr., Artistic Railing Inc., representing self

Revise as follows:

2407.1.1 Loads. The <u>glass</u> ~~panels and their support system~~ shall be designed to withstand the loads specified in Section<u>s 1607.7.1 and 1607.7.1.2.</u> ~~1607.7.~~ A safety factor of four shall be used.

Reason: I am requesting that 2407.1.1 be revised to more clearly define which loads the glass is required to meet when glass is used in guards or handrails. Currently 2407.1.1 states that the glass be DESIGNED to withstand the loads of 1607.7, however 1607.7.1.2 is not directly a design load, but arguably a test load, as the text states "shall be able to resist" without the notation requirement of design.

"**1607.7.1.1 Concentrated load.** Handrails and *guards* **SHALL BE ABLE** to **RESIST A SINGLE CONCENTRAITED LOAD OF 200 POUNDS…"**

Further more the 2006 IBC commentary for section 2407.1.1 states in the last sentence " It is not intended that an in-place glass guard or handrail system be tested for or capable of withstanding four times the design load"

This confusion with the current language in 2407.1.1 brings forth the question is the code directing us to use only the loads in 1607.7 that are design loads?

The intent is a four times design load as 1607.7.1 directs the reader to section 2407 when glass is part of a guard or handrail system. The four times safety factor would require the guard or handrail to be designed to resist a 200 plf (0.73 kN/m) rather than the set 50 plf. However, 1607.7.1.2 which notes a 200 pound concentrated would be elevated to a 800 pound load which is not an intended requirement.

The removal of the language "panels and their support system" from the text is redundant and not needed as the glass is required to meet the loads and when installed section 1607.7.1 requires the loads to be transfer through the support structure.

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

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

ICCFILENAME: ZUZIK-S1-2407.1.1

S222–09/10
Figure 721.5.1(2), Figure 721.5.1(3), Table 2506.2, Table 2507.2; IRC R702.2.1, R702.2.2, R702.3.1

Proponent: Michael Gardner, representing Gypsum Association

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I- IBC STRUCTURAL

1. Revise as follows:

FIGURE 721.5.1(2)
GYPSUM WALLBOARD PROTECTED
STRUCTURAL STEEL COLUMNS WITH SHEET STEEL
(No change to figure)

2. Type X gypsum wallboard in accordance with ASTM ~~C 36~~ C 1396. For single-layer applications, the wallboard shall be applied vertically with no horizontal joints. For multiple-layer applications, horizontal joints are permitted at a minimum spacing of 8 feet, provided that the joints in successive layers are staggered at least 12 inches. The total required thickness of wallboard shall be determined on the basis of the specified fire-resistance rating and the weight-to-heated-perimeter ratio (*W/D*) of the column. For fire-resistance ratings of 2 hours or less, one of the required layers of gypsum wallboard may be applied to the exterior of the sheet steel column covers with 1-inch-long Type S screws spaced 1 inch from the wallboard edge and 8 inches on center. For such installations, 0.0149-inch minimum thickness galvanized steel corner beads with 1½-inch legs shall be attached to the wallboard with Type S screws spaced 12 inches on center.

(No change to footnotes not shown)

FIGURE 721.5.1(3)
GYPSUM WALLBOARD PROTECTED STRUCTURAL STEEL COLUMNS WITH STEEL STUD/SCREW ATTACHMENT SYSTEM
(No change to figure)

3. Type X gypsum wallboard in accordance with ASTM ~~C 36~~ C 1396. For single-layer applications, the wallboard shall be applied vertically with no horizontal joints. For multiple-layer applications, horizontal joints are permitted at a minimum spacing of 8 feet, provided that the joints in successive layers are staggered at least 12 inches. The total required thickness of wallboard shall be determined on the basis of the specified fire-resistance rating and the weight-to-heated-perimeter ratio (*W/D*) of the column.

(No change to footnotes not shown)

TABLE 2506.2
GYPSUM BOARD MATERIALS AND ACCESSORIES

MATERIAL	STANDARD
~~Exterior Soffit Board~~	~~ASTM C 931~~
~~Gypsum backing board and gypsum shaftliner board~~	~~ASTM C 442~~
~~Gypsum ceiling board~~	~~ASTM C 1395~~
~~Gypsum sheathing~~	~~ASTM C 79~~
~~Gypsum wallboard~~	~~ASTM C 36~~
~~Predecorated gypsum board~~	~~ASTM C 960~~
~~Water-resistant gypsum backing board~~	~~ASTM C 630~~

(Portions of Table not shown remain unchanged)

TABLE 2507.2
LATH, PLASTERING MATERIALS AND ACCESSORIES

MATERIAL	STANDARD
~~Gypsum lath~~	~~ASTM C 37~~
~~Gypsum base for veneer plasters~~	~~ASTM C 588~~

(Portions of Table not shown remain unchanged)

PART II- IRC

1. Revise as follows:

R702.3.1. Materials. All gypsum board materials and accessories shall conform to ASTM ~~C 36, C 79~~, C 475, C 514, ~~C 630~~, ~~C 931, C 960,~~ C 1002, C 1047, C 1177, C 1178, C 1278, ~~C 1395~~, C 1396, or C 1658 and shall be installed in accordance with the provisions of this section. Adhesives for the installation of gypsum board shall conform to ASTM C 557.

R702.2.1 Gypsum plaster. Gypsum plaster materials shall conform to ASTM C 5, C 28, C 35, ~~C 37~~, C 59, C 61, C 587, ~~C 588~~, C 631, C 847, C 933, C 1032 and C 1047 and shall be installed or applied in conformance with ASTM C 843 and C 844. Gypsum lath or gypsum base for veneer plaster shall conform to ASTM C 1396. Plaster shall not be less than three coats when applied over metal lath and not less than two coats when applied over other bases permitted by this section, except that veneer plaster may be applied in one coat not to exceed 3/16 inch (4.76 mm) thickness, provided the total thickness is in accordance with Table R702.1(1).

R702.2.2 Cement plaster. Cement plaster materials shall conform to ASTM ~~C 37~~, C 91 (Type M, S or N), C 150 (Type I, II, and III), ~~C 588~~, C 595 [Type IP, I (PM), IS and I (SM), C 847, C 897, C 926, C 933, C 1032, C 1047 and C 1328, and shall be installed or applied in conformance with ASTM C 1063. Gypsum lath shall conform to ASTM C 1396. Plaster shall not be less than three coats when applied over metal lath and not less than two coats when applied over other bases permitted by this section, except that veneer plaster may be applied in one coat not to exceed 3/16 inch (4.76 mm) thickness, provided the total thickness is in accordance with Table R702.1(1).

Reason:
PART I- In December 2004, a single composite ASTM International reference Specification C 1396/ C1396M, *Specification for Gypsum Board*, replaced nine individual standards previously used to designate specific gypsum board products employed in residential and non-residential construction. The nine withdrawn standards are no longer being maintained by ASTM.

ASTM C 1396 is presently contained in Tables 2506.2 and 2507.2 of IBC Chapter 25 as a reference standard.

Proposal is submitted to reflect the use of the composite standard. It eliminates references to standards that are no longer available.

PART II- In December 2004, a single composite ASTM International reference Specification C 1396/ C1396M, *Specification for Gypsum Board*, replaced nine individual standards previously used to designate specific gypsum board products employed in residential and non-residential construction. The nine withdrawn standards are no longer being maintained by ASTM.

ASTM C 1396 is presently contained in the IRC as a reference standard.

Proposal is submitted to reflect the use of the composite standard. It eliminates references to standards that are no longer available.

Proposal is submitted to remove the reference to the withdrawn standards.

Section R702.2.1 has been modified to reflect the elimination of the individual standards for gypsum lath and gypsum veneer base.

Section R702.2.2 has been modified to reflect the elimination of the individual standard for gypsum lath. The reference to the veneer base standard (C 588) has been eliminated, as it is not appropriate to install cement plaster directly to gypsum veneer base.

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

PART I- IBC:

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

PART II- IRC:

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

ICCFILENAME: GARDNER-S1-Figure 721.5.2(2).doc-RB2-702.2-RB4-702.3

S223–09/10
2509.2

Proponent: Jose M. Estrada, representing USG Corporation

1. Revise as follows:

2509.2 Base for Tile. Glass mat water-resistant gypsum backing panel, discrete nonasbestos fiber-cement interior substrate sheets, water-resistant fiber-reinforced gypsum backers or nonasbestos fiber-mat reinforced cement substrate sheets in compliance with ASTM C 1178, C 1288, C 1278 or C 1325 and installed in accordance with manufacturer recommendations shall be used as a base for the wall tile in tub and shower areas and wall and ceiling panels in shower areas. Water-resistant gypsum backing board shall be used as a base for tile in water closet compartment walls when installed in accordance with GA -216 or ASTM C 840 and manufacturers recommendations. Regular gypsum wallboard is permitted under tile or wall panels in other wall and ceiling areas when installed in accordance with GA-216 or ASTM C 840.

Reason: The purpose of this proposal is to include an ASTM material standard for current provisions of the IBC. ASTM C 1278 products are engineered and manufactured specifically for interior water-resistant backing. The proposed ASTM material standard has been recognized by the International Residential Code (IRC) since the 2007 Supplement. The water-resistant products complying with this ASTM standard have a demonstrated track record, which has been documented substantially and historically, in consensus industry publications such as the TCA Handbook for Ceramic Tile Installation, published by the Tile Council of North America, where the ASTM C1278 products have been recognized for use in wet areas, including their use as a base for the wall tile in tub and shower surrounds since 2007. The wall and floor designs for the ASTM

C1278 products listed in the TCA Handbook for wet area application are equivalent to those of ASTM C 1178, C 1288 and C 1325 products. The products covered under ASTM C 1278 for use as a base for tile have a proven track record in the field, where hundreds of millions of feet have been installed since its release to the market. The inclusion of this standard will allow for more competitive product bidding in turn reducing overall construction costs.

Bibliography:

The Tile Council of North America. 2009 TCA Handbook for Ceramic Tile Installation. Anderson, SC: TCNA, 2009.

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

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

ICCFILENAME: ESTRADA-S1-2509.2

S224–09/10
2509.2; IRC R702.4.2

Proponent: Keith Poerschke representing National Gypsum Company

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE IBC STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I – IBC

Revise as follows:

2509.2 Base for tile. Glass mat water-resistant gypsum backing panels, discrete nonasbestos fiber-cement interior substrate sheets or nonasbestos fiber-mat reinforced cementitious backer units ~~cement substrate sheets~~ in compliance with ASTM C 1178, C 1288 or C 1325 and installed in accordance with manufacturer recommendations shall be used as a base for wall tile in tub and shower areas and wall and ceiling panels in shower areas. Water-resistant gypsum backing board shall be used as a base for tile in water closet compartment walls when installed in accordance with GA-216 or ASTM C 840 and manufacturer recommendations. Regular gypsum wallboard is permitted under tile or wall panels in other wall and ceiling areas when installed in accordance with GA-216 or ASTM C 840.

PART II – IRC

Revise as follows:

R702.4.2 Fiber-cement, fiber-mat reinforced cement, glass mat gypsum backers and fiber-reinforced gypsum backers. Fiber-cement, fiber-mat reinforced cementitious backer units ~~cement~~, glass mat gypsum backers or fiber-reinforced gypsum backers in compliance with ASTM C 1288, C 1325, C 1178 or C 1278, respectively, and installed in accordance with manufacturers' recommendations shall be used as backers for wall tile in tub and shower areas and wall panels in shower areas.

Reason: The ASTM C17 subcommittee, which is responsible for the C1325 standard, voted and approved a title change and other modifications in 2008. The title of ASTM C 1325 was changed to cementitious backer units, which is more representative of how these products have been referred to for decades. This change should be reflected in the language of IBC Section 2509.2 and IRC R702.4.2. Additional changes were made to the standard to more accurately describe what constitutes failure in the test methods and to bring the requirements of the standard more in line with the ANSI standard for the product.

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

PART I – IBC STRUCTURAL

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

PART II – IRC BUILDING/ENERGY

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

ICCFILENAME: POERSCHKE-S1-2509.2

S225–09/10
2510.6; IRC R703.6.3

Proponent: Kimdolyn Boone, representing DuPont Building Innovations

THIS IS A 2 PART CODE CHANGE. PART I WILL BE HEARD BY THE STRUCTURAL COMMITTEE. PART II WILL BE HEARD BY THE IRC BUILDING/ENERGY COMMITTEE. SEE THE TENTATIVE HEARING ORDERS FOR THESE COMMITTEES.

PART I- IBC STRUCTURAL

Revise as follows:

2510.6 Water-resistive barriers. Water-resistive barriers shall be installed as required in Section 1404.2 and, where applied over wood-based sheathing, shall include a water-resistive vapor-permeable barrier with a performance at least equivalent to two layers of Grade D paper. <u>The individual layers shall be installed such that each layer is installed ship lapped fashion and any flashing (installed in accordance with Section 1405.4) intended to drain to the water-resistive barrier is directed between the layers.</u>

> **Exception:** Where the water-resistive barrier that is applied over wood-based sheathing has a water resistance equal to or greater than that of 60-minute Grade D paper and is separated from the stucco by an intervening, substantially nonwater-absorbing layer or drainage space.

PART II- IRC BUILDING/ENERGY

Revise as follows:

R703.6.3 Water-resistive barriers. Water-resistive barriers shall be installed as required in Section R703.2 and, where applied over wood-based sheathing, shall include a water-resistive vapor-permeable barrier with a performance at least equivalent to two layers of Grade D paper. <u>The individual layers shall be installed such that each layer is installed ship lapped fashion and any flashing (installed in accordance with Section R703.8) intended to drain to the water-resistive barrier is directed between the layers.</u>

> **Exception:** Where the water-resistive barrier that is applied over wood-based sheathing has a water resistance equal to or greater than that of 60 minute Grade D paper and is separated from the stucco by an intervening, substantially nonwater-absorbing layer or designed drainage space.

Reason: Clarification of current requirement of the code

Currently the code requires 2 layers be installed but does not address how these layers are installed. There are 2 methods of installation and these methods provide different performance Attributes.

1. 2 layer system – Each layer of WRB is individually installed in a ship lapped fashion. The interior layer is configured to form a continuous drainage plane and is integrated with the flashing.
2. ply system - Both layers are installed and lapped together. The outboard layer is integrated the flashing.

The benefits of using two layers of water resistive barrier (WRB) can only be realized if the method and manner of the installation establish a continuous drainage plane, separated from the stucco.

In a 2-layer system each layer provides a separate & distinct function. The primary function of the inboard layer is to resist water penetration into the cavity. This layer should be integrated with window and door flashings, the weep screed at the bottom of the wall and any through-wall flashings or expansion joints. This layer becomes the drainage plane for incidental water. The primary function of the outboard layer (layer that comes in contact with the stucco) is to separate the stucco from the water resistive barrier. This layer has historically been called a sacrificial layer, intervening layer or bond break layer.

When structures are detailed as 2 layer systems, it is possible to install each layer to meet its intended function. A continuous drainage plane can be established on the inboard layer. However, this is not the case with a 2-ply system. A 2-ply system functions as a single layer and the only benefit provided is additional water resistance. If additional water holdout is the only benefit you are seeking then a superior 1 layer WRB should be sufficient.

This dual function concept has also been noted in building science research studies. A few of the conclusions in the Final Report For Energy Efficient, Mold-Resistant Materials and Construction Practices for New California Homes (California Energy Commission August 2008) are as follows:

> *A Capillary break between stucco and WRB is required for optimal gravity drainage.*
> *Double layer provides space, outer layer provides bond break.*
> *Sill pan drains to the interior layer (the functional WRB)*

Cost Impact: For installations that currently install 2 ply systems this may have a slight increase in the labor costs.

PART I-IBC STRUCTURAL

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

PART II-IRC BUILDING/ENERGY

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

ICCFILENAME: BOONE-S1-2510.6-RB-2-R703.6.3